Geotechnical Evaluation
Austin Bluffs Tank and Pump Station
Colorado Springs, Colorado
Revised

Prepared For:

Black & Veatch

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Attention: Mr. Dan Kugler

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PURPOSE AND SCOPE OF STUDY

This report presents the results of a geotechnical evaluation performed by GROUND Engineering Consultants, Inc. (GROUND) for Black & Veatch in support of design of the proposed water tank and pump station in Colorado Springs, Colorado. Our study was conducted in general accordance with a portion of GROUND's Propo
dated August 30, 2024 and the associated contract documents.

A field exploration program was conducted to obtain information on the subsurface conditions. Material samples obtained during the subsurface exploration were tested in the laboratory to provide data on the engineering characteristics of the on-site soils and bedrock. The results of the field exploration and laboratory testing are presented herein.

This report has been prepared to summarize the data obtained and to present our findings and conclusions based on the proposed development/improvements and the subsurface conditions encountered. Design parameters and a discussion of engineering considerations related to the proposed improvements are included herein. This report should be understood and utilized in its entirety; specific sections of the text, drawings, graphs, tables, and other information contained within this report are intended to be understood in the context of the entire report. This includes the *Closure* section of the report which outlines important limitations on the information contained herein.

This report was prepared for design purposes of Black & Veatch based on our understanding of the proposed project at the time of preparation of this report. The data, conclusions, opinions, and geotechnical parameters provided herein should not be construed to be sufficient for other purposes, including the use by contractors, or any other parties for any reason not specifically related to the design of the project. Furthermore, the information provided in this report was based on the exploration and testing methods described below. Deviations between what was reported herein and the actual surface and/or subsurface conditions may exist, and in some cases those deviations may be significant.

PROPOSED CONSTRUCTION

The following project documents were provided to GROUND by Black & Veatch for our development of the parameters and considerations in this report:

- Site Plan: *Austin Bluffs 5 MG Tank Replacement*. 30% Design. Prepared by Black & Veatch. Dated March 2025.
- Scope of Work: *FW: CSU Austin Bluffs/Centennial PS/Pln RFP.* Emailed on July 23, 2024 from Black & Veatch.
- Pre-Proposal Conference Agenda: Colorado Springs Utilities. Austin Bluffs, Centennial, and Portal Park Pump Stations and Transmission. Pipelines Study/Alternative Analysis, Design and Engineering Services During Construction. Dated July 9, 2024.

Based on these documents and correspondence with Black & Veatch, we understand that proposed construction will include a new pump station with an approximate 5,850 square-foot footprint and a new 5-million-gallon water tank with a diameter of 163 feet. The existing tank at this location will be demolished and the new tank footprint will be shifted 10 feet to the west of the existing tank. The proposed floor elevation of the tank is 6,660 feet with an overflow water level of 6,692 feet (NAVD88 datum). We understand the tank will be about half to three-quarters full most of the time but it may be drained for a few months at a time.

The proposed tank will be roughly at the same elevation as the existing tank and surrounding ground surface. The proposed pump station finished floor is planned to be 6,660 feet, requiring foundation walls 10 to 12 feet tall on the west side of the building. Below grade piping on the order of 5 to 10 feet deep is also planned. Below grade vaults are not planned.

If our described understanding/interpretation of the proposed project is incorrect or project elements differ in any way from that expressed above, including changes to improvement locations, dimensions, orientations, loading conditions, elevations/grades, etc., and/or additional buildings/structures/site improvements are incorporated into this project, either after the original information was provided

to us or after the date of this report, GROUND or another geotechnical engineer must be retained to reevaluate the conclusions and parameters presented herein.

Performance Expectations Based on our experience with similar facilities in the project area and provided documents (listed above), we assume that total post-construction, tank and pump station foundation movements on the order of 1 to 2 inches are acceptable to, and anticipated by the owner, as are the resultant distress and maintenance measures. We understand that some types of tanks can tolerant significantly larger total post-construction movements, on the order of 6 inches in some cases. Therefore, if greater estimates of post-construction movement for the tank or the pump station are acceptable, GROUND should be notified so that the parameters and conclusions in this report can be revised. Additionally, GROUND will be available to discuss the risks and remedial approaches outlined in this report, as well as other potential approaches, upon request, if post-construction movements of these magnitudes are not acceptable and anticipated.

SITE CONDITIONS

At the time of our exploration, the project site was a relatively flat depression cut out from the upper portion of an irregular, northeast-trending ridge. The area was occupied in large part by an existing, 5-million-gallon, steel tank. and enclosed by chain-link fencing. The flanks of the ridge descended with about 300 feet of relief in the general project area.

On the southern side of the tank, there was a roughly 10-foot high, shotcrete-covered, approximately 1:1 (horizontal : vertical) descending slope with 2 rows of soil nails or tie back anchors. Relatively steep slopes also descended 10 to 15 feet to west of the tank

and a short distance to the east. The latter exposed sandstone bedrock. Another relatively steep, approximately 30-foot slope ascended northward of the site.

Most slopes in the area stood at angles from about 1:1 to 2:1 (horizontal : vertical) and supported and supported bushes, trees, and other vegetation.

The area surrounding the site consisted of a below-grade reservoir to the north of the existing tank, residential areas to the northeast and northwest, an antenna to the southwest, and undeveloped rugged terrain on all other sides.

Review historical aerial imagery available on Google Earth and the USGS Earth Explorer website[1] indicated that the below grade reservoir was constructed in mid-1950s and the existing tank was constructed in the 1960s. It appears that fill associated with the tank construction was pushed out roughly 50 feet onto the edge of the steep slopes to the east of the tank, possibly steepening and adding weight to the uppermost portion of the slope. The surrounding residential development appeared to be developed in the early 2000s.

SUBSURFACE EXPLORATION

Subsurface exploration for the project was conducted in March 2025. A total of 7 test holes were drilled within or near the tank footprint with a conventional, truck-mounted drilling rig to evaluate the subsurface conditions as well as to retrieve samples for laboratory testing and analysis for this scope. At Test Holes 1, 4, 5, and 6, 4-inch diameter solid stem auger was utilized. At Test Holes 2, 3, and 7, 3¼-inch inner diameter hollow-stem auger and NQ wireline coring equipment was utilized. Due to the nature (relatively poor cementation) of the material, much of the cored material washed out during the drilling process, and only limited material was recovered. Ground surface elevations of the test holes were estimated from the client provided plans. The test holes were advanced to depths of about 30.3 to 50 feet below existing grades corresponding to

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[1] Aerial Photo Single Frame Images. Courtesy of the U.S. Geological Survey. EarthExplorer. <https://earthexplorer.usgs.gov/>. Accessed on April 22, 2025.

elevations of about 6634.7 to 6610 feet. A GROUND engineer directed the subsurface exploration, logged the borings in the field, and prepared the samples for transport to our laboratory.

Samples of the subsurface materials were retrieved with a 2-inch inner diameter California liner sampler, a 1⅜-inch inner diameter Standard Penetration Test sampler. The samplers were driven into the substrata with blows from a 140-pound hammer falling 30 inches, in general accordance with (in the case of the 1⅜-inch sampler) the Standard Penetration Test described by ASTM Method D1586. Penetration resistance values, when properly evaluated, indicate the relative density or consistency of soils. Depths at which the samples were obtained and associated penetration resistance values are shown on the test hole logs. Rock quality designations (RQD) were estimated where wireline coring was performed and are shown on the test hole logs.

The approximate locations of the test holes are shown in Figure 1. Summary logs of the test holes are provided in Figure 2. A legend and notes are provided in Figure 3. Detailed logs of test holes are provided in Appendix A. Representative photographs of the recovered core are presented in Appendix B.

LABORATORY TESTING

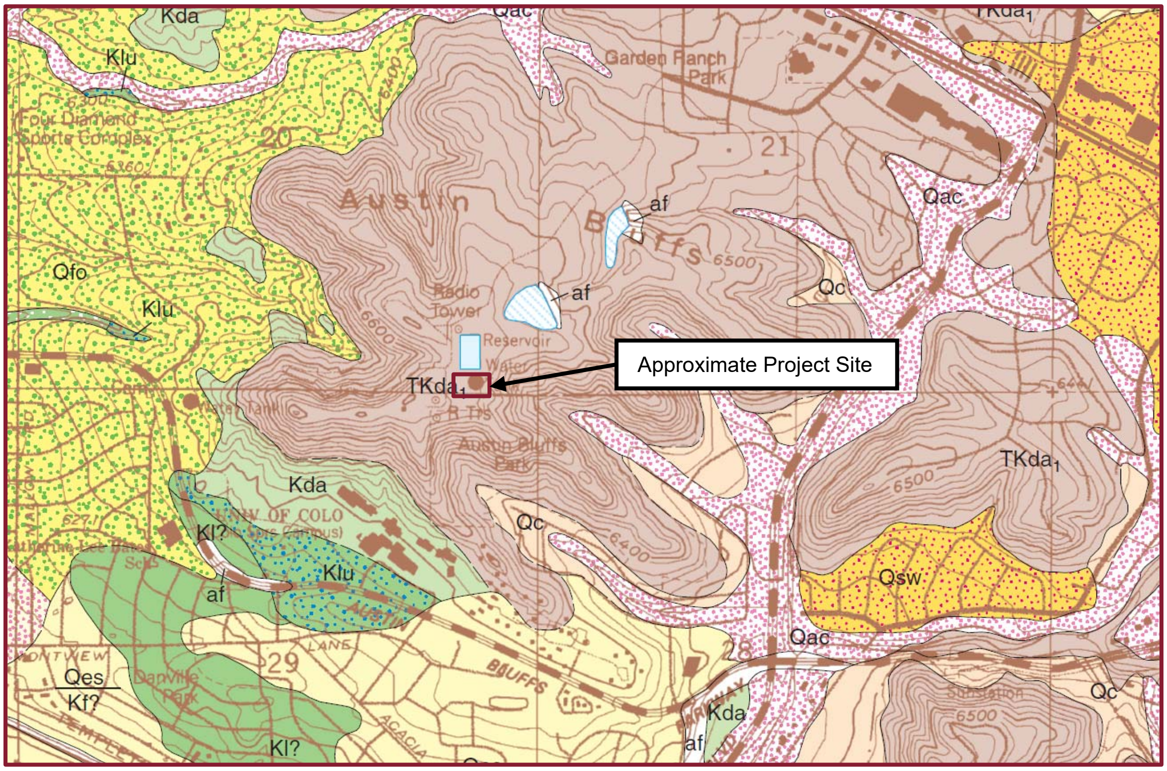
Samples retrieved from our borings were examined and visually classified in the laboratory by the project engineer. Laboratory testing of soil samples included standard property tests, such as natural moisture contents, dry unit weights, grain size analyses, and Atterberg limits. Swell-consolidation, unconfined compressive strength, water-soluble sulfate, and a suite of corrosivity tests were completed on selected samples, as well. Laboratory tests were performed in general accordance with applicable ASTM protocols. Results of the laboratory testing program are summarized in Tables 1 and 2. Gradation and hydrometer plots are provided in Figures 4 through 7. Plots of the swell-consolidation testing are provided in Figures 8 through 12.

SUBSURFACE CONDITIONS

Geologic Setting Published geologic maps, such as Thorson, Carroll, and Morgan (2001),[2] depict the site as underlain by Facies unit one of the Upper part of the Dawson Formation (**TKda**$_1$). A portion of that map is reproduced below. That unit of the Dawson Formation is described as consisting of cliff-forming, very coarse sandstone and conglomerate with claystone interbeds.

Alluvial deposits, in the project area typically consist of fine to coarse sands with gravels. Coarse gravels and cobbles are present locally. These coarse gravels and cobbles can be relatively large and require special handling or processing.

The Dawson Formation in the project area consist of consist of commonly cliff-forming, very coarse sandstone and conglomerate with claystone interbeds. The claystones can be moderately to highly expansive. The unit also includes lenses of sandstone that are well-cemented, very hard, and can be difficult to excavate.



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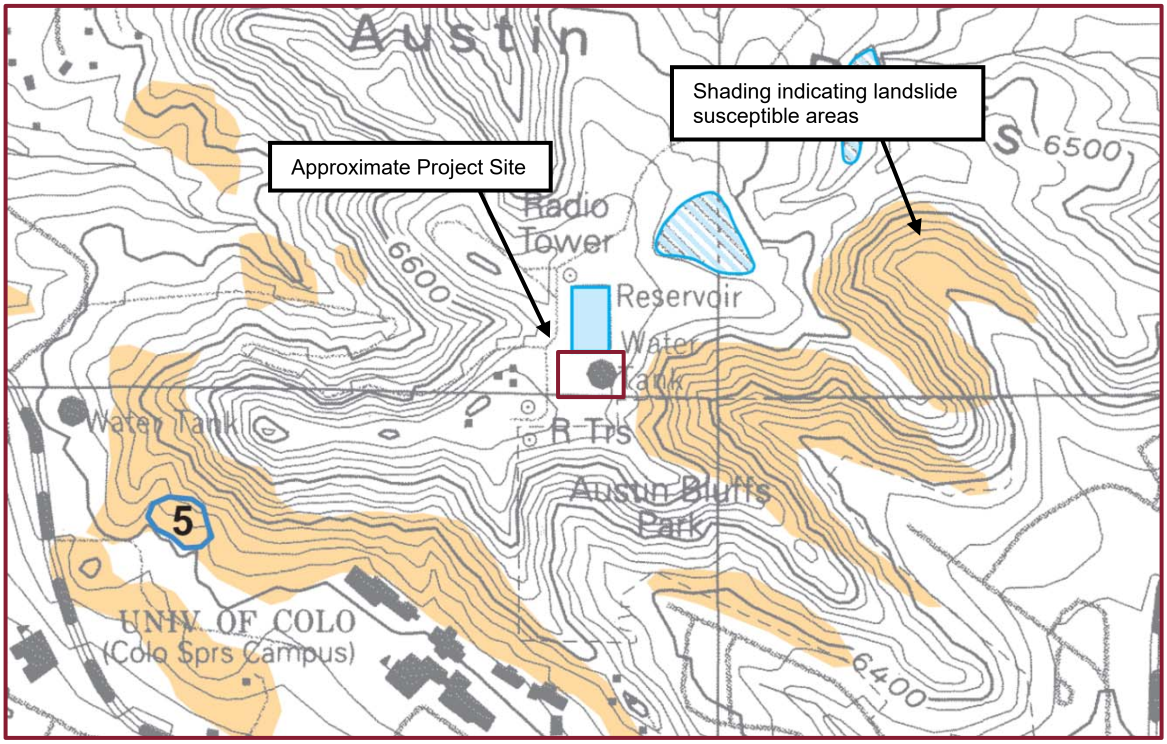
[2] Jon P. Thorson, Christopher J. Carroll, Matthew L. Morgan. 1977, *Geologic Map of the Pikeview Quadrangle, El Paso County, Colorado*, Colorado Geological Survey, Open File Map 01-3, 1:24,000.

Landsliding Risk Mapping by White and Wait (2003)[3] indicated that the slopes generally to the east of the site are landslide-susceptible, as were the lower slopes to the south, southwest, and west of the site. Landslide susceptible areas are indicated by the orange shading on a portion of the White and Wait map reproduced below. "Landslide-susceptible" was defined as having a higher risk of slope instability than other areas of Colorado Springs, based on slope angle, surficial/bedrock geology, geomorphology, and landslide history. A landslide from an unpublished source is mapped 1,900 feet west-southwest of the site near the University of Colorado at Colorado Springs campus, as indicated by the blue-outlined area.



The steep slopes to the east of the site were mapped to be underlain the same Dawson Formation bedrock as the project site. Based on the Thorson, Carroll, and Morgan mapping,[2] the bedrock in the project area dips (is tilted) eastward at shallow angles, i.e., at angles less steep than the eastward descending slopes near the site, thus allowing the bedding surfaces to 'daylight' on the slopes east of the site. This condition supports the

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[3] Jonathan L. White and T.C. Wait. 2003, *Colorado Springs Landslide Susceptibility Map, El Paso County, Colorado.* Colorado Geological Survey. Map Series 42.

conclusion that the slopes to the east of the site are relatively susceptible to slope failures/landsliding, particularly if they were to become saturated.

A detailed slope stability analysis and geologic hazard study was not performed as part of this scope, although we can provide a fee estimate for a slope stability analysis upon request. Based our initial field observations and review of readily available aerial photographs, we did not identify features indicative of large scale/deep seated, slope failures at the project site or on the slopes to the east. However, we observed relatively minor slumps, and moderate to severe erosion and mass wasting along the main drainage channel east of the site. Given the approximately 90- to 160-foot setbacks of the proposed tank from the tops-of-slope, and the performance of the existing tank, we consider the likelihood of landsliding or slumping to affect the proposed tank during its design-life to be low, but no zero. Ongoing erosion on the slopes to the east likely will continue, and tank maintenance should include periodic observation of those slopes for erosion and signs of slope movements.

Site Subsurface Conditions In general, the test holes at the site penetrated approximately 2 to 3 inches of topsoil[4] before penetrating either fill soils that extended to depths of 1 to 2 feet (Test Holes 1, 3, 5, 6, and 7) and/or sands that extended to depths of about 7 to 9 feet (Test Holes 2, 4, and 6). These materials were underlain by claystone and sandstone bedrock that extended to the depths explored. The upper few feet of the bedrock were weathered at Test Holes 1, 5, and 6.

Fill soils were recognized in most of the test holes and could be encountered elsewhere at the site, particularly beneath the existing tank after its demolition. Delineation of the complete lateral and vertical extents of the fills at the site and their compositions was beyond our present scope of services. If more detailed information regarding fill extents and compositions at the site are of significance, they should be evaluated using test pits.

Fill consisted of fine to coarse, clayey to silty sands with local gravels. They were non- to slightly plastic, dry to moist, and light brown to brown to gray-brown in color.

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[4] ‘Topsoil’ as used herein is defined geotechnically. The materials so described may or may not be suitable for landscaping or as a growth medium for such plants as may be proposed for the project.

Sands consisted of fine to coarse, clayey to silty sands with local gravels. They were non- to slightly plastic, loose to dense, dry to moist, and light brown to brown to gray-brown in color. Iron staining was noted locally.

Weathered Claystones were moderately plastic, medium hard, moist, and gray in color. Secondary carbonates were noted locally.

Sandstone and Claystone Bedrock consisted of highly interbedded claystones and fine to coarse sandstones. They were poorly to well-cemented, non- to moderately plastic, hard to very hard, slightly moist to very moist, and light brown to brown to gray to dark gray in color. Iron staining was noted commonly, lignite and secondary carbonates were noted locally.

We interpret the fill soils to be materials placed during the excavation of the tank site and construction of the existing tank. The native sands we interpret to be alluvial deposits. We interpret the sandstone and claystone bedrock to be Dawson Formation materials.

