Geotechnical Evaluation
Meridian Ranch Field House
Falcon, Colorado
Revised

Prepared For:

Meridian Service Metro District

11886 Stapleton Drive
Falcon, Colorado 80831

Attention: Mr. Jim Nikkel

Job Number: 23-8008 **January 10, 2024**

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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) in support of the design and construction of the proposed Meridian Ranch Field House facility planned for construction north of the intersection of Rainbow Bridge Drive and Mt. Harvard Drive in Falcon, Colorado. Our study was conducted in general accordance with GROUND's Proposal Number 2309-1909R, dated September 28, 2023.

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 classification and engineering characteristics of the site soils. 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 Meridian Service Metro District, based on our understanding of the 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

Based on provided information,[1] we understand that present plans call for the construction of a 2-story field house building with an approximate footprint area of 42,200 square feet. Approximately 23,000 square feet of the field house building is planned to be a pre-engineered metal building with a synthetic turf field. We assume that the remainder of the building will utilize conventional metal or wood framing. Provided information also indicated that the anticipated total loads for isolated pads was 120 kips and 5,000 pounds per linear foot for strip footings.

In addition to the field house building, an approximately 2,400 square foot, single-story, "office" type building is planned to the north of the field house building. No below-grade levels are planned for construction. Paved parking areas, new underground utilities, site stormwater improvements, and local landscaping are also planned for the facility.

Based on the provided preliminary grading plans,[2] we understand that cuts and fills on the order of 5 feet likely will be needed to achieve project lines and grades. These plans also indicate that a finished floor elevation of 7093.50 feet is planned for the field house. The western portion of the north side and west side of the field house will retain several feet of soil. No separate retaining walls are planned at this time.

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 projects, we assume that post-construction, building foundation and floor, including the interior turf field, movements on the order of 1 inch are acceptable to, and anticipated by the owner, as are

[1] *Meridian Ranch Field House. Schematic Design.* Prepared by LKA Partners, Inc. Dated September 21, 2023.

[2] *Meridian Ranch Field House. Grading Plan.* Prepared by LKA Partners, Inc. Dated August 9, 2023. Sheets C5.0 and C5.1.

the resultant distress and maintenance measures. Similarly, we anticipate that movements of somewhat greater magnitude (1 to 2 inches) are acceptable and anticipated for flatwork, although movement estimates closer to 1 inch may be preferable near the building. Assuming that traffic speeds will be relatively low, still greater movements (3+ inches locally) are acceptable and anticipated for the parking area, as well as for flatwork that is not adjacent to the building. 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 subsurface exploration, the project site consisted a largely undeveloped lot north of the intersection of Rainbow Bridge Drive and Mt Harvard Drive in Falcon, Colorado. The site was bordered by Rainbow Bridge Drive to the west, single-family residences to the north and south, and a reach of a channelized, ephemeral drainage to east. Additional single-family residences, ongoing residential construction, parks, and undeveloped land further surrounded the site.

The project supported a variety of native grasses, weeds, and other relatively small vegetation. Gravels, cobbles, and boulders, similarly sized concrete blocks, PVC pipe, and other construction debris also were observed on the ground surface. Strom water improvements, including storm sewer outlets were observed near the eastern margin of the site. At the time of the subsurface exploration, they were discharging water in to the channelized stream. Standing water was observed locally in low spot around the site, such as the location of Test Hole P-1.

The ground surface descended to the south and east towards the channel and displayed about 20 feet of relief across the project site. About 10 feet of relief were observed across the proposed field house building footprint. Additionally, portions of the site appeared to have been previously graded.

Based on our review of available historical imagery available on Google Earth®, it appeared development of the project area began in the early 2000s with infrastructure and grading of the nearby residential development occurring in the mid to late 2000s. Construction of the residences appeared to occur in phases with adjacent phases of single-family residences being constructed in mid 2010s and appears to be continuing through present day. Development near the project site also included the channelization of the adjacent ephemeral street to the east. Significant volumes of fill appear to have been placed to construct the channelization, including fills on the project site. Other earthwork operations appeared to change the existing grades at the project site – both cuts and fills appeared to have occurred.

SUBSURFACE EXPLORATION

Subsurface exploration for the project was conducted in October 2023. A total of 15 test holes were drilled with conventional, truck-mounted drilling rig advancing 4-inch diameter solid stem continuous flight auger to evaluate the subsurface conditions and retrieve samples for laboratory testing. Of these, 9 test holes were advanced near or within the proposed building footprints to depths of about 29 to 40 feet below existing grades or to elevations of about 7066 and 7053 feet. One test hole was drilled in the proposed stormwater infiltration area to a depth of about 20 feet below existing grade or an elevation of about 7071 feet. The remaining 5 test holes were drilled within the proposed private pavement areas to depths of about 5 to 9 feet or relatively elevations of about 7093 to 7076 feet. Elevations of the test holes were estimated from the client provided grading information; some variance from actual elevations should be anticipated. A GROUND professional directed the subsurface exploration, logged the test holes 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 and a 1⅜-inch inner diameter standard penetration sampler. The samplers were driven into the substrata with blows from a 140-pound hammer falling 30 inches, in general accordance with 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.