Groundwater was not encountered in the test holes at the time of drilling. The test holes were backfilled upon drilling completion per Code of Colorado Regulations (2 CCR 402-2). Groundwater levels can be expected to fluctuate, however, in response to annual and longer-term cycles of precipitation, irrigation, surface drainage, nearby rivers and creeks, land use, and the development of transient, perched water conditions. The groundwater observations performed during our exploration must be interpreted carefully as they are short-term and do not constitute a groundwater study. In the event Black & Veatch desires additional/repeated groundwater level observations, GROUND should be contacted; additional exploration and fees will be necessary in this regard.

Swell-Consolidation Testing of selected samples of on-site soils recovered from the test holes indicated swells up to about 7.0 percent under surcharge loads approximating in-place overburden pressures. Significant consolidations were not measured during our swell-consolidation testing. (See Table 1.)

ENGINEERING SEISMICITY

Based on extrapolation of available data to depth and our experience in the project area, we consider the tank site likely to meet the criteria for a Seismic Site Classification of **C** according to the ASCE 7-16 (Table 20.3-1). (Exploration and/or shear wave velocity testing to a depth of 100 feet or more was not part of our present scope of services.) If, however, a quantitative assessment of the site seismic properties is desired, then shear wave velocity testing should be performed. Shear wave velocity testing was not part of our present scope of services, but a proposal for this service can be provided upon request. We consider the likelihood of achieving a Site Class B to be moderate.

Using longitude and latitude coordinates obtained from Google Earth and the Applied Technology Council's Hazard by Location tool (https://asce7hazardtool.online/), the tank site is indicated to possess an S_{DS} value of **0.177** and an S_{D1} value of **0.058** for the site latitude and longitude and a Site Class of C.

GEOTECHNICAL CONSIDERATIONS FOR DESIGN

The conclusions and parameters provided in this report were based on the data presented herein, our experience in the general project area with similar structures, and our engineering judgment with regard to the applicability of the data and methods of forecasting future performance. A variety of engineering parameters were considered as indicators of potential future soil movements.

Our parameters and conclusions were based on our judgment of "likely movement potentials," (i.e., the amount of movement likely to be realized if site drainage is generally effective, estimated to a reasonable degree of engineering certainty) as well as our assumptions about the owner's willingness to accept geotechnical risk. "Maximum possible" movement estimates necessarily will be larger than those presented herein. They also have a significantly lower likelihood of being realized in our opinion, and generally require more expensive measures to address.

We encourage Black & Veatch, upon receipt of this report, to discuss these risks and the geotechnical alternatives with us. In addition to the risks and remedial approaches presented in this report, Black & Veatch and the facility owner, also must understand the risk-cost trade-offs addressed by the civil and structural engineering disciplines in order to direct the design team to the portion of the Higher Cost/Lower Risk–Lower Cost/Higher Risk spectrum in which this project should be designed. If Black & Veatch or the owner does not understand these risks, it is critical that additional information or clarification be requested so that the owner's expectations reasonably can be met.

Depth of Wetting at the Site The "depth of wetting" (the depth to which foundation soils will gain moisture and experience volume change over the design-life of a structure) estimated for a given site strongly affects the anticipated performance of structures at that site. Based on the data obtained at this site and our experience with similar geotechnical settings, a depth of wetting of 20 feet below the buried tank bottom was used to develop geotechnical parameters for the foundation systems. Depths of wetting of 20 feet is equal to or greater than the depth of wetting found at about 92 percent and 72 percent, respectively, of the sites evaluated in a study by Walsh and others (2009).[5]

"Depths of wetting" of 30, 40, or 70 feet or more have been considered (e.g., Chao and others, 2006)[6] and have been encountered locally in the field. Depths of wetting of such magnitudes, however, generally are in unusual geologic conditions, such as the steeply dipping bedrock along the eastern margin of the Rampart Range to the west, or identified forensically in unusual circumstances such as a pipe leak that has remained unrepaired for an extended period. In our experience, such deep depths of wetting are considered only rarely in engineering consulting practice in more typical geologic settings in the Colorado Front Range area.

GROUND considers the above "depths of wetting" to be appropriately conservative for the proposed project. However, if Black & Veatch or the facility owner prefers that a more conservative (or less conservative) depth be used to develop geotechnical parameters for design, GROUND should be contacted to revise the criteria provided herein.

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[5] Walsh, K.D., C.A. Colby, W.N. Houston and S.A. Houston, 2009, *Method for Evaluation of Depth of Wetting in Residential Areas*, Journal of Geotechnical and Geoenvironmental Engineering, American Society of Civil Engineers, Vol. 135, No. 2, pp. 169 – 176.

[6] Chao, K-C, D.D. Overton, and J.D. Miller, 2006, *The Effects of Site Conditions on the Predicted Time Rate of Heave*, Unsaturated Soils 2006, American Society of Civil Engineers, Special Publication No. 147, pp. 2086 – 2097.

General Geotechnical Risk In GROUND's opinion the primary source of geotechnical risk at the site is the presence of expansive claystone bedrock. Swells ranging up to approximately 7.0 percent were measured in samples of the claystone. We consider swells of these magnitudes to be representative of the expansive materials at the project site, where present, and could affect nearly all improvements at the site. Such expansive claystone has caused significant and damaging post-construction movements in greater project area.

However, these expansive claystones were interbedded on a relatively fine scale with sandstone beds that comprised half or more of the shallow bedrock section underlying the site. Therefore, in GROUND's opinion the risk of expansive bedrock heave is lower at the subject site than at other locations where claystones constitute a higher portion of the bedrock. Regardless, we some movements from expansive bedrock heave likely will result.

Another source of geotechnical risk at the site is the present of undocumented fill soils, which are considered to be geotechnically unsuitable to support new construction due to their unknown compositions and consistencies. Undocumented fills were recognized in the test holes to depths of about 2 feet below existing grade, but greater depths could be present locally, and additional, undocumented fills could be generated by demolition of the existing tank and its foundations. (In our experience, fills generated by demolition of existing facilities seldom are compacted effectively.)

Where fill is present beneath a proposed improvement, undocumented fill soils will need to be removed and replaced as properly compacted fill or the effects of undocumented fill soils otherwise mitigated.

Performance of the Existing Tank Based on correspondence with Black & Veatch, the existing tank was founded on shallow foundations and has performed in a relatively satisfactory manner for more than 50 years. Post-construction tank movements have been typical/acceptable post-construction movements and the related distress and related maintenance has been within project expectations. Based on this site history, we anticipate that a shallow foundation and floor system will provide approximately equal performance, provided that the site is appropriate maintained (i.e., if effective drainage is maintained, and leaks and other distress are addressed in a timely manner, etc.)

Likely Post-Construction Movement Estimates Based on the data developed for this study and our experience in the project area, we estimate that post-construction movements on the order of 1½ inches for the pump station are likely, with differential movements of similar magnitude, where the proposed tank or pump station is to be supported directly on the existing bedrock at the site. Somewhat higher movements, e.g., 2 inches, are likely for elements supported on the existing soils. Movements of these magnitudes result in distress to structures experiencing them.

General Foundation and Floor Types

Deep Foundations and Structural Floors At this site, supporting the proposed tank and/or pump station on deep foundations, such as drilled piers, bearing in underlying bedrock with lengths likely on the order of 25 feet, will provide the least risk of post-construction movements—estimated to be approximately ½ inch or less. Similarly, utilizing a structural tank bottom floor system, also supported on deep foundations, will provide the least risk of floor movements. We anticipate that use of drilled pier foundations may not be cost effective for this project, but GROUND can provide parameters for a drilled pier foundation upon request.

Shallow Foundations and Slab-on-Grade Floors As higher risk, but commonly used alternative, shallow foundations footings—a ring footing or a mat foundation for the tank, a spread footing foundation for the pump station—could be used. We estimate that shallow foundations bearing directly on firm, undisturbed bedrock are subject to likely, post-construction movements on the order of 1½ inches, with differential movements of about 1 inch over spans of approximately 40 feet.

The tank bottom and a slab-on-grade floor for the pump station should bear on 2½-foot thick section of properly compacted, CDOT Class 6 Aggregate Base Course for similar estimates of likely, post-construction movements.

Additional geotechnical parameters for shallow foundations and slab-on-grade floors are provided below.

FOUNDATION SYSTEMS

The foundation parameters and considerations provided below were developed based on the performance expectations, geotechnical risks, and site conditions discussed in the prior sections of this report. The foundation systems used should be based on the owner's tolerance of post-construction movements and the associated cost-risk trade-offs. The use of these parameters assumes that the above discussed, system-associated risks and post-construction movement estimates are acceptable for the project.

Shallow Foundations

Geotechnical Parameters for Shallow Foundation Design

1) Shallow foundations should bear on firm, undisturbed bedrock.

   If a portion of a foundation excavation exposes fill or native overburden soils (sands), or loose, densely fractured rock, then those materials should be excavated down to firm bedrock and the deepened portion of the excavation backfilled with concrete.

2) A shallow foundation bearing on firm, undisturbed bedrock may be designed for an allowable soil bearing pressure of **4,500 psf** for footings up to 10 feet in width (least lateral dimension) or for a 175-foot diameter mat slab.

   These values may be increased by ⅓ for transient loads such as wind or seismic loading. For larger foundations, a lower allowable bearing pressure may be appropriate.

   Immediate compression of the bearing soils as the footings are loaded to the provided allowable bearing pressure is estimated to be about ½ inch, based on an assumption of drained foundation conditions. This estimate of foundation movement from immediate compression of the foundation soils is a component of the total, likely, post-construction movement estimated for the structures at this site.

To reduce differential settlements between footings or along continuous footings, footing loads should be as uniform as possible. Differentially loaded footings will settle differentially.

If foundation soils are subjected to an increase/fluctuation in moisture content, however, the effective bearing capacity will be reduced and greater post-construction movements than those estimated above may result.

3) An allowable vertical modulus of subgrade reaction (**K**v) of **175 pci** may be used for design of a mat foundation bearing on firm undisturbed bedrock. This value is for a 1-foot by 1-foot plate; it should be adjusted for slab dimension.

4) Geotechnical parameters for resistance to lateral loads are provided in the *Lateral Loads* section of this report.

5) Spread footings should have a minimum lateral dimension of **16 or more inches** for linear strip/ring footings and **24 or more inches** for isolated pad footings. Actual footing dimensions should be determined by the structural engineer.

6) Footings should bear at an elevation **3 or more feet** below the lowest adjacent exterior finish grades to have adequate soil cover for frost protection.

7) Continuous foundation walls should be reinforced as designed by a structural engineer to span an unsupported length of at least **10 feet**.

8) Geotechnical parameters for lateral resistance to foundation loads are provided in the *Lateral Loads* section of this report.

9) Connections of all types must be flexible and/or adjustable to accommodate the anticipated, post-construction movements of the structures.

10) To the extent possible, utility lines should not be routed under shallow foundations, particularly isolated pad foundations. Where doing so cannot be avoided, there is increased risk to both the pipe and the foundation. Measures should be included in design to protect both the footings from increased settlement (such as backfilling the utility trench with Controlled Low Strength Material" (CLSM), i.e., a lean, sand-

cement slurry ("flowable fill") or a similar material) and to protect the pipe from deformation.

Where utility lines penetrate footings or stem walls, etc., measures should be included to accommodate the likely total and differential, post-construction movements discussed in this report. Some footings also may experience lateral displacements as structural loads are applied.

Shallow Foundation Construction

11) The contractor should take adequate care when making excavations not to compromise the bearing or lateral support for nearby improvements.

12) Care should be taken when excavating the foundations to avoid disturbing the supporting materials particularly in excavating the last few inches.

13) Footing excavation bottoms may expose loose, organic, or otherwise deleterious materials, including debris. Firm materials may become disturbed by the excavation process. All such unsuitable materials should be excavated and replaced with concrete or the foundation deepened.

14) Foundation-supporting materials may be disturbed or deform excessively under the wheel loads of heavy construction vehicles as the excavations approach footing bearing levels. Construction equipment should be as light as possible to limit development of this condition. The movement of vehicles over proposed foundation areas should be restricted.

15) The contractor should take adequate care to keep excavated surfaces free of standing water.

16) Fill placed against the sides of the footings should be properly compacted in accordance with the *Project Earthwork* section of this report.

ON-GRADE TANK BOTTOM/FLOOR

The geotechnical parameters below may be used for design of slab-on-grade floors for the proposed tank and pump station. ACI Sections 301/302/360 provide guidance regarding concrete slab-on-grade design and construction.

Slab-on-Grade Floors

Geotechnical Parameters for Design of Concrete Tank Bottom and Pump Station Slab-on-Grade Floor

1) An on-grade tank bottom or pump station floor should bear on 2½ feet of properly compacted, CDOT Class 6 Aggregate Base Course placed on firm, undisturbed bedrock. Parameters and considerations for fill placement and compaction are provided in the *Project Earthwork* section of this report. The aggregate base course section beneath a tank bottom or floor slab should be of uniform thickness.

   However, if a tank bottom or floor excavation exposes firm, undisturbed bedrock—which is more likely at the pump station than at the tank location—then that tank bottom or floor system may bear directly on the bedrock. If a portion of a tank bottom or floor excavation exposes fill or native overburden soils (sands), or loose, densely fractured rock, then those materials should be excavated down to firm bedrock and the deepened portion of the excavation backfilled with concrete. Additionally, where an irregular bearing surface is excavated in the bedrock or where it otherwise appears beneficial, a mud mat/mud slab (lean concrete leveling layer) should be used to construct a firm, level surface beneath the tank bottom and/or the floor slab.

2) A concrete tank bottom and slab-on-grade floor should be adequately reinforced. Floor slab design, including slab thickness, concrete strength, jointing, and slab reinforcement should be developed by a structural engineer.

3) An allowable vertical modulus of subgrade reaction (**Kv**) of **130 pci** may be used for design of a concrete, tank bottom or slab-on-grade floor bearing on properly compacted aggregate base course, or a **Kv** of **175 pci** if it were bearing on firm, undisturbed bedrock.

These values are for a 1-foot by 1-foot plate; they should be adjusted for slab dimension.

4) A floor slab in the pump station should be separated from all bearing walls and columns with slip joints, which allow unrestrained vertical movement.

   Slip 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 to assure that slab isolation is maintained in order to reduce the likelihood of damage to walls and other interior improvements.

5) A floor slab in the pump station should be provided with properly designed control joints.

   ACI, AASHTO, and other industry groups provide guidelines for proper design and construction concrete slabs-on-grade and associated jointing. The design and construction of such joints should account for cracking as a result of shrinkage, curling, tension, loading, and curing, as well as proposed slab use. Joint layout based on the slab design may require more frequent, additional, or deeper joints, and should reflect the configuration and proposed use of the slab.

   Particular attention in slab joint layout should be paid to areas where slabs consist of interior corners or curves (e.g., at column blockouts or reentrant corners) or where slabs have high length to width ratios, significant slopes, thickness transitions, high traffic loads, or other unique features. Improper placement or construction will increase the potential for slab cracking.

6) Interior partitions resting on the pump station floor slab should be provided with slip joints so that if the slabs move, the movement cannot be transmitted to the upper structure. This detail is also important for wallboards and doorframes. Slip joints should allow **2 inches or more** of vertical, differential movement. Accommodation for differential movement also should be made where partitions meet bearing walls.

7) Post-construction movements may not displace the slab-on-grade floor and project piping in the underlying soils to the same extent. Design of floor penetrations,

connections, and fixtures should accommodate **up to 2 inches** of differential movement.

8) Moisture can be introduced into a slab subgrade during construction and additional moisture will be released from the slab concrete as it cures. A properly compacted layer of free-draining gravel, **4 or more inches** in thickness, should be placed beneath the slabs. This layer will help distribute floor slab loadings, ease construction, reduce capillary moisture rise, and aid in drainage. Selection and specification of subslab gravel should be coordinated with soil gas mitigation systems, where such systems are used.

   The free-draining gravel should contain **less than 5 percent** material passing the No. 200 Sieve, **less than 50 percent** passing on the No. 4 Sieve, and a maximum particle size of **2 inches**.

   It has been our experience that the use of CDOT 6 Aggregate Base Course has been employed as a base layer for concrete slabs on grade. While CDOT Class 6 Aggregate Base Course is not an equivalent to the free-draining gravel, discussed above, and does not provide an equivalent degree of capillary break, it is often used to address ACI’s recommendation for a coarse-grained, granular base material that is easily trimmable and has relatively little fines (material finer than the No. 200 sieve). The Class 6 section beneath the tank bottom or pump station floor can serve as this base layer or thickened for that purpose, as necessary.

   The capillary break and the drainage space provided by the gravel layer also may reduce the potential for excessive water vapor fluxes from the slab after construction as mix water is released from the concrete.

9) A vapor barrier beneath the pump station floor slab can be beneficial with regard to reducing exterior moisture moving into the building, through the slab, but can retard downward drainage of construction moisture. Uneven moisture release can result in slab curling. Elevated vapor fluxes can be detrimental to the adhesion and performance of many floor coverings and may exceed various flooring manufacturers’ usage criteria.

ACI 302.2R-15, however, recommends placement of a vapor barrier under concrete slabs-on-ground when they will receive (or could receive in the future) moisture-sensitive floor coverings, coatings, adhesives, underlayments, and/or stored goods. ACI 302 provides further guidance on the location of the vapor barrier beneath the slab.

When a vapor barrier is used, it should consist of a minimum 15-mil thickness, extruded polyolefin plastic (no recycled content or woven materials), maintain a permeance less than 0.01 perms per ASTM E96 or ASTM E1249 before and after mandatory conditioning testing, and comply with ASTM E1745-17 (Class “A”). Vapor barriers should be installed in accordance with ASTM E1643-18 and the manufacturer’s guidelines.

Polyethylene (“poly”) sheeting (even if 15 mils in thickness which polyethylene sheeting commonly is not) does not meet the ASTM E-1745 criteria and should not be used as vapor barrier material. It can be easily torn and/or punctured, does not possess necessary tensile strength, gets brittle, tends to decompose over time, and has a relatively high permeance.

Construction Considerations for Slab-on-Grade Floors

10) The contractor should take adequate care to keep excavated surfaces free of standing water.

11) Loose, CDOT Class 6 Aggregate Base Course exposed on the prepared surface on which a tank bottom or floor slab will be cast should be scarified and recompacted properly.

12) A concrete tank bottom and pump station floor slab should be constructed and cured in accordance with applicable industry standards and slab design specifications.

13) All piping should be carefully tested before operation. Where plumbing lines enter through the floor, a positive bond break should be provided.

PUMP STATION BELOW-GRADE WALLS

The following parameters and considerations should be used for below-grade foundation walls for the pump station building.

Wall Design Parameters Equivalent fluid pressures for use in design of foundation walls are provided in the *Lateral Loads* section of this report.

To realize the lower equivalent fluid unit weight provided for CDOT Class 1 Structure Backfill, that material should be placed behind the wall to a minimum distance equal or greater than **half of the wall height**. If CDOT Class 1 Structure Backfill were used, then the upper **1 foot** of the wall backfill should be a relatively impermeable soil or otherwise protected to reduce surface water infiltration into the backfill.

The at-rest and active unit weights in the *Lateral Loads* section of this report are for well drained conditions with a horizontal upper backfill surface. The additional loading of an upward sloping backfills, hydrostatic loads if sufficient drainage is not provided, as well as loads from traffic, stockpiled materials, etc., should be included in retaining wall design.

Below-Grade Wall Drainage As outlined in the *Subsurface Drainage* section of this report, below-grade walls should be provided with an underdrain at or below the base of the wall or foundation. Drainage elements backing the wall should convey water into the underdrain system.

If the wall were designed for undrained conditions, then including a drain may not be necessary.

Below-Grade Wall Construction Considerations Wall backfill soils should be compacted properly, but the contractor should take care regarding compaction methods and efforts so that excessive lateral pressures on the walls do not result.

Some settlement of wall backfill will occur even where the material was placed and compacted correctly. This settlement likely will be differential, increasing with depth of fill. Regrading to reestablish effective surface drainage away from the structure should be anticipated.

Where shallowly founded structures or pavements are be placed on backfilled zones, the associated risks should be understood by the owner. Structural design, pipe connections, etc., should take into account (differential) foundation wall backfill settlements. A geotechnical engineer should be retained to provide design parameters where improvements are placed in backfilled areas.

LATERAL LOADS

Shallow Foundations Resisting Lateral Loads Values for equivalent fluid pressures and the coefficient for frictional resistance to sliding are provided below. These values provided below were based on a moist unit weight (γ') of 127 pcf and an angle of internal friction (ϕ) of 26 degrees for the site soils and bedrock reworked as properly compacted fill, and γ' of 135 pcf and ϕ of 34 degrees for select granular fill. These values are unfactored. Appropriate factors of safety should be included in design calculations.

EQUIVALENT FLUID WEIGHTS (DRAINED CONDITION)

Backfill Material	**Condition**			**Friction Coefficient**
	Active	**At-Rest**	**Passive**	
Site Fill, Native Soils, and Bedrock Reworked as Properly Compacted Fill	**50 pcf**	**72 pcf**	**285 pcf** (to a maximum of 2,850 psf)	**0.33**
Select Granular Fill (CDOT Class Structure Backfill)	**39 pcf**	**60 pcf**	**440 pcf** (to a maximum of 4,400 psf)	**0.45**

EQUIVALENT FLUID WEIGHTS (SUBMERGED CONDITION)

Backfill Material	Condition			Friction Coefficient
	Active	**At-Rest**	**Passive**	
Site Fill, Native Soils, and Bedrock Reworked as Properly Compacted Fill	**88 pcf**	**99 pcf**	**285 pcf** (to a maximum of 2,850 psf)	**0.33**
Select Granular Fill (CDOT Class Structure Backfill)	**84 pcf**	**95 pcf**	**280 pcf** (to a maximum of 2,800 psf)	**0.45**

Where the full passive soil pressure is used to resist lateral loads, it should be understood that significant lateral strains will be required to mobilize the full value indicated above, likely 1 inch or more.

WATER-SOLUBLE SULFATES

The concentration of water-soluble sulfates measured in selected samples of site soils was about 0.01 percent by weight. (See Table 2.) Such a concentration of soluble sulfates represents a **negligible** environment for sulfate attack on concrete exposed to these materials. Degrees of attack are based on the scale of “negligible,” “moderate,” “severe,” and “very severe” as described in the “Design and Control of Concrete Mixtures,” published by the Portland Cement Association (PCA). The Colorado Department of Transportation (CDOT) utilizes a corresponding scale with four classes of severity of sulfate exposure (Class 0 to Class 3) as described in the table below.

REQUIREMENTS TO PROTECT AGAINST DAMAGE TO
CONCRETE BY SULFATE ATTACK FROM EXTERNAL SOURCES OF SULFATE

Severity of Sulfate Exposure	Water-Soluble Sulfate ($SO_4^=$) In Dry Soil *(%)*	Sulfate (SO_4) In Water *(ppm)*	Water Cementitious Ratio *(maximum)*	Cementitious Material Requirements
Class 0	0.00 to 0.10	0 to 150	0.45	Class 0
Class 1	0.11 to 0.20	151 to 1500	0.45	Class 1
Class 2	0.21 to 2.00	1501 to 10,000	0.45	Class 2
Class 3	2.01 or greater	10,001 or greater	0.40	Class 3

Based on our test results and PCA and CDOT guidelines, sulfate resistant cement conforming to one of the following Class 0 requirements in all concrete exposed to site soils and bedrock:

Class 0 (Negligible)

1) ASTM C150 Type I, II, III, or V.
2) ASTM C595 Type IL, IP, IP(MS), IP(HS), or IT.

SOIL CORROSIVITY

Data were obtained to support an initial assessment of the potential for corrosion of ferrous metals in contact with earth materials at the site, based on the conditions at the time of GROUND's evaluation. The test results are summarized in Table 2.

Reduction-Oxidation testing indicated red-ox potentials as low as approximately -74 millivolts. Such low potentials typically create a more corrosive environment.

Sulfide Reactivity testing included a "positive" result in the site soils. The presence of sulfides in the soils suggests a more corrosive environment.

Chlorides testing indicated a chloride concentration as high as 17 parts per million.

Soil Resistivity In order to assess the "worst case" for mitigation planning, samples of materials retrieved from the borings and prior test holes were tested for resistivity in the laboratory, after being saturated with water, rather than in the field. Resistivity also varies inversely with temperature. Therefore, the laboratory measurements were made at a controlled temperature. Measurements of electrical resistivity indicated values between approximately 5,521 and 10,558 ohm-centimeters in selected samples of site soils.

pH Where pH is less than 4.0, soil serves as an electrolyte; the pH range of about 6.5 to 7.5 indicates soil conditions that are optimum for sulfate reduction. In the pH range above 8.5, soils are generally high in dissolved salts, yielding a low soil resistivity.[7] Testing indicated pH values of about 8.4 and 8.6.

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[7] American Water Works Association ANSI/AWWA C105/A21.5-05 Standard.

Corrosivity Assessment The American Water Works Association (AWWA) has developed a point system scale used to predict corrosivity. The scale is intended for protection of ductile iron pipe but is valuable for project steel selection. When the scale equals 10 points or higher, protective measures for ductile iron pipe are indicated. The AWWA scale is presented below.

Table A.1 Soil-Test Evaluation

Soil Characteristic / Value	Points
Redox Potential	
< 0 (negative values)	5
0 to +50 mV	4
+50 to +100 mV	3½
> +100 mV	0
Sulfide Reactivity	
Positive	3½
Trace	2
Negative	0
Soil Resistivity	
<1,500 ohm-cm	10
1,500 to 1,800 ohm-cm	8
1,800 to 2,100 ohm-cm	5
2,100 to 2,500 ohm-cm	2
2,500 to 3,000 ohm-cm	1
>3,000 ohm-cm	0
pH	
0 to 2.0	5
2.0 to 4.0	3
4.0 to 6.5	0
6.5 to 7.5	0 *
7.5 to 8.5	0
>8.5	3
Moisture	
Poor drainage, continuously wet	2
Fair drainage, generally moist	1
Good drainage, generally dry	0

* If sulfides are present <u>and</u> low or negative redox-potential results (< 50 mV) are obtained, add three (3) points for this range.

The soil characteristics refer to the conditions at and above pipe installation depth. We anticipate that drainage at the site after construction will be effective. Nevertheless, based on the values obtained for the soil parameters, the fill and native soils appear to comprise a severely corrosive environment for ferrous metals (11½ points).

If additional information or evaluation is needed regarding soil corrosivity, then the American Water Works Association or a corrosion engineer should be contacted. It should be noted, however, that changes to the site conditions during construction, such as the import of other soils, or the intended or unintended introduction of off-site water, might alter corrosion potentials significantly.

PROJECT EARTHWORK

The earthwork criteria below are based on our interpretation of the geotechnical conditions encountered in the test holes. Where these criteria differ from applicable municipal specifications, e.g., for trench backfill compaction along a public utility line, the latter should be considered to take precedence.

General Considerations Project grading should be performed as early as possible in the construction sequence to allow settlement of fills and surcharged ground to be realized to the greatest extent prior to subsequent construction.

Prior to earthwork construction, existing construction debris, vegetation, and other deleterious materials should be removed and disposed of off-site. Relic underground utilities should be abandoned in accordance with applicable regulations, removed as necessary, and properly capped.

Topsoil, including roots and organic landscaping materials, present on-site should not be incorporated into ordinary fills. Instead, topsoil should be stockpiled during initial grading operations for placement in areas to be landscaped or for other approved uses.

Use of Existing Fill Soils Fill materials were recognized in the test holes during subsurface exploration and are likely are present elsewhere on the site, particularly after demolition of the existing tank. We anticipate that the majority of the existing fill soils will be suitable for reuse as fill. However, because not all the fill soils were sampled and tested, it is possible that some of the fill soils may not be suitable for reuse as compacted fill, due to the presence of deleterious materials such as trash, organic material, coarse cobbles and boulders, or construction debris. Therefore, excavated fill materials should be evaluated and tested, as appropriate, with regard to reuse.

Use of Existing Native Soils Based on the samples retrieved from the test holes, we anticipate that the existing site soils free of organic materials, coarse cobbles, boulders, or other deleterious materials will be suitable, in general, for reuse as compacted fill for general fills. Fragments of rock, cobbles, sandstone fragments (as well as inert construction debris, e.g., concrete or asphalt) up to **3 inches** in maximum dimension may be included in project fills, in general. Such materials should be evaluated on a case-by-case basis, where identified during earthwork.

Imported Fill Materials Materials imported to the site as (common) fill should be free of organic material, and other deleterious materials. Imported material should exhibit **60 percent or less** passing the No. 200 Sieve and a plasticity index of **15 or less**. Materials proposed for import should be approved prior to transport to the site.

Select, Granular Fill Material to be imported to the site to be used as select, granular, foundation wall backfill should meet the criteria for CDOT Class 1 Structure Backfill.

Material imported to the site to be used beneath a tank bottom or pump station floor slab should meet the criterial for CDOT Class 6 Aggregate Base Course. Note that material meeting the criteria for Class 6 Aggregate Base Course commonly will meet the criteria for Class 1 Structure Backfill.

GRADATION CRITERIA FOR
CDOT CLASS 1 STRUCTURE BACKFILL &
CDOT CLASS 6 AGGREGATE BASE COURSE

Sieve Size or Parameter	Class 1 Structure Backfill	Class 6 Aggregate Base Course
2-inch	100% passing	100% passing
1-inch		100% passing
¾-inch		95 to 100% passing
No. 4	30% to 100% passing	30% to 65% passing
No. 50	10% to 60% passing	
No. 200	5% to 20% passing	3% to 12% passing
Liquid Limit	$\leq$ 35	$\leq$ 30
Plasticity Index	$\leq$ 6	$\leq$ 6

Materials proposed for select, granular fill should be tested and approved for that use prior to import to the site.

Fill Platform Preparation Prior to filling, the top **12 inches** of in-place materials on which fill soils will be placed (except for sound bedrock or utility trench bottoms where bedding will be placed) should be scarified, moisture conditioned and properly compacted in accordance with the criteria below to provide a uniform base for fill placement.

If surfaces to receive fill expose loose, wet, soft, or otherwise deleterious material, additional material should be excavated, or other measures taken to establish a firm platform for filling. A surface to receive fill must be effectively stable prior to placement of fill, including trench bottoms prior to placement of bedding.

General Considerations for Fill Placement Fill soils should be thoroughly mixed to achieve a uniform moisture content, placed in uniform lifts not exceeding 8 inches in loose thickness, and properly compacted.

The bedrock beneath the site consisted of highly interbedded sandstone and claystone. Excavated bedrock to be placed as fill should be mixed and processed to create a relatively uniform soil mixture. Variable fill compositions will result in higher post-construction movements. The excavated bedrock materials will require a well-coordinated effort to moisture treat, process, place, and compact properly. The excavated bedrock should be broken down in to a soil-like mass. Greater than typical watering and mixing, and compaction equipment that aids in breaking down such material likely will be needed. Crushing or other methods should be anticipated to reduce excavated bedrock sufficiently. Applied water will be taken up into the structures of the claystone. The contractor should anticipate that handling and processing the excavated bedrock more than once may be necessary to achieve the requirements herein.

Excavated bedrock, include those present in the existing fill, to be used as trench backfill, will require additional moisture conditioning and processing in an open area outside of trenches prior to placement as backfill.

No fill materials should be placed, worked, rolled while they are frozen, thawing, or during poor/inclement weather conditions.

Where soils on which foundation elements will be placed are exposed to freezing temperatures or repeated freeze–thaw cycling during construction—commonly due to water ponding in foundation excavations—bearing capacity typically is reduced and/or settlements increased due to the loss of density in the supporting soils. After periods of freezing conditions, the contractor should rework areas affected by the formation of ice to reestablish adequate bearing support.

Care should be taken with regard to achieving and maintaining proper moisture contents during placement and compaction. Materials that are not properly moisture conditioned may exhibit significant pumping, rutting, and deflection at moisture contents near optimum and above. The contractor should be prepared to handle soils of this type, including the use of chemical stabilization, if necessary.

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

Compaction Criteria CDOT Class 6 Aggregate Base Course placed beneath the tank bottom or the pump station floor slab should be compacted to **96 or more percent** of the maximum dry density at moisture contents **within 3 percent** of the optimum moisture content as determined by ASTM D1557, the modified Proctor.

Soils that classify as **GP, GW, GM, GC, SP, SW, SM, or SC** in accordance with the USCS classification system (granular materials) to be used as common fill, and CDOT Class 1 Structure Backfill placed behind pump station foundation walls should be compacted to **95 or more percent** of the maximum dry density at moisture contents **within 2 percent** of the optimum moisture content as determined by ASTM D1557, the modified Proctor.

Soils that classify as **ML, MH, CL, or CH** should be compacted to **at least 95 percent** of the maximum dry density at moisture contents between **1 percent below and 3 percent above** the optimum moisture content as determined by ASTM D698, the standard Proctor.

Use of Squeegee Relatively uniformly graded fine gravel or coarse sand, i.e., “squeegee,” or similar materials commonly are proposed for backfilling foundation excavations, utility trenches (excluding approved pipe bedding), and other areas where employing compaction equipment is difficult. In general, this procedure should not be followed for the following reasons.

Although commonly considered “self-compacting,” uniformly graded granular materials require densification after placement, typically by vibration. The equipment to densify these materials is not available on many jobsites.

Even when properly densified, uniformly graded granular materials are permeable and allow water to reach and collect in the lower portions of the excavations backfilled with those materials. This leads to wetting of the underlying soils and resultant potential loss of bearing support as well as increased local heave or settlement.

Wherever possible, excavations should be backfilled with approved, on-site soils placed as properly compacted fill. Where achieving adequate compaction is difficult, then Controlled Low Strength Material” (CLSM), i.e., a lean, sand-cement slurry (“flowable fill”) or a similar material should be used for backfilling.

Where “squeegee” or similar materials are proposed for use by the Contractor, the design team should be notified by means of a Request for Information (RFI), so that the proposed use can be considered on a case-by-case basis. Where “squeegee” meets the project requirements for pipe bedding material, however, it is acceptable for that use.

Settlements Settlements will occur in newly filled ground, typically on the order of 1 to 2 percent of the fill depth. This is separate from settlement of the existing soils left in place. If fill placement is performed properly and is tightly controlled, in GROUND’s experience the majority (on the order of 60 to 80 percent) of that settlement typically will take place during earthwork construction, provided the contractor achieves the compaction levels indicated herein. For a 12-foot fill, for example, that corresponds to a total settlement of about 2 inches about 1 inch of which will occur during construction. The remaining potential settlements likely will take several months or longer to be realized, and may be exacerbated if these fills are subjected to changes in moisture content. At this site, settlement monitoring programs during construction may be beneficial.

Cut and Filled Slopes Permanent (final) graded slopes supported by local soils up to **15 feet** in height should be constructed no steeper than **3 : 1** (horizontal : vertical) without slope-specific stability analyses. (The existing slopes at steeper angles exhibited relatively low factors of safety.) Minor raveling or surficial sloughing should be anticipated on slopes cut at this angle until vegetation is well reestablished. Surface drainage should be designed to direct water away from slope faces into designed drainage pathways or structures.

Steeper slope angles and heights may be possible but will require detailed slope stability analysis based on final proposed grading plans. A geotechnical engineer should be retained to evaluate this on a case-by-case basis.

EXCAVATION CONSIDERATIONS

Excavation Difficulty Test holes for the subsurface exploration were advanced to the depths indicated on the test hole logs by means of conventional, truck-mounted, geotechnical drilling equipment advancing continuous flight auger and wireline coring equipment. The bedrock at the site was variable in hardness, including some very hard layers, e.g., that exhibited penetration resistance values of 50 blows for 2 inches to 50 blows for “0.” Practical auger refusal was encountered in some of the test holes. The contractor should be prepared to excavate, handle, process, and export such materials discussed above, where excavations extend into bedrock materials. Specialized rock-breaking equipment and/or localized rock blasting may be necessary. If allowed, limited, light blasting could be cost-effective to facilitate bedrock excavations.

Undocumented fill soils were encountered in the test holes and can present excavation difficulties. Given the inherent nature of undocumented fill soils, including demolition fills, materials that may be awkward or otherwise difficult to handle (e.g., relatively large pieces of construction debris) may be encountered the undocumented fill soils. The contractor and the project team should be prepared to handle such materials.

Temporary Excavations and Personnel Safety Excavations in which personnel will be working must comply with all applicable OSHA Standards and Regulations, particularly CFR 29 Part 1926, OSHA Standards–Excavations, adopted March 5, 1990. The contractor’s “responsible person” should evaluate the soil exposed in the excavations as part of the contractor’s safety procedures. GROUND has provided the information in this report solely as a service to Black & Veatch, and is not assuming responsibility for construction site safety or the contractor's activities.

The contractor should take care when making excavations not to compromise the bearing or lateral support for any adjacent, existing improvements.

Should site constraints prohibit the use of sloped excavations, temporary shoring should be used. GROUND is available to provide shoring design upon request.

Groundwater Groundwater was not encountered in the test holes at the time of drilling to the depths explored. The test holes were backfilled upon drilling completion per Code of Colorado Regulations (2 CCR 402-2). Groundwater levels can be expected to fluctuate, however, in response to annual and longer-term cycles of precipitation, irrigation, surface drainage, nearby rivers and creeks, land use, and the development of transient, perched water conditions. The groundwater observations performed during our exploration must be interpreted carefully as they are short-term and do not comprise a groundwater study. In the event that Black & Veatch desires additional/repeated groundwater level observations, GROUND should be contacted; additional exploration and fees will be necessary in this regard.