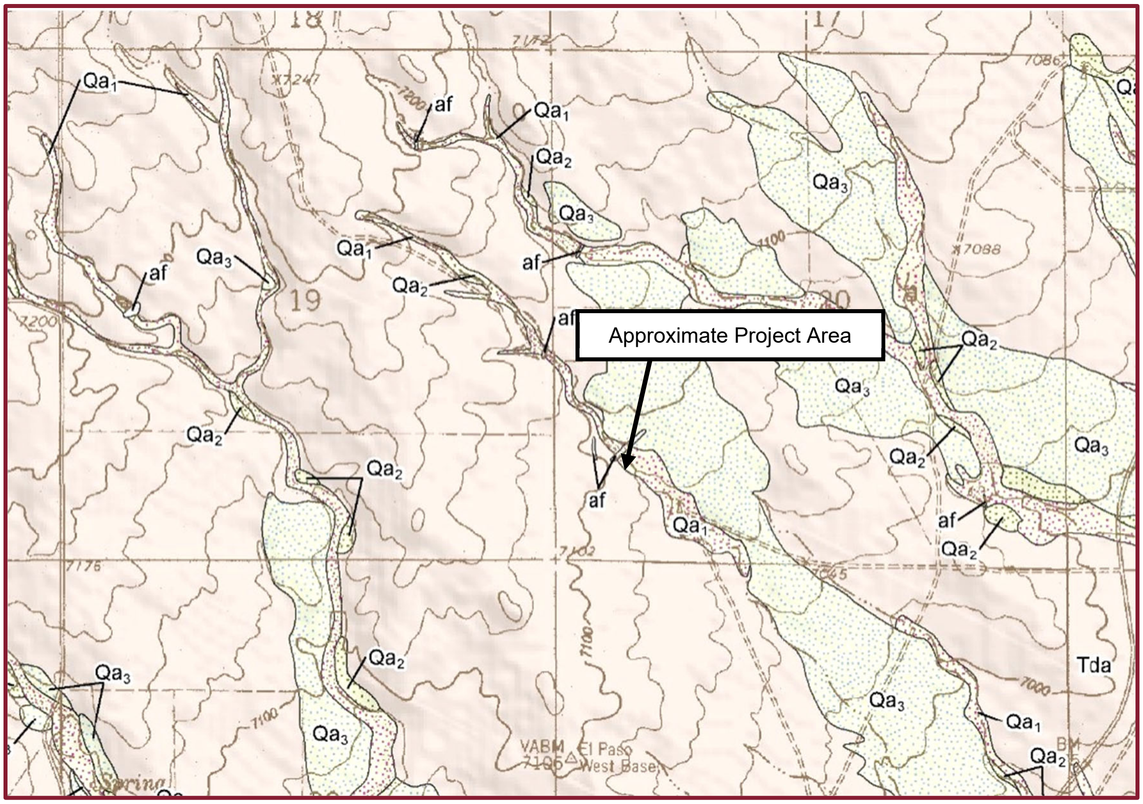
The approximate locations of the test holes are shown in Figure 1. Summary logs of the test holes are provided in Figure 2 and 3. A legend and notes are provided in Figure 4. Detailed logs of the test holes are presented in *Appendix A*.

LABORATORY TESTING

Samples retrieved from the test holes 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, water-soluble sulfate content, 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 plots are provided in Figures 5 through 11.

SUBSURFACE CONDITIONS

Regional Geology Published geologic maps such as Morgan and White (2012)[3] depict the project area as underlain by artificial fill (**af**), the Holocene Alluvium One unit (**Qa_1**), and the Paleocene to Eocene Dawson Arkose (**Tda**), which we interpreted to underlie the Artificial Fill and Alluvium One. A portion of that geologic map is reproduced below.



Artificial fill, was described by the Morgan and White geologic map as:

> Riprap, engineered fill, and refuse placed during construction of roads, railroads, buildings, dams, and landfills. Generally consists of unsorted silt, sand, clay, and rock fragments. The average thickness of the unit is less than 20 feet. Artificial fill may be subject to settlement, slumping, and erosion if not adequately compacted.

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[3] Morgan, M.L. and White, J.L. (2012) *Falcon Quadrangle Geologic Map, El Paso County, Colorado.* Colorado Geological Survey. Open-File Report OF-12-05. 1:24,000.

This description generally matches our experience in the greater project area, and we generally consider artificial fill, when it has not been properly constructed, to have potential exhibit excessive heave and/or settlement. Artificial fill can also have highly variable compositions and constancies and may include deleterious materials like trash, construction debris, organic materials, etc.

In the project area, alluvium (stream laid deposits), generally consist of fine to coarse sands and gravels with varying fraction of silts and clays. Cobbles and boulders, as well as silts and clay beds and lenses, can be present locally. Some of the coarse clasts in these deposits (cobbles and boulders) may be relatively large and difficult and/or awkward to handle. They also may not be suitable for use in some project fills.

The Dawson Arkose was described as consisting largely of fine to medium grained, clayey and silty, friable sandstone interbedded with claystone and siltstone beds. The claystones and siltstones typically are moderately to highly expansive and despite the unit being described as friable, the formation includes