It is possible that groundwater may be encountered in project excavations. The contractor should be prepared to dewater the excavation during construction. Pumps adequate to discharge water and/or well points to draw down the water level may be appropriate methods. Other methods may also be necessary. The dewatering approach should ultimately be determined by the contractor based on their means and methods experience. Dewatering operations may be necessary as both temporary and long-term/permanent installations. If seepage or groundwater is encountered during excavation or at any time during construction, a geotechnical engineer and project team should be contacted to evaluate the conditions. The presence of groundwater in these types of situations and associated potential design changes can have an impact to both the financial and schedule components of a project.

Surface Water The contractor should take proactive measures to control surface waters during construction and maintain good surface drainage conditions to direct waters away from excavations and into appropriate drainage structures. A properly designed drainage swale should be provided at the tops of the excavation slopes. In no case should water be allowed to pond near project excavations.

Temporary slopes should also be protected against erosion. Erosion along the slopes will result in sloughing and could lead to a slope failure.

DRAIN/FILL PIPING INSTALLATION

The measures and criteria below are based on GROUND's evaluation of the local, geotechnical conditions. Where the parameters herein differ from applicable municipal requirements, the latter should be considered to govern.

Pipe Support The bearing capacity of the site soils appeared adequate, in general, for support of typical utility lines. The pipes and contents are less dense than the soils which will be displaced for installation. Therefore, in general GROUND anticipates no significant pipe settlements in these materials where properly bedded from loading alone.

Trench bottoms may expose existing fill soils, or soft, loose, or otherwise deleterious materials. Firm materials may be disturbed by the excavation process. All such unsuitable materials should be excavated and replaced with properly compacted fill. Where existing fill soils are left in place, locally greater pipe settlements may result, causing "bellies" in the pipes. (In the case of a pressurized water line, such deflections may be of less consequence.)

Areas allowed to pond water will require excavation and replacement with properly compacted fill. The contractor should take particular care to ensure adequate support near pipe joints which are less tolerant of extensional strains.

Where thrust blocks are needed, the parameters provided in the *Lateral Loads* section of this report may be used for design.

Trench Backfilling Some settlement of compacted soil trench backfill materials should be anticipated, even where all the backfill is placed and compacted correctly. Typical settlements are on the order of 1 to 2 percent of fill thickness. However, the need to compact to the lowest portion of the backfill must be balanced against the need to protect the pipe from damage from the compaction process. Some thickness of backfill may need to be placed at compaction levels lower than specified (or smaller compaction equipment used together with thinner lifts) to avoid damaging the pipe. Protecting the pipe in this manner can result in somewhat greater surface settlements. Therefore, although other alternatives may be available, the following options are presented for consideration:

Controlled Low Strength Material Because of these limitations, a conservative approach consists of backfilling the entire depth of the trench (both bedding and common backfill zones) with “controlled low strength material” (CLSM), i.e., a lean, sand-cement slurry, “flowable fill,” or similar material along all trench alignment reaches with low tolerances for surface settlements.

CLSM used as pipe bedding and trench backfill should exhibit a 28-day unconfined compressive strength between **50 to 150 psi** so that reexcavation is not unusually difficult.

Placement of the CLSM in several lifts or other measures likely will be necessary to avoid ‘floating’ the pipe. Measures also should be taken to maintain pipe alignment during CLSM placement.

Compacted Soil Backfilling Where compacted soil backfilling is employed, using the site soils or similar materials as backfill, the risk of backfill settlements entailed in the selection of this higher risk alternative must be anticipated and accepted by the Black & Veatch.

We anticipate that the on-site soils excavated from trenches will be suitable, in general, for use as common trench backfill within the above-described limitations. Backfill soils should be free of vegetation, organic debris and other deleterious materials. Fragments of rock, cobbles, and inert construction debris (e.g., concrete or asphalt) coarser than 3 inches in maximum dimension should not be incorporated into trench backfills.

Soils placed for compaction as trench backfill should be conditioned to a relatively uniform moisture content, placed, and compacted in accordance with the parameters in the *Project Earthwork* section of this report.

Pipe Bedding Pipe bedding materials, placement and compaction should meet the specifications of the pipe manufacturer and applicable municipal standards. Bedding should be brought up uniformly on both sides of the pipe to reduce differential loadings.

As discussed above, the use of CLSM or similar material in lieu of granular bedding and compacted soil backfill should be considered where the tolerance for surface settlement is low. (Placement of CLSM as bedding to at least 12 inches above the pipe can protect the pipe and assist construction of a well-compacted conventional backfill, although possibly at an increased cost relative to the use of conventional bedding.)

If granular bedding is specified, the contractor should not anticipate that significant volumes of on-site soils will be suitable for that use without significant processing. Materials proposed for use as pipe bedding should be tested for suitability prior to use.

With regard to potential migration of fines into granular pipe bedding, design and installation should follow ASTM D2321, Appendix X1.8. If the granular bedding does not meet filter criteria for the enclosing soils, then non-woven filter fabric (e.g., Mirafi® 140N, or the equivalent) should be placed around the bedding to reduce migration of fines into the bedding which can result in severe, local surface settlements. Where this protection is not provided, settlements can develop/continue several months or years after completion of the project. In addition, clay or concrete cutoff walls should be installed to interrupt the granular bedding section to reduce the rates and volumes of water transmitted along the sewer alignment which can contribute to migration of fines.

Other Considerations We anticipate that the potential for local heave of the site soils to result in extensional strains to utility pipes. Therefore, pipes should be provided with restrained joints to reduce the potential for failure at joints. Connections to the building or other structures should be flexible and easily replaced or adjusted. Non-pressurized lines should be evaluated periodically for deformations such as pipe “bellies” that would impair their efficiency, and appropriate repairs made. Maintenance plans should anticipate greater than typical utility line maintenance and replacement because of the site soils susceptible to heave that will remain beneath utility lines.

SURFACE DRAINAGE

The site soils are relatively stable with regard to moisture content–volume relationships at their existing moisture contents. Other than the anticipated, post-placement settlement of fills, post-construction soil movements will result primarily from the introduction of water into the soils underlying the proposed structures, hardscaping, and pavements. Based on the site surface and subsurface conditions encountered in this study, we do not anticipate a rise in the local water table sufficient to approach floor elevations. Therefore, wetting of the soils likely will result from infiltrating surface waters (precipitation, irrigation, etc.), and water flowing along constructed pathways such as bedding in utility pipe trenches.

The following drainage measures should be followed both for during construction and as part of project design. The facility should be observed periodically to evaluate the surface drainage and identify areas where drainage is ineffective. Routine maintenance of site drainage should be undertaken throughout the design life of the proposed facility – routine maintenance may include local fine grading or other measures so that proper drainage may be reestablished. If these measures are not implemented and maintained effectively, the movement estimates provided in this report could be exceeded.

1) Wetting or drying of the under-tank/pump station areas should be avoided during and after construction. Permitting increases/variations in moisture to the adjacent or supporting soils may result in increased total and/or differential movements.

2) Positive surface drainage measures away from the tank and pump station should be provided and maintained to reduce water infiltration into foundation soils. Underdrains should not be relied upon in surface drainage design to collect and discharge surface waters.

   A minimum slope of **12 inches in the first 10 feet** in the areas not covered with pavement or concrete slabs should be established. For areas covered with asphalt pavement or concrete slabs, slopes **should comply with ADA requirements where required**. Increasing slopes to **a minimum of 3 percent in the first 10 feet** in the areas covered with pavement or concrete slabs will reduce, but not eliminate, the potential for moisture infiltration and subsequent volume change of the underling soils.

   In no case should water be allowed to pond near or adjacent to foundation elements, hardscaping, etc.

3) Drainage also should be established <u>and maintained</u> to direct water away from any hardscaping as well as drain/fill pipe trench alignments which are not tolerant of increased post-construction movements.

The ground surface near foundation elements should be able to convey water away readily. Cobbles or other materials that tend to act as baffles and restrict surface flow should not be used to cover the ground surface near the foundations.

Where the ground surface does not convey water away readily, additional post-construction movements and distress should be anticipated.

SUBSURFACE DRAINAGE

As a component of project civil design, properly functioning, subsurface drain systems ("underdrains") can be beneficial for collecting and discharging saturated subsurface waters. Underdrains will not collect water infiltrating under unsaturated (vadose) conditions, or moving via capillarity, however. In addition, if not properly constructed and maintained, underdrains can transfer water into foundation soils, rather than remove it. This will tend to induce settlement of the subsurface soils, and may result in distress. Underdrains can, however, provide an added level of protection against relatively severe post-construction movements by draining saturated conditions near individual structures should they arise, and limiting the volume of wetted soil.

It is GROUND's opinion that it will be beneficial to include a perimeter underdrain system to help limit wetting of the foundation bearing soils. However, we understand that the owner and project team may consider that the reduction of risk provided by a properly constructed and maintained underdrain system does not justify the costs associated with including an underdrain. In such a case, an underdrain system can be excluded. If an underdrain system is excluded, then there will be an increased risk of the likely post-construction movements estimated in this report being exceeded. GROUND considers this risk to be relatively low, but it is not zero. Where an underdrain system is excluded, extra care should be taken to establish and maintain effective surface drainage, identify and repair wet utility leaks in a timely manner, seal open cracks joints, and restore effective surface drainage as necessary to limit the volume of water infiltrating the site.

Geotechnical Parameters for Underdrain Design Where an underdrain system is included in project drainage design, design should incorporate the parameters below. The actual underdrain layout, outlets, and locations should be developed by a civil engineer. We understand that, at this site, gravity discharge cannot be achieved readily. A typical, cross-section detail of underdrains can be provided upon request.

An underdrain system should be tested by the contractor after installation and after placement and compaction of the overlying backfill to verify that the system functions properly.

1) An underdrain system for the tank or pump station should consist of perforated, rigid, PVC collection pipe at least **4 inches** in diameter, non-perforated, rigid, PVC discharge pipe at least **4 inches** in diameter, free-draining gravel, and filter fabric.

2) The free-draining gravel should be naturally occurring (not recycled) material with **5 percent or less** passing the No. 200 Sieve and **50 percent or more** retained on the No. 4 Sieve, and have a maximum particle size of **2 inches**.

3) Each collection pipe should be surrounded on the sides and top (only) with **6 or more inches** of free-draining gravel.

   The gravel surrounding the collection pipe(s) should be wrapped with filter fabric (Mirafi 140N® or the equivalent) to reduce the migration of fines into the drain system.

4) The underdrain system should be designed to discharge **20 gallons per minute or more** of collected water.

5) The high point(s) for the collection pipe flow lines should be below the grade beam or shallow foundation bearing elevation as shown on the detail. Multiple high points can be beneficial to reducing the depths to which the system would be installed.

   The collection and discharge pipe for the underdrain system should be laid on a slope as determined by the underdrain designer.

Underdrain “clean-outs” should be provided at intervals of no more than **150 feet** to facilitate maintenance of the underdrains. Clean-outs also should be provided at collection and discharge pipe elbows of **60 degrees or more**.

6) The underdrain discharge pipes should be connected to one or more sumps from which water can be removed by pumping, or to outlet(s) for gravity discharge. We suggest that collected waters be discharged directly into the storm sewer system, if possible.

7) Regular maintenance of the underdrain systems should be performed to ensure that the system continues work properly.

CLOSURE

Geotechnical Review The author of this report or a GROUND principal should be retained to review project plans and specifications to evaluate whether they comply with the intent of the measures discussed in this report. The review should be requested in writing.

The geotechnical conclusions and parameters presented in this report are contingent upon observation and testing of project earthwork by representatives of GROUND. If another geotechnical consultant is selected to provide materials testing, then that consultant must assume all responsibility for the geotechnical aspects of the project by concurring in writing with the parameters in this report, or by providing alternative parameters.

Materials Testing Black & Veatch or the facility owner should consider retaining a geotechnical engineer to perform materials testing during construction. The performance of such testing or lack thereof, however, in no way alleviates the burden of the contractor or subcontractor from constructing in a manner that conforms to applicable project documents and industry standards. The contractor or pertinent subcontractor is ultimately responsible for managing the quality of his work; furthermore, testing by the geotechnical engineer does not preclude the contractor from obtaining or providing whatever services that he deems necessary to complete the project in accordance with applicable documents.

Limitations This report has been prepared for Black & Veatch as it pertains to design of the proposed water tank and pump station as described herein. It should not be assumed to contain sufficient information for other parties or other purposes. Black & Veatch has agreed to the terms, conditions, and liability limitations outlined in our proposal and associated contract documents between Black & Veatch and GROUND. Reliance upon our report is not granted to any other potential owner, contractor, or lender.

In addition, GROUND has assumed that project construction will commence by fall 2025 and end within 1 year of the project commencement date. Any changes in project plans or schedule should be brought to the attention of a geotechnical engineer, in order that the geotechnical conclusions in this report may be reevaluated and, as necessary, modified. **If our described understanding/interpretation of the proposed project is incorrect or project elements differ in any way from that expressed herein, including changes to improvement locations, dimensions, orientations, loading conditions, elevations/grades, etc., and/or additional buildings/structures/site improvements are incorporated into this project, either after the original information was provided to us or after the date of this report, GROUND or another geotechnical engineer must be retained to reevaluate the conclusions and parameters presented herein.**

The geotechnical conclusions and parameters in this report were based on subsurface information from a limited number of exploration points, as shown in Figure 1, as well as the means and methods described herein. Subsurface conditions were interpolated between and extrapolated beyond these locations. It is not possible to guarantee the subsurface conditions are as indicated in this report. Actual conditions exposed during construction may differ from those encountered during site exploration. Design modifications may be necessary by the project team; this may result in an increase in project costs and schedule delays. In addition, a contractor who obtains information from this report for development of his scope of work or cost estimates does so solely at his own risk and may find the geotechnical information in this report to be inadequate for his purposes or find the geotechnical conditions described herein to be at variance with his experience in the greater project area. The contractor should obtain the additional geotechnical information that is necessary to develop his workscope and cost estimates with sufficient precision. This includes, but is not limited to, information regarding excavation conditions, earth material usage, current depths to groundwater, etc. Because of the necessarily limited nature of the subsurface exploration performed for this study, the contractor should be allowed to evaluate the site using test pits or other means to obtain additional subsurface information to prepare his bid.

If during construction, surface, soil, bedrock, or groundwater conditions appear to be at variance with those described herein, work should cease and a geotechnical engineer should be retained at once, so that our conclusions and design parameters for this site may be reevaluated in a timely manner and dependent aspects of project design can be modified, as necessary.

The materials present on-site are stable at their natural moisture content, but may change volume or lose bearing capacity or stability with changes in moisture content. Performance of the proposed structure and pavement will depend on implementation of the conclusions and information in this report and on proper maintenance after construction is completed. Because water is a significant cause of volume change in soils and rock, allowing moisture infiltration may result in movements, some of which will exceed estimates provided herein and should therefore be expected by Black & Veatch.

ALL DEVELOPMENT CONTAINS INHERENT RISKS. It is important that ALL aspects of this report, as well as the estimated performance (and limitations with any such estimations) of proposed improvements are understood by Black & Veatch. Utilizing the geotechnical parameters and measures herein for planning, design, and/or construction constitutes understanding and acceptance of the conclusions with regard to risk and other information provided herein, associated improvement performance, as well as the limitations inherent within such estimates. Ensuring correct interpretation of the contents of this report by others is not the responsibility of GROUND. If any information referred to herein is not well understood, it is imperative that Black & Veatch or the owner contact the author or a GROUND principal immediately. We will be available to meet to discuss the risks and remedial approaches presented in this report, as well as other potential approaches, upon request.

This report was prepared in accordance with generally accepted soil and foundation engineering practice in the project area at the date of preparation. Current applicable codes may contain criteria regarding performance of structures and/or site improvements which may differ from those provided herein. Our office should be contacted regarding any apparent disparity.

GROUND makes no warranties, either expressed or implied, as to the professional data, opinions or conclusions contained herein. Because of numerous considerations that are beyond GROUND's control, the economic or technical performance of the project cannot be guaranteed in any respect.

This document, together with the concepts and conclusions presented herein, as an instrument of service, is intended only for the specific purpose and client for which it was prepared. Reuse of, or improper reliance on this document without written authorization and adaption by GROUND Engineering Consultants, Inc., shall be without liability to GROUND Engineering Consultants, Inc.

GROUND appreciates the opportunity to complete this portion of the project and welcomes the opportunity to provide Black & Veatch or the facility owner with a proposal for construction observation and materials testing.

Sincerely,

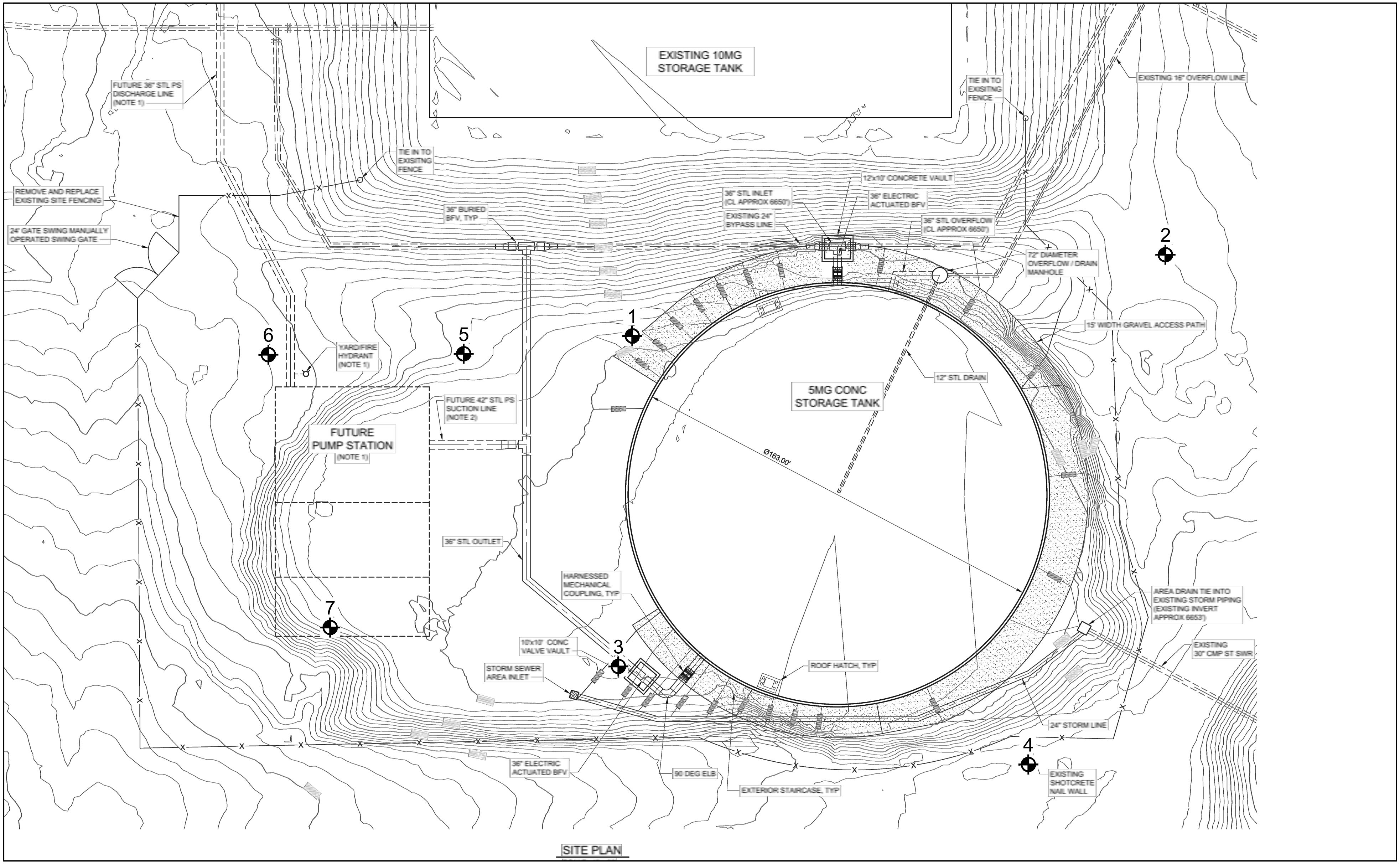
GROUND Engineering Consultants, Inc.

BRIAN H. RECK
48265

Brian H. Reck
5/14/2025

Ben Fellbaum, P.G., P.E.

Reviewed by Brian H. Reck, P.G., C.E.G., P.E.



SITE PLAN

SITE PLAN PROVIDED BY CLIENT

7
⊕ Indicates test hole numbers and approximate locations.

NOT TO SCALE

GROUND
ENGINEERING

JOB NO.: 25-8003a
FIGURE: 1

LOCATION OF TEST HOLES

LOGS OF THE TEST HOLES

PROJECT: Austin Bluffs Tank and Pump Station

CLIENT: Black & Veatch

JOB NO: 25-8003a

SITE LOCATION: Colorado Springs, CO

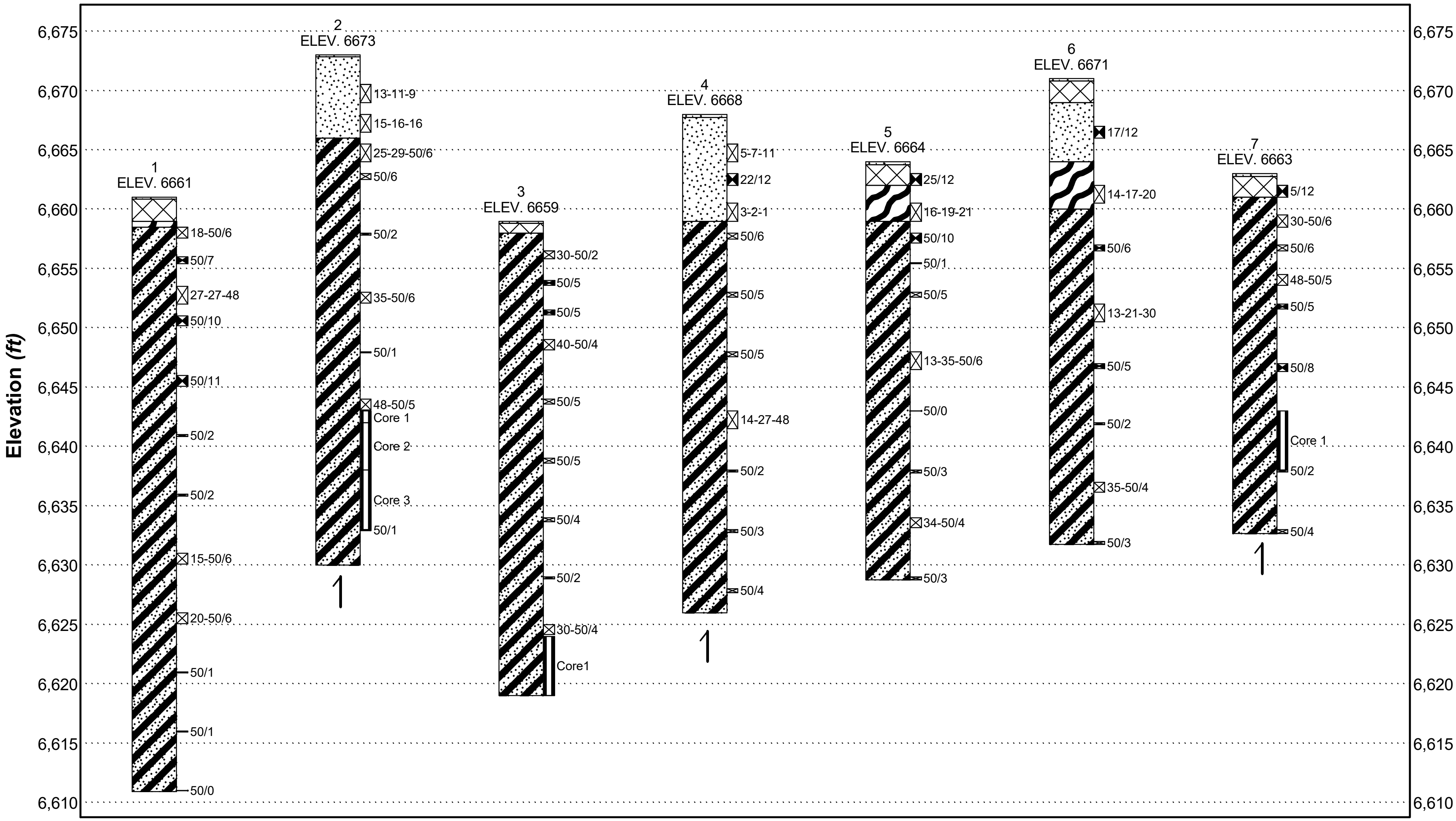


FIGURE: 2

GROUND
ENGINEERING

LEGEND AND NOTES

PROJECT: Austin Bluffs Tank and Pump Station
JOB NO: 25-8003a
CLIENT: Black & Veatch
SITE LOCATION: Colorado Springs, CO

MATERIAL SYMBOLS

⁘	TOPSOIL
⊠	FILL
░	SAND
≋	WEATHERED CLAYSTONE
▨	CLAYSTONE and SANDSTONE BEDROCK

NOTE: See Detailed Logs for Material descriptions.

SAMPLER SYMBOLS

⧗	**Modified California Liner Sampler** 23 / 12 Drive sample blow count indicates 23 blows of a 140 pound hammer falling 30 inches were required to drive the sampler 12 inches.
⊠	**Standard Penetration Test Sampler** 20-25-30 Drive sample blow count, indicates 20, 25, and 30 blows of a 140 pound hammer falling 30 inches were required to drive the sampler 18 inches in three 6 inch increments.

NOTES

1. Test holes were drilled on 3/25/2025, 3/26/2025, and 3/27/2025 with 4" solid stem auger, 3 1/4" ID hollow stem auger, and wireline coring equipment.

2. Locations of the test holes were determined in the field using a hand held GPS device by GROUND.

3. Elevations of test holes were estimated from client provided documents and the logs of test holes are hung to elevation.

4. The test hole locations and elevations should be considered accurate only to the degree implied by the method used.

5. The lines between materials shown on the test hole logs represent the approximate boundaries between material types and the transitions may be gradual.

6. Groundwater level readings shown on the logs were made at the time and under the conditions indicated. Fluctuations in the water level may occur with time.

7. The material descriptions on these logs are for general classification purposes only. See full text of this report for descriptions of the site materials & related information.

8. All test holes were immediately backfilled upon completion of drilling, unless otherwise specified in this report.

ABBREVIATIONS

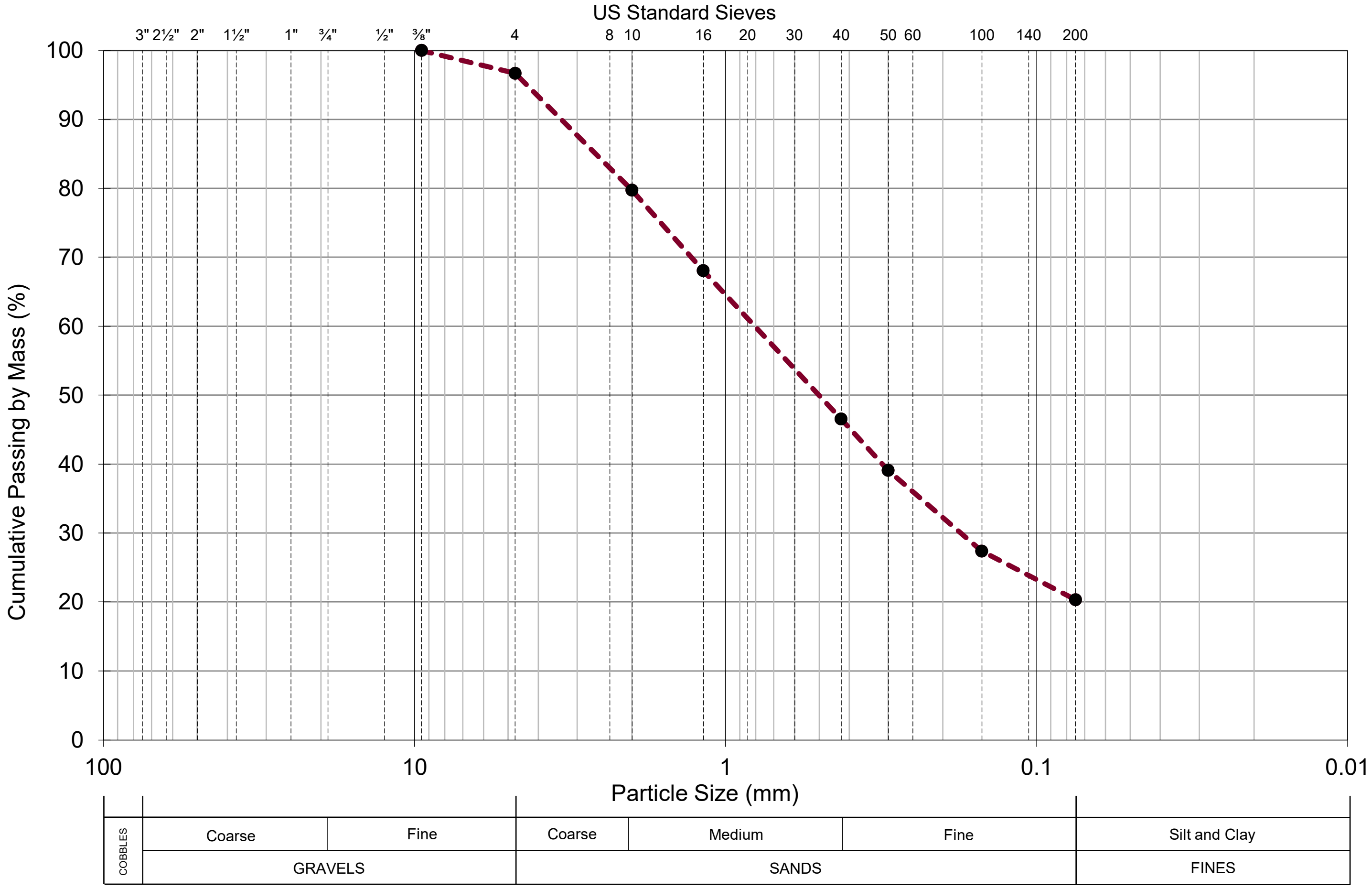
▽	Water Level at Time of Drilling, or as Shown
▼	Water Level at End of Drilling, or as Shown
▼	Water Level After 24 Hours, or as Shown

NV	No Value
NP	Non-Plastic
1	Refusal

FIGURE: 3

Austin Bluffs Tank and Pump Station

Gradation (ASTM D422-63[2007])



Coarse Gradation			Fine Gradation			Grading	
US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	Coefficient	Value
6 in	150	-	No. 4	4.75	97	D_{90}	3.378
5 in	125	-	No. 8	2.36	-	D_{85}	2.618
4 in	100	-	No. 10	2.00	80	D_{80}	2.029
3 in	75	-	No. 16	1.18	68	D_{60}	0.805
2.5 in	63	-	No. 20	0.85	-	D_{50}	0.501
2 in	50	-	No. 30	0.60	-	D_{40}	0.313
1.5 in	37.5	-	No. 40	0.425	47	D_{30}	0.175
1 in	25.0	-	No. 50	0.300	39	D_{15}	-
3/4 in	19.0	-	No. 60	0.250	-	D_{10}	-
1/2 in	12.5	-	No. 100	0.150	27	D_{05}	-
3/8 in	9.5	100	No. 140	0.106	-	C_u	-
No. 4	4.75	97	No. 200	0.075	20.3	C_c	-

Location: Test Hole 3 at 2.5 feet
Description: SANDSTONE Bedrock

Classification: SM / A-1-b (0)
Liquid Limit: NV
Plasticity Index: NP

Gravel (%): 3
Sand (%): 77
Silt/Clay (%): 20.3

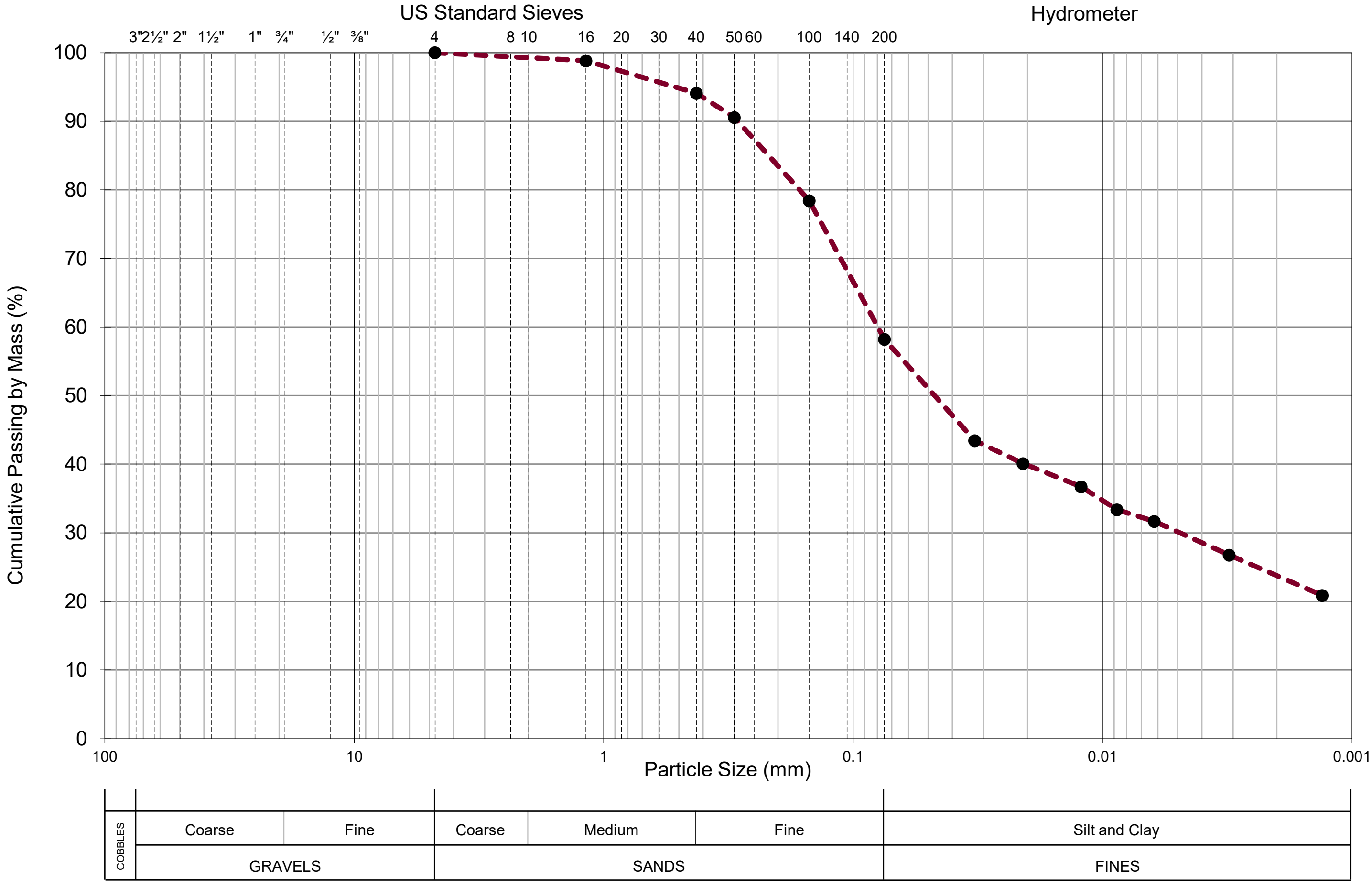
Figure 4

GROUND ENGINEERING

Job No.: 25-8003a

Austin Bluffs Tank and Pump Station

Gradation and Hydrometer (ASTM D422-63[2007])



Coarse Gradation			Fine Gradation			Hydrometer		Grading	
US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	Particle Size (mm)	Passing by Mass (%)	Coefficient	Value
6 in	150	-	No. 4	4.75	100	0.033	43	D_{90}	0.290
5 in	125	-	No. 8	2.36	-	0.021	40	D_{85}	0.218
4 in	100	-	No. 10	2.00	-	0.012	37	D_{80}	0.164
3 in	75	-	No. 16	1.18	99	0.009	33	D_{60}	0.080
2.5 in	63	-	No. 20	0.85	-	0.006	32	D_{50}	0.047
2 in	50	-	No. 30	0.60	-	0.003	27	D_{40}	0.021
1.5 in	37.5	-	No. 40	0.425	94	0.001	21	D_{30}	0.005
1 in	25.0	-	No. 50	0.300	91	-	-	D_{15}	-
3/4 in	19.0	-	No. 60	0.250	-	-	-	D_{10}	-
1/2 in	12.5	-	No. 100	0.150	78	-	-	D_{05}	-
3/8 in	9.5	-	No. 140	0.106	-	-	-	C_u	-
No. 4	4.75	100	No. 200	0.075	58.2	-	-	C_c	-

Location: Test Hole 1 at 2.5 feet
Description: CLAYSTONE Bedrock

Classification: s(CL) / A-6 (4)
Liquid Limit: 27
Plasticity Index: 12
Activity: 0.5

Gravel (%): 0
Sand (%): 42
Silt/Clay (%): 58.2
< .002 mm (%): 24

Results apply only to the specific items and locations referenced and at the time of testing. For the hydrometer portion of the test, a composite temperature correction and meniscus correction were applied to each reading. This report should not be reproduced, except in full, without the written permission of GROUND Engineering Consultants, Inc.

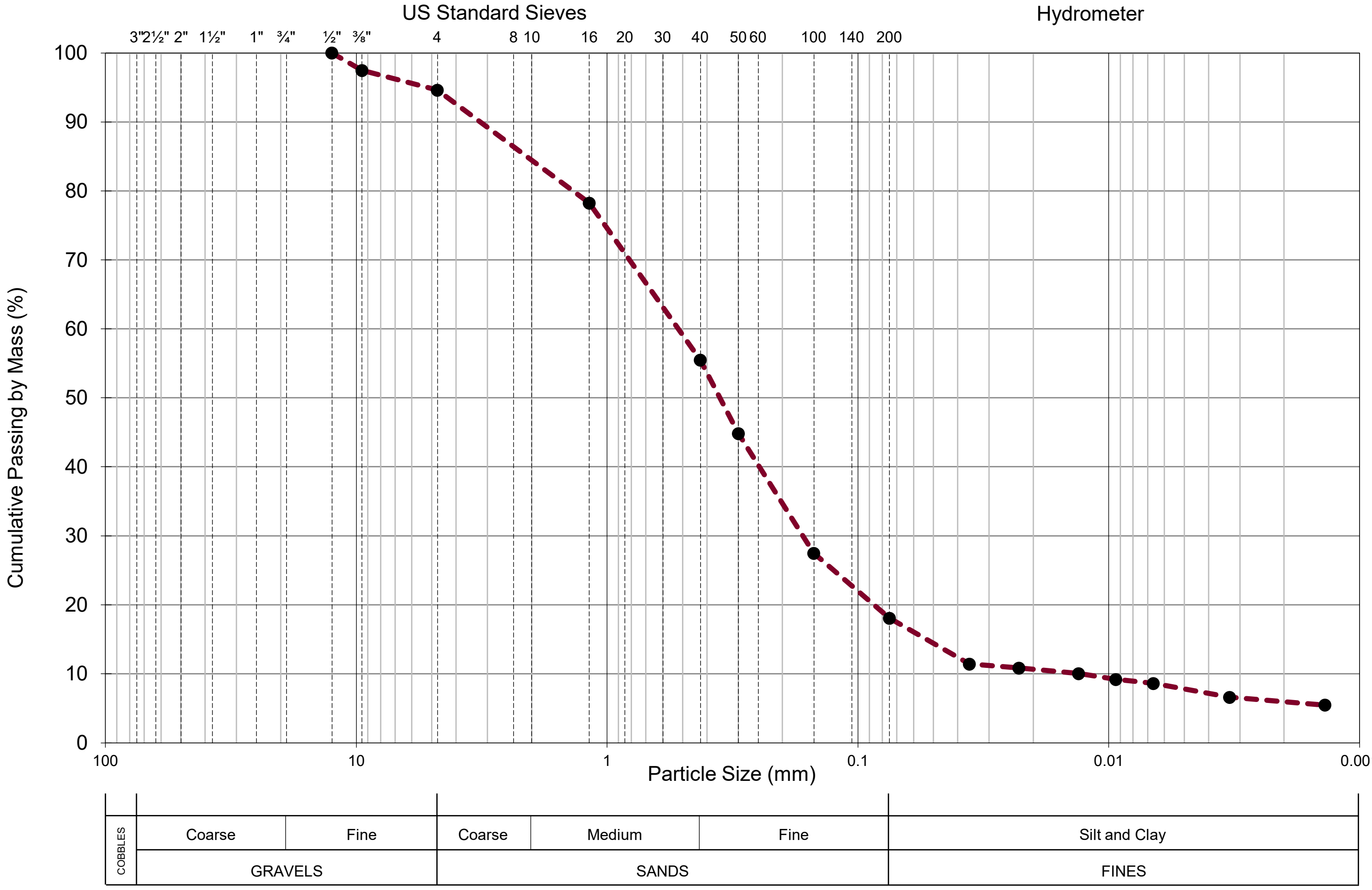
Figure 5

GROUND ENGINEERING

Job No.: 25-8003a

Austin Bluffs Tank and Pump Station

Gradation and Hydrometer (ASTM D422-63[2007])



Coarse Gradation			Fine Gradation			Hydrometer		Grading	
US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	Particle Size (mm)	Passing by Mass (%)	Coefficient	Value
6 in	150	-	No. 4	4.75	95	0.036	11	D_{90}	3.206
5 in	125	-	No. 8	2.36	-	0.023	11	D_{85}	2.097
4 in	100	-	No. 10	2.00	-	0.013	10	D_{80}	1.372
3 in	75	-	No. 16	1.18	78	0.009	9	D_{60}	0.520
2.5 in	63	-	No. 20	0.85	-	0.007	9	D_{50}	0.355
2 in	50	-	No. 30	0.60	-	0.003	7	D_{40}	0.247
1.5 in	37.5	-	No. 40	0.425	55	0.001	5	D_{30}	0.166
1 in	25.0	-	No. 50	0.300	45	-	-	D_{15}	0.053
3/4 in	19.0	-	No. 60	0.250	-	-	-	D_{10}	0.013
1/2 in	12.5	100	No. 100	0.150	27	-	-	D_{05}	-
3/8 in	9.5	97	No. 140	0.106	-	-	-	C_u	39.764
No. 4	4.75	95	No. 200	0.075	18.1	-	-	C_c	4.037

Location: Test Hole 7 at 3.5 feet
Description: Silty SAND

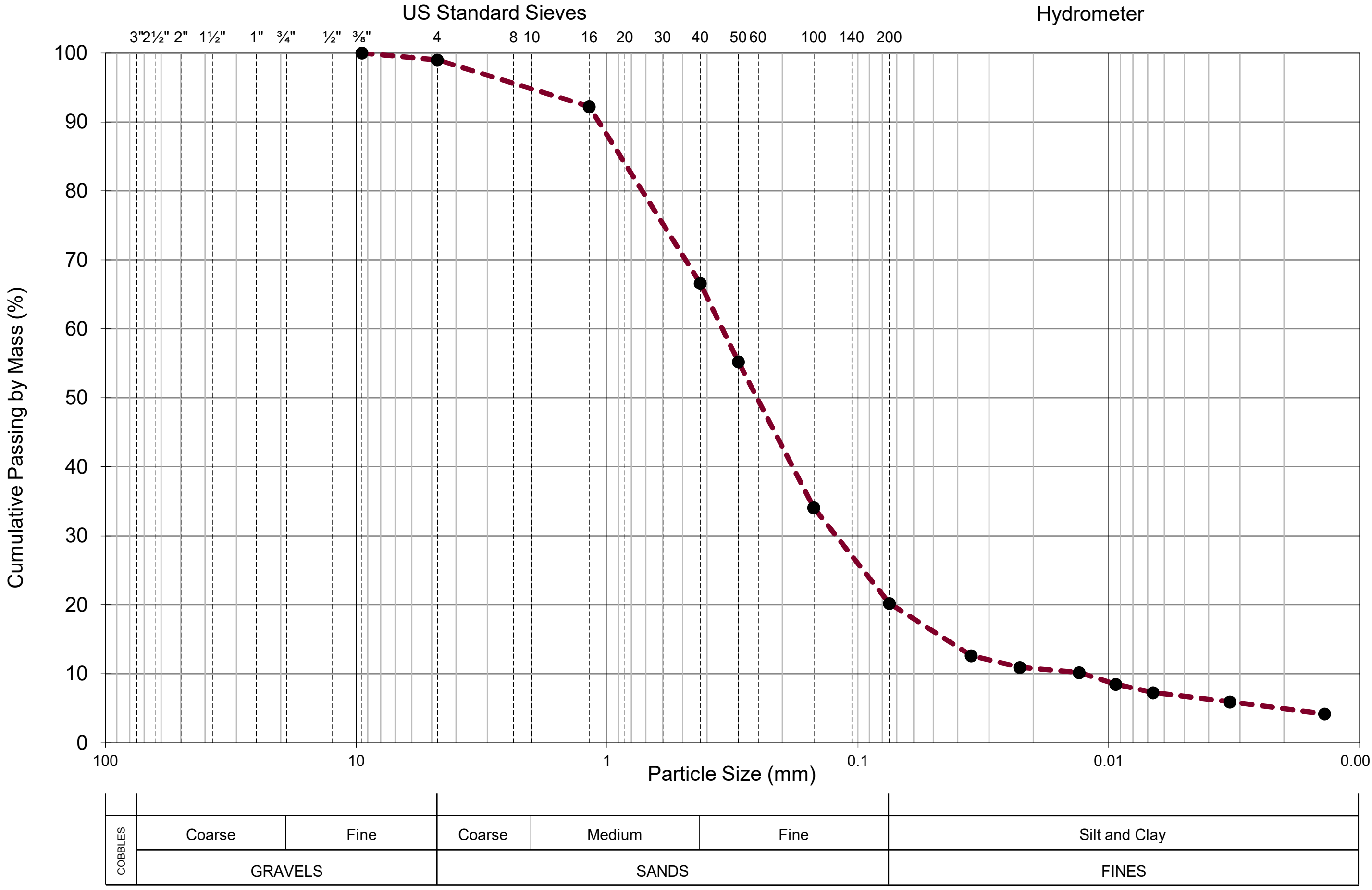
Classification: SM / A-2-4 (0)
Liquid Limit: NV
Plasticity Index: NP
Activity: -

Gravel (%): 5
Sand (%): 77
Silt/Clay (%): 18.1
< .002 mm (%): 6

Results apply only to the specific items and locations referenced and at the time of testing. For the hydrometer portion of the test, a composite temperature correction and meniscus correction were applied to each reading. This report should not be reproduced, except in full, without the written permission of GROUND Engineering Consultants, Inc.

Austin Bluffs Tank and Pump Station

Gradation and Hydrometer (ASTM D422-63[2007])



Coarse Gradation			Fine Gradation			Hydrometer		Grading	
US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	Particle Size (mm)	Passing by Mass (%)	Coefficient	Value
6 in	150	-	No. 4	4.75	99	0.035	13	D_{90}	1.080
5 in	125	-	No. 8	2.36	-	0.023	11	D_{85}	0.885
4 in	100	-	No. 10	2.00	-	0.013	10	D_{80}	0.725
3 in	75	-	No. 16	1.18	92	0.009	8	D_{60}	0.347
2.5 in	63	-	No. 20	0.85	-	0.007	7	D_{50}	0.253
2 in	50	-	No. 30	0.60	-	0.003	6	D_{40}	0.182
1.5 in	37.5	-	No. 40	0.425	67	0.001	4	D_{30}	0.122
1 in	25.0	-	No. 50	0.300	55	-	-	D_{15}	0.045
3/4 in	19.0	-	No. 60	0.250	-	-	-	D_{10}	0.013
1/2 in	12.5	-	No. 100	0.150	34	-	-	D_{05}	0.002
3/8 in	9.5	100	No. 140	0.106	-	-	-	C_u	27.424
No. 4	4.75	99	No. 200	0.075	20.2	-	-	C_c	3.402

Location: Test Hole 3 at 7.5 feet
Description: SANDSTONE Bedrock

Classification: SM / A-2-4 (0)
Liquid Limit: NV
Plasticity Index: NP
Activity: -

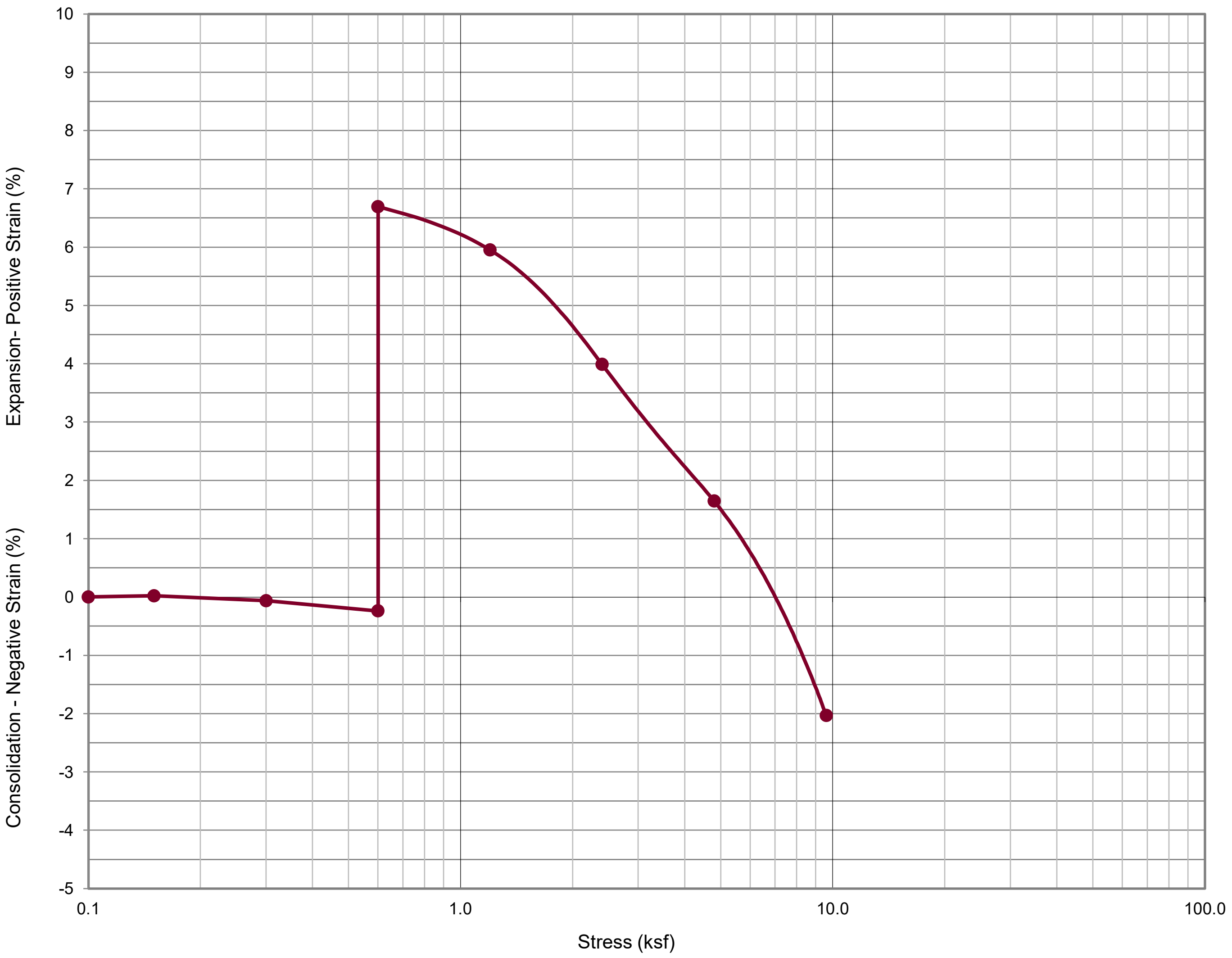
Gravel (%): 1
Sand (%): 79
Silt/Clay (%): 20.2
< .002 mm (%): 5

Results apply only to the specific items and locations referenced and at the time of testing. For the hydrometer portion of the test, a composite temperature correction and meniscus correction were applied to each reading. This report should not be reproduced, except in full, without the written permission of GROUND Engineering Consultants, Inc.

Figure 7

Austin Bluffs Tank and Pump Station

Denver Swell Test



Swell / Collapse *(%)* *
6.9

* Negative indicates collapse.

Location: 1 at 5 feet

Description: CLAYSTONE Bedrock

Classification: s(CL) / A-6 (6)

< No. 200 (%): 62.3

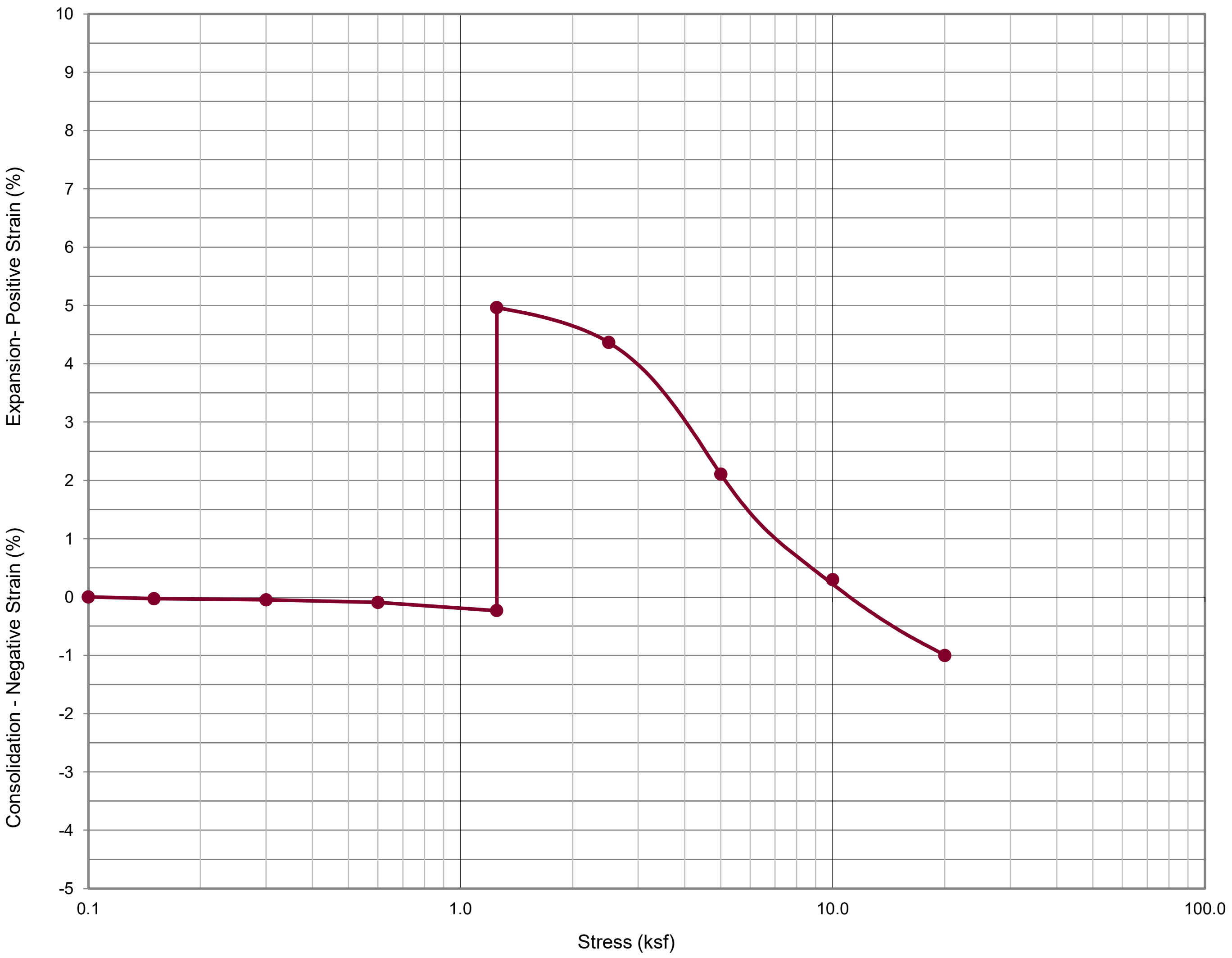
Liquid Limit: 31

Plasticity Index: 14

Tested in general accordance with ASTM D4546 (1986) and published research by F.H. Chen (1988). Results apply only to the specific items and locations referenced and at the time of testing. This report should not be reproduced, except in full, without the written permission of GROUND Engineering Consultants, Inc.

Austin Bluffs Tank and Pump Station

Denver Swell Test



Swell / Collapse *(%)* *
5.2

* Negative indicates collapse.

Location: 1 at10feet

Description: CLAYSTONE Bedrock

Classification: (CL)s / A-6 (15)

< No. 200 (%): 82.4

Liquid Limit: 39

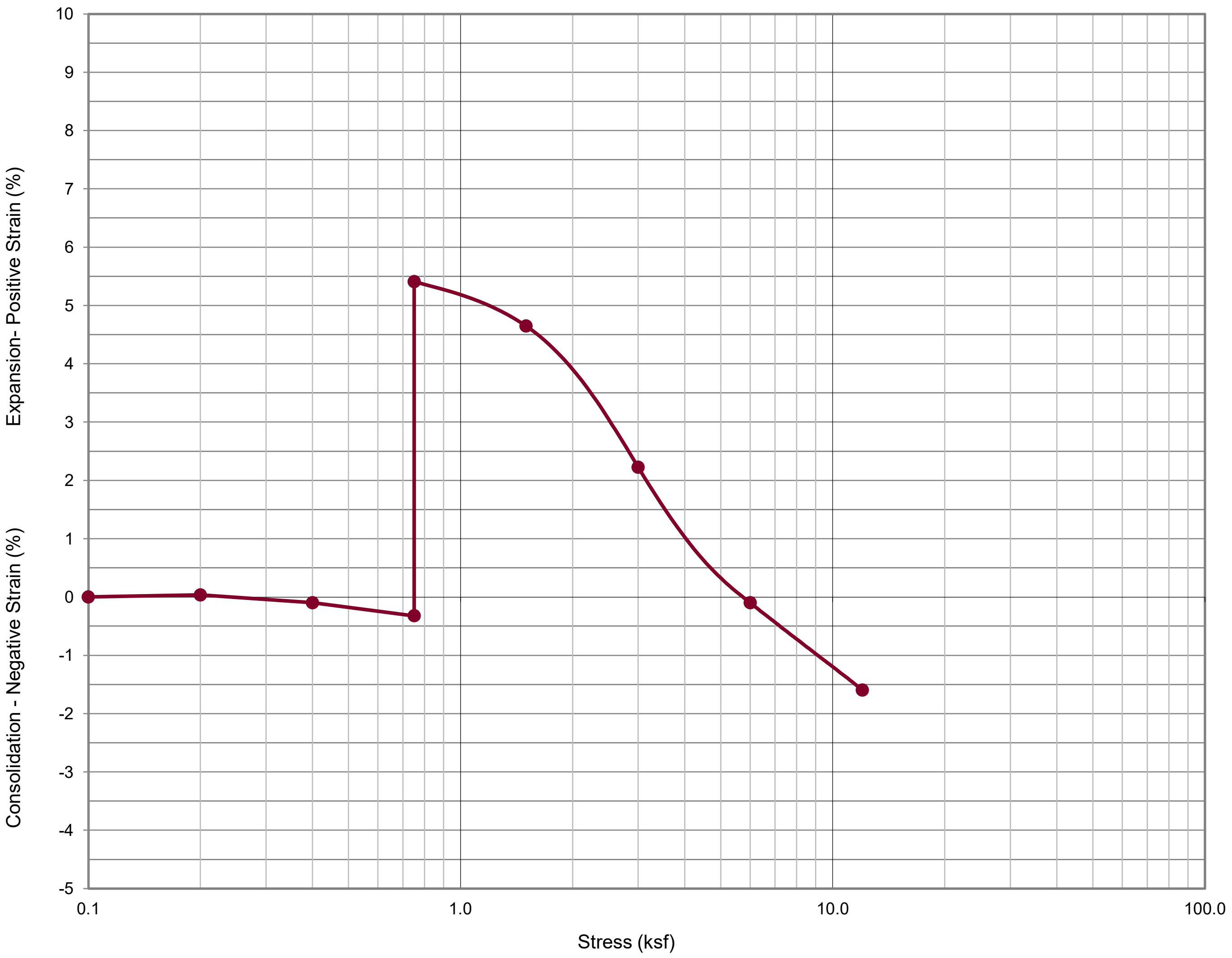
Plasticity Index: 19

Tested in general accordance with ASTM D4546 (1986) and published research by F.H. Chen (1988). Results apply only to the specific items and locations referenced and at the time of testing. This report should not be reproduced, except in full, without the written permission of GROUND Engineering Consultants, Inc.

Job No.: 25-8003a

Austin Bluffs Tank and Pump Station

Denver Swell Test



Swell / Collapse *(%)* *
5.7

* Negative indicates collapse.

Location: 5 at 6 feet

Description: CLAYSTONE Bedrock

Classification: (CL)s / A-6 (12)

< No. 200 (%): 75.2

Liquid Limit: 35

Plasticity Index: 18

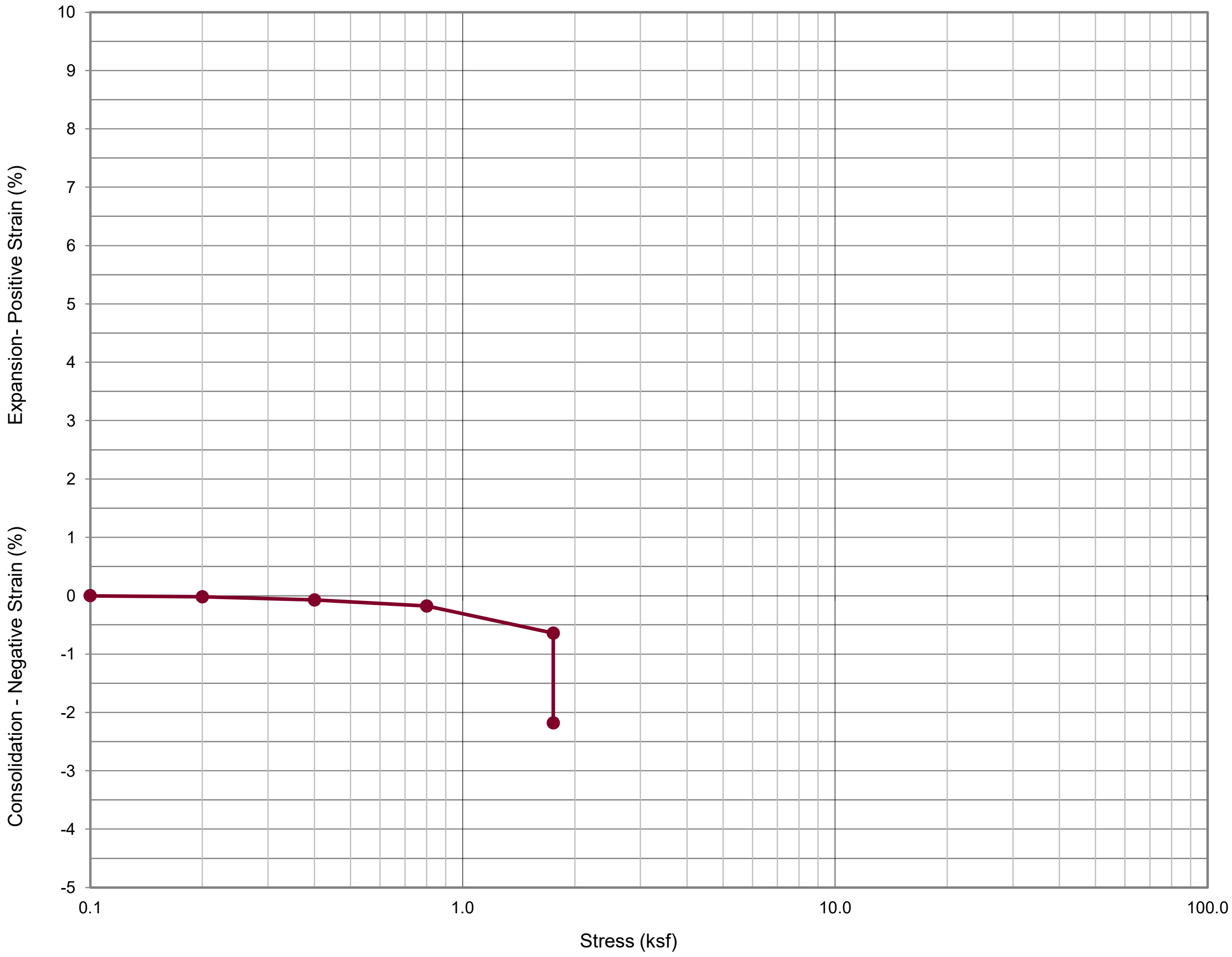
Tested in general accordance with ASTM D4546 (1986) and published research by F.H. Chen (1988). Results apply only to the specific items and locations referenced and at the time of testing. This report should not be reproduced, except in full, without the written permission of GROUND Engineering Consultants, Inc.

GROUND ENGINEERING

Job No.: 25-8003a

Austin Bluffs Tank and Pump Station

Denver Swell Test



Swell / Collapse *(%) **
-1.5

* Negative indicates collapse.

Location: 6 at 14 feet

Description: SANDSTONE Bedrock

Classification: SC / A-6 (1)

< No. 200 (%): 38.3

Liquid Limit: 27

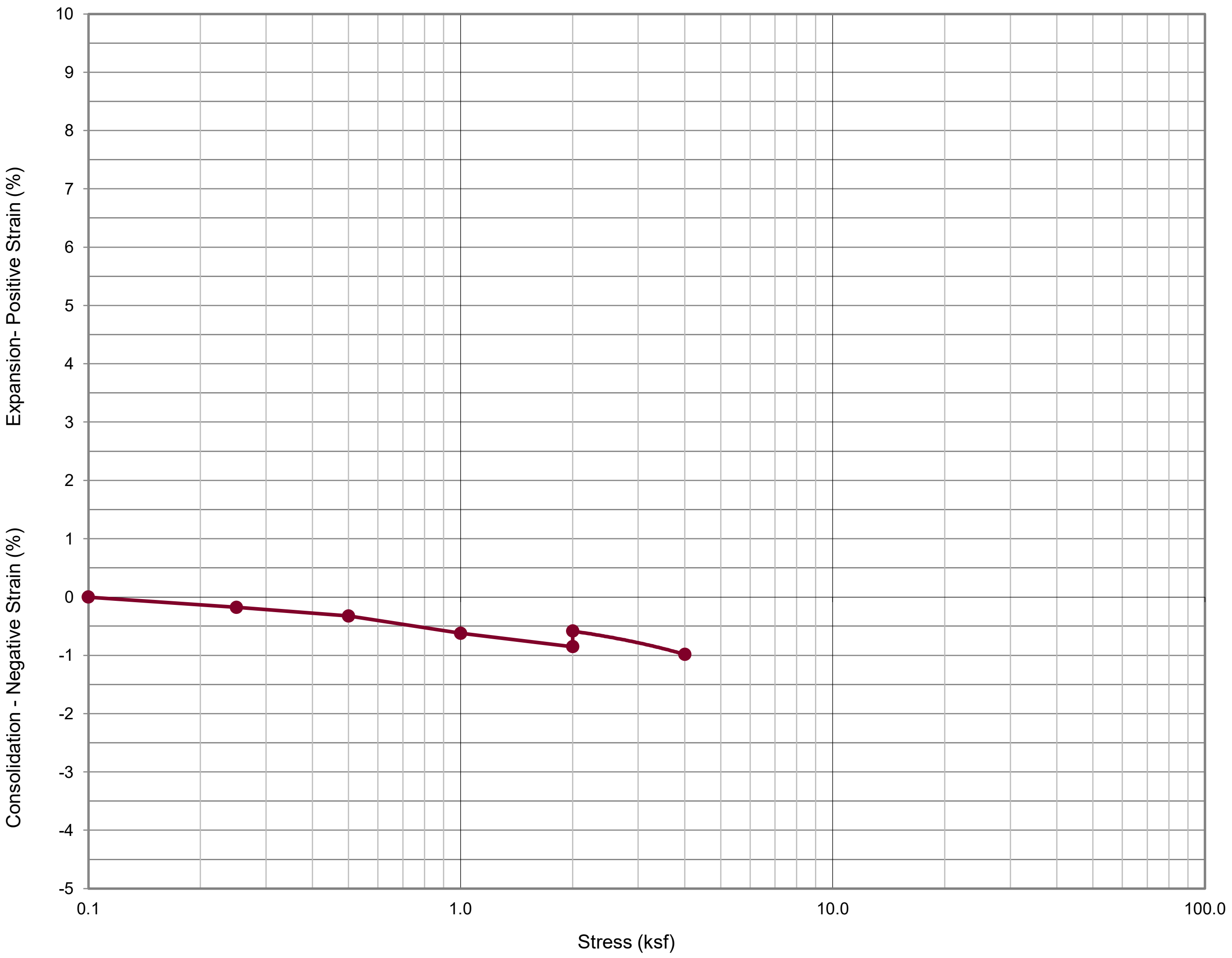
Plasticity Index: 11

Tested in general accordance with ASTM D4546 (1986) and published research by F.H. Chen (1988). Results apply only to the specific items and locations referenced and at the time of testing. This report should not be reproduced, except in full, without the written permission of GROUND Engineering Consultants, Inc.

Figure 11

Austin Bluffs Tank and Pump Station

Denver Swell Test



Swell / Collapse *(%)* *
0.3

* Negative indicates collapse.

Location: 7 at 16 feet

Description: CLAYSTONE Bedrock

Classification: s(CL) / A-6 (4)

< No. 200 (%): 55.9

Liquid Limit: 27

Plasticity Index: 13

Tested in general accordance with ASTM D4546 (1986) and published research by F.H. Chen (1988). Results apply only to the specific items and locations referenced and at the time of testing. This report should not be reproduced, except in full, without the written permission of GROUND Engineering Consultants, Inc.

Austin Bluffs Tank and Pump Station

TABLE 1: SUMMARY OF LABORATORY TEST RESULTS

Sample Location		Natural Moisture Content	Natural Dry Density	Gradation			Atterberg Limits		Swell/Consolidation		Unconfined Compressive Strength		USCS Equivalent Classification	AASHTO Equivalent Classification (Group Index)	Sample Description
Test Hole No.	Depth (feet)	(%)	(pcf)	Gravel (%)	Sand (%)	Fines (%)	Liquid Limit	Plasticity Index	Volume Change (%)	Surcharge Pressure (psf)	(psi)	(ksf)			
1	2.5	7.4	SD	0	42	58.2	27	12	-	-	-	-	s(CL)	A-6 (4)	CLAYSTONE Bedrock
1	5	9.7	126.6	0	38	62.3	31	14	7.0	600	-	-	s(CL)	A-6 (6)	CLAYSTONE Bedrock
1	10	11.1	123.1	0	18	82.4	39	19	5.2	1,250	-	-	(CL)s	A-6 (15)	CLAYSTONE Bedrock
1	15	12.6	123.1	0	27	73.5	37	18	-	-	90.5	13.03	(CL)s	A-6 (12)	CLAYSTONE Bedrock
2	5	5.7	-	5	64	30.8	18	2	-	-	-	-	SM	A-2-4 (0)	Silty SAND
2	20	9.6	-	1	43	55.6	34	16	-	-	-	-	s(CL)	A-6 (6)	CLAYSTONE Bedrock
2	31	-	-	-	-	-	-	-	-	-	622.8	89.68	-	-	CLAYSTONE Bedrock
3	2.5	5.8	-	3	77	20.3	NV	NP	-	-	-	-	SM	A-1-b (0)	SANDSTONE Bedrock
3	7.5	6.3	-	1	79	20.2	NV	NP	-	-	-	-	SM	A-2-4 (0)	SANDSTONE Bedrock
4	2.5	7.1	-	3	67	29.9	22	7	-	-	-	-	SC-SM	A-2-4 (0)	Silty, Clayey SAND
4	15	6.7	-	2	65	33.5	20	5	-	-	-	-	SC-SM	A-2-4 (0)	SANDSTONE Bedrock
5	1	3.5	108.0	4	69	26.7	24	7	-	-	-	-	SC-SM	A-2-4 (0)	FILL: Silty, Clayey SAND
5	6	10.7	124.9	0	25	75.2	35	18	5.7	750	-	-	(CL)s	A-6 (12)	CLAYSTONE Bedrock
6	9	11.7	119.2	0	24	75.9	34	17	-	-	-	-	(CL)s	A-6 (11)	WEATHERED CLAYSTONE
6	14	7.5	131.3	4	58	38.3	27	11	-1.5	1,750	-	-	SC	A-6 (1)	SANDSTONE Bedrock
6	24	8.3	115.3	0	40	60.4	26	8	-	-	74.5	10.73	s(CL)	A-4 (2)	CLAYSTONE Bedrock
7	1	11.4	120.7	1	62	36.9	23	6	-	-	-	-	SC-SM	A-4 (0)	FILL: Silty, Clayey SAND
7	3.5	5.8	-	6	76	18.2	NV	NP	-	-	-	-	SM	A-2-4 (0)	Silty SAND
7	16	8.4	131.6	0	44	55.9	27	13	0.2	2,000	-	-	s(CL)	A-6 (4)	CLAYSTONE Bedrock

SD = Sample disturbed, NV = No value, NP = Non-plastic

Austin Bluffs Tank and Pump Station

TABLE 2: SUMMARY OF SOIL CORROSION TEST RESULTS

Sample Location		Water Soluble Sulfates	pH	Redox Potential	Sulfide Reactivity	Resistivity	Chlorides	USCS Equivalent Classification	AASHTO Equivalent Classification (Group Index)	Sample Description
Test Hole No.	Depth *(feet)*	*(%)*		*(mv)*		*(ohm-cm)*	*(ppm)*			
6	9	0.01	8.4	- 66	Trace	5,521	17	(CL)s	A-6 (11)	WEATHERED CLAYSTONE
7	1	0.01	8.6	- 74	Positive	10,558	< 10	SC-SM	A-4 (0)	FILL: Silty, Clayey SAND

Job 25-8003a

Appendix A

Detail Logs of Test Holes

GROUND
ENGINEERING

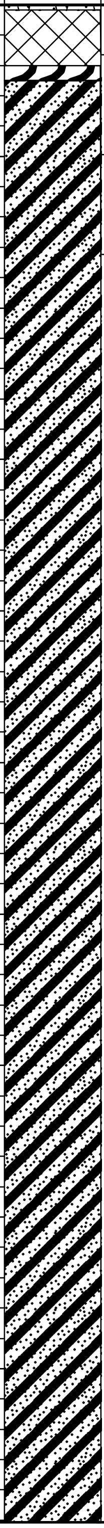
TEST HOLE 1

PAGE 1 OF 1

PROJECT: Austin Bluffs Tank and Pump Station
JOB NO: 25-8003a
CLIENT: Black & Veatch
SITE LOCATION: Colorado Springs, CO

Elevation (ft)	Depth (ft)
6661	0
6656	5
6651	10
6646	15
6641	20
6636	25
6631	30
6626	35
6621	40
6616	45
6611	50

Graphic Log



Material Descriptions and Drilling Notes

TOIPSOIL: Approximately 2 inches of topsoil.

FILL: Fine to coarse, clayey to silty sands with local gravels. They were non- to slightly plastic, dry to moist, and light brown to brown to gray-brown in color.

WEATHERED CLAYSTONE: Moderately plastic, medium hard, moist, and gray in color. Secondary carbonates were noted locally.

CLAYSTONE and SANDSTONE: Highly interbedded claystones and fine to coarse sandstones. They were poorly to well-cemented, non- to moderately plastic, hard to very hard, slightly moist to very moist, and light brown to brown to gray to dark gray in color. Iron staining was noted commonly, lignite and secondary carbonates were noted locally.

Auger crunching/grinding noted at 6.5 feet, indicating cemented bedrock.

Severe auger crunching/grinding noted at 16.5 feet, indicating cemented bedrock.

Refusal at 17', offset 8' feet south.

Severe auger crunching/grinding noted at 38 feet, indicating cemented bedrock.

Bottom of test hole at approx. 50.08 feet.

Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Gradation: Gravel %	Gradation: Sand %	Gradation: Fines %	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	Swell/Consolidation (%) at Surcharge Pressure (psf)	USCS Equivalent Classification
☒	18-50/6	7.4	SD		42	58	27	12		s(CL)
⧓	50/7	9.7	126.6		38	62	31	14	7 (-)	s(CL)
☒	27-27-48									
⧓	50/10									
⧓	50/11	12.6	123.1		26	74	37	18		(CL)s
—	50/2									
—	50/2									
☒	15-50/6									
☒	20-50/6									
—	50/1									
—	50/1									
—	50/0									

GROUND
ENGINEERING

TEST HOLE 2

PAGE 1 OF 1

PROJECT: Austin Bluffs Tank and Pump Station **JOB NO:** 25-8003a

CLIENT: Black & Veatch **SITE LOCATION:** Colorado Springs, CO

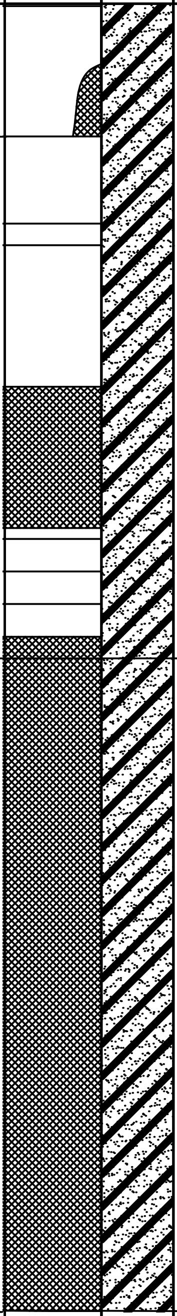
Elevation (ft)	Depth (ft)	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Gradation			Atterberg Limits		Swell/Consolidation (%) at Surcharge Pressure (psf)	USCS Equivalent Classification
								Gravel %	Sand %	Fines %	Liquid Limit	Plasticity Index		
6673	0		**TOIPSOIL:** Approximately 2 inches of topsoil.											
			SAND: Fine to coarse, clayey to silty sands with local gravels. They were non- to slightly plastic, loose to dense, dry to moist, and light brown to brown to gray-brown in color. Iron staining was noted locally.	☒	13-11-9									
6668	5		*Clay lens about 5 inches thick noted in 5 foot driven sample.*	☒	15-16-16	5.7		5	64	31	18	2		SM
			CLAYSTONE and SANDSTONE: Highly interbedded claystones and fine to coarse sandstones. They were poorly to well-cemented, non- to moderately plastic, hard to very hard, slightly moist to very moist, and light brown to brown to gray to dark gray in color. Iron staining was noted commonly, lignite and secondary carbonates were noted locally.	☒	25-29-50/6									
6663	10			☒	50/6									
6658	15				50/2									
6653	20			☒	35-50/6	9.6		1	43	56	34	16		s(CL)
6648	25				50/1									
6643	30			☒	48-50/5									
			Wireline coring performed from 30 to 40 feet. NO coring recovery from 35 to 40 feet, likely due to uncemented bedrock.	▮	Core 1									
				▮	Core 2									
6638	35			▮	Core 3									
6633	40				50/1									

Refusal at 43 feet.

GROUND ENGINEERING

Job Number: 25-8003a	Test Hole: 2	Project Name: Austin Bluffs Tank and Pump Station	Elevation: 6673	**Legend**: M - Mechanical Break S - Stepped Fracture
Logged By: CJ	Page: 1 of 1	Project Location: Colorado Springs, CO	Bedrock Depth: 7.0 Ft ±	Planarity Index: P1 - Planar P2 - Somewhat Planar P3 - Somewhat Irregular P4 - Irregular
Date Drilled: 3/27/2025	Hole Diameter: 3 1/4"	Client: BLACK & VEATCH	Total Depth: 43.00 Ft ±	Roughness Index: R1 - Smooth R2 - Fairly Smooth R3 - Fairly Rough R4 - Rough
		File Name: 8003CLs.DWG	Plunge: 90°	

DEPTH (FEET)	SAMPLE DATA TYPE	BLOW COUNT	CORE DATA PERCENT RECOVERY	RQD	FRACTURES DESCRIPTION	LOG	GRAPHIC LOG	LITHOLOGIC DESCRIPTIONS	DRILLING CONDITIONS NOTES
30			0%	0%	No recovery			CLAYSTONE and SANDSTONE: Highly interbedded claystones and fine to coarse sandstones. They were poorly to well-cemented, non- to moderately plastic, hard to very hard, slightly moist to very moist, and light brown to brown to gray to dark gray in color. Iron staining was noted commonly, lignite and secondary carbonates were noted locally	Begin Run 1 End Run 1
31					M M				Begin Run 2
32									
33			69%	69%	No recovery				
34					M M M No recoivery				End Run 2
35			0%	0%					Begin Run 3
36									
37									
38									
39									End Run 4
40									



GROUND ENGINEERING

TEST HOLE 3

PAGE 1 OF 1

PROJECT: Austin Bluffs Tank and Pump Station **JOB NO:** 25-8003a

CLIENT: Black & Veatch **SITE LOCATION:** Colorado Springs, CO

Elevation *(ft)*	Depth *(ft)*	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content *(%)*	Natural Dry Density *(pcf)*	Gradation Gravel %	Gradation Sand %	Gradation Fines %	Atterberg Limits Liquid Limit	Atterberg Limits Plasticity Index	Swell/Consolidation (%) at Surcharge Pressure *(psf)*	USCS Equivalent Classification
6659	0		**TOIPSOIL:** Approximately 2 inches of topsoil.											
			FILL: Fine to coarse, clayey to silty sands with local gravels. They were non- to slightly plastic, dry to moist, and light brown to brown to gray-brown in color.		30-50/2	5.8		3	77	20	NV	NP		SM
6654	5		**CLAYSTONE and SANDSTONE:** Highly interbedded claystones and fine to coarse sandstones. They were poorly to well-cemented, non- to moderately plastic, hard to very hard, slightly moist to very moist, and light brown to brown to gray to dark gray in color. Iron staining was noted commonly, lignite and secondary carbonates were noted locally.		50/5									
					50/5	6.3		1	79	20	NV	NP		SM
6649	10				40-50/4									
6644	15				50/5									
6639	20				50/5									
6634	25				50/4									
6629	30				50/2									
6624	35				30-50/4									
			Wireline coring performed from 35 to 40 feet.		Core1									
6619	40													

Bottom of test hole at approx. 40 feet.

GROUND
ENGINEERING

Job Number: 25-8003a	Test Hole: 3	Project Name: Austin Bluffs Tank and Pump Station	Elevation: 6659	**Legend**: M - Mechanical Break S - Stepped Fracture
Logged By: CJ	Page: 1 of 1	Project Location: Colorado Springs, CO	Bedrock Depth: 1.0 Ft ±	Planarity Index: P1 - Planar P2 - Somewhat Planar P3 - Somewhat Irregular P4 - Irregular
		Client: BLACK & VEATCH	Total Depth: 40.0 Ft ±	Roughness Index: R1 - Smooth R2 - Fairly Smooth R3 - Fairly Rough R4 - Rough
Date Drilled: 3/25/2025	Hole Diameter: 3 1/4"	File Name: 6003CLs.DWG	Plunge: 90°	

DEPTH (FEET)	SAMPLE DATA		CORE DATA				GRAPHIC LOG	LITHOLOGIC DESCRIPTIONS	DRILLING CONDITIONS NOTES
	TYPE	BLOW COUNT	PERCENT RECOVERY	RQD	FRACTURES				
					DESCRIPTION	LOG			
35						▩	▨	CLAYSTONE and SANDSTONE: Highly interbedded claystones and fine to coarse sandstones. They were poorly to well-cemented, non- to moderately plastic, hard to very hard, slightly moist to very moist, and light brown to brown to gray to dark gray in color. Iron staining was noted commonly, lignite and secondary carbonates were noted locally	Begin Run 1
36									
					No recovery				
37			40%	40%					
38									
					M				
39					M				
40									End Run 1

GROUND
ENGINEERING

TEST HOLE 4

PAGE 1 OF 1

PROJECT: Austin Bluffs Tank and Pump Station **JOB NO:** 25-8003a

CLIENT: Black & Veatch **SITE LOCATION:** Colorado Springs, CO

Elevation *(ft)*	Depth *(ft)*	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content *(%)*	Natural Dry Density *(pcf)*	Gradation Gravel %	Gradation Sand %	Gradation Fines %	Atterberg Limits Liquid Limit	Atterberg Limits Plasticity Index	Swell/Consolidation (%) at Surcharge Pressure *(psf)*	USCS Equivalent Classification
6668	0													
		▒	**TOIPSOIL:** Approximately 2 inches of topsoil.											
		▒	**SAND:** Fine to coarse, clayey to silty sands with local gravels. They were non- to slightly plastic, loose to dense, dry to moist, and light brown to brown to gray-brown in color. Iron staining was noted locally.											
		▒		☒	5-7-11	7.1		3	67	30	22	7		SC-SM
6663	5	▒												
		▒		⧓	22/12									
		▒		☒	3-2-1									
6658	10	▨	**CLAYSTONE and SANDSTONE:** Highly interbedded claystones and fine to coarse sandstones. They were poorly to well-cemented, non- to moderately plastic, hard to very hard, slightly moist to very moist, and light brown to brown to gray to dark gray in color. Iron staining was noted commonly, lignite and secondary carbonates were noted locally.											
		▨		☒	50/6									
6653	15	▨												
		▨		☒	50/5	6.7		2	64	34	20	5		SC-SM
6648	20	▨												
		▨		☒	50/5									
6643	25	▨												
		▨		☒	14-27-48									
6638	30	▨												
		▨		☒	50/2									
6633	35	▨												
		▨		☒	50/3									
6628	40	▨												
		▨		☒	50/4									

Refusal at 42 feet.

GROUND ENGINEERING

TEST HOLE 5

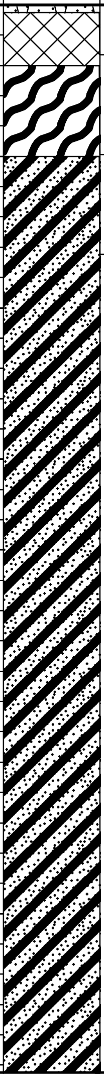
PAGE 1 OF 1

PROJECT: Austin Bluffs Tank and Pump Station

JOB NO: 25-8003a

CLIENT: Black & Veatch

SITE LOCATION: Colorado Springs, CO

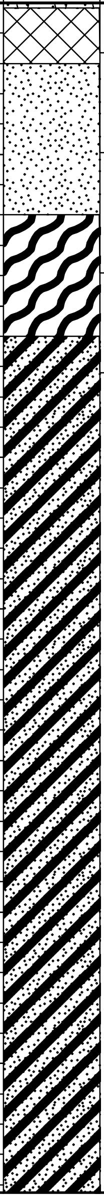


Elevation *(ft)*	Depth *(ft)*	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content *(%)*	Natural Dry Density *(pcf)*	Gradation			Atterberg Limits		Swell/Consolidation (%) at Surcharge Pressure *(psf)*	USCS Equivalent Classification
							Gravel %	Sand %	Fines %	Liquid Limit	Plasticity Index		
6664	0	**TOIPSOIL:** Approximately 2 inches of topsoil.											
		FILL: Fine to coarse, clayey to silty sands with local gravels. They were non- to slightly plastic, dry to moist, and light brown to brown to gray-brown in color.	⧓	25/12	3.5	108	4	69	27	24	7		SC-SM
			☒	16-19-21									
6659	5	**WEATHERED CLAYSTONE:** Moderately plastic, medium hard, moist, and gray in color. Secondary carbonates were noted locally.	⧓	50/10	10.7	124.9		25	75	35	18	5.7 (-)	(CL)s
		CLAYSTONE and SANDSTONE: Highly interbedded claystones and fine to coarse sandstones. They were poorly to well-cemented, non- to moderately plastic, hard to very hard, slightly moist to very moist, and light brown to brown to gray to dark gray in color. Iron staining was noted commonly, lignite and secondary carbonates were noted locally.		50/1									
6654	10		☒	50/5									
6649	15		☒	13-35-50/6									
6644	20			50/0									
6639	25		☒	50/3									
6634	30		☒	34-50/4									
6629	35		☒	50/3									

Bottom of test hole at approx. 35.25 feet.

TEST HOLE 6

PROJECT: Austin Bluffs Tank and Pump Station
JOB NO: 25-8003a
CLIENT: Black & Veatch
SITE LOCATION: Colorado Springs, CO



Elevation (ft)	Depth (ft)	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Gradation Gravel %	Gradation Sand %	Gradation Fines %	Atterberg Limits Liquid Limit	Atterberg Limits Plasticity Index	Swell/Consolidation (%) at Surcharge Pressure (psf)	USCS Equivalent Classification
6671	0		**TOIPSOIL:** Approximately 2 inches of topsoil.											
			FILL: Fine to coarse, clayey to silty sands with local gravels. They were non- to slightly plastic, dry to moist, and light brown to brown to gray-brown in color.											
6666	5			⧓	17/12									
			SAND: Fine to coarse, clayey to silty sands with local gravels. They were non- to slightly plastic, loose to dense, dry to moist, and light brown to brown to gray-brown in color. Iron staining was noted locally.											
6661	10		**WEATHERED CLAYSTONE:** Moderately plastic, medium hard, moist, and gray in color. Secondary carbonates were noted locally.	☒	14-17-20	11.7	119.2		24	76	34	17		(CL)s
			CLAYSTONE and SANDSTONE: Highly interbedded claystones and fine to coarse sandstones. They were poorly to well-cemented, non- to moderately plastic, hard to very hard, slightly moist to very moist, and light brown to brown to gray to dark gray in color. Iron staining was noted commonly, lignite and secondary carbonates were noted locally.											
6656	15			⧓	50/6	7.5	131.3	4	58	38	27	11	-1.5 (1750)	SC
6651	20			☒	13-21-30									
6646	25			⧓	50/5	8.3	115.3		40	60	26	8		s(CL)
			Auger grinding noted at 26 feet, indicating cemented bedrock.											
6641	30			⧓	50/2									
6636	35		*Vertical iron-stained fractures in driven sample at 34 feet.*	☒	35-50/4									
				⧓	50/3									

Bottom of test hole at approx. 39.25 feet.

TEST HOLE 7
PAGE 1 OF 1

PROJECT: Austin Bluffs Tank and Pump Station
JOB NO: 25-8003a

CLIENT: Black & Veatch
SITE LOCATION: Colorado Springs, CO

Elevation *(ft)*	Depth *(ft)*	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content *(%)*	Natural Dry Density *(pcf)*	Gradation: Gravel %	Gradation: Sand %	Gradation: Fines %	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	Swell/Consolidation (%) at Surcharge Pressure *(psf)*	USCS Equivalent Classification
6663	0													
			TOIPSOIL: Approximately 3 inches of topsoil.		5/12	11.4	120.7	1	62	37	23	6		SC-SM
			FILL: Fine to coarse, clayey to silty sands with local gravels. They were non- to slightly plastic, dry to moist, and light brown to brown to gray-brown in color.		30-50/6	5.8		6	76	18	NV	NP		SM
6658	5													
			CLAYSTONE and SANDSTONE: Highly interbedded claystones and fine to coarse sandstones. They were poorly to well-cemented, non- to moderately plastic, hard to very hard, slightly moist to very moist, and light brown to brown to gray to dark gray in color. Iron staining was noted commonly, lignite and secondary carbonates were noted locally.		50/6									
					48-50/5									
6653	10													
			Minor auger grinding noted at 5 feet.		50/5									
6648	15													
					50/8	8.4	131.6		44	56	27	13	0.2 (2000)	s(CL)
			Minor auger grinding noted at 18 feet.											
6643	20													
			Auger refusal at 20 feet. Wireline coring performed from 20 to 25 feet. No coring recovery, likely due to uncemented bedrock.		Core 1									
6638	25				50/2									
6633	30													
					50/4									

Refusal at 31 feet.

GROUND
ENGINEERING

Job Number: 25-8003a	Test Hole: 7	Project Name: Austin Bluffs Tank and Pump Station	Elevation: 6665	**Legend**: M - Mechanical Break S - Stepped Fracture
Logged By: CJ	Page: 1 of 1	Project Location: Colorado Springs, CO	Bedrock Depth: 2.0 Ft ±	Planarity Index: P1 - Planar P2 - Somewhat Planar P3 - Somewhat Irregular P4 - Irregular
Date Drilled: 3/25/2025	Hole Diameter: 3 1/4"	Client: BLACK & VEATCH	Total Depth: 30.33 Ft ±	Roughness Index: R1 - Smooth R2 - Fairly Smooth R3 - Fairly Rough R4 - Rough
		File Name: 6003CLs.DWG	Plunge: 90°	

DEPTH (FEET)	SAMPLE DATA		CORE DATA				GRAPHIC LOG	LITHOLOGIC DESCRIPTIONS	DRILLING CONDITIONS NOTES
					FRACTURES				
	TYPE	BLOW COUNT	PERCENT RECOVERY	RQD	DESCRIPTION	LOG			
20			0%	0%	No recovery	▩	▨	CLAYSTONE and SANDSTONE: Highly interbedded claystones and fine to coarse sandstones. They were poorly to well-cemented, non- to moderately plastic, hard to very hard, slightly moist to very moist, and light brown to brown to gray to dark gray in color. Iron staining was noted commonly, lignite and secondary carbonates were noted locally	Begin Run 1
21									
22									
23									
24									
25									End Run 1

Appendix B

Selected Core Photos

Test Hole 1 Run 1a

Test Hole 1 Run 1b

Test Hole 2 Run 1

Test Hole 2 Run 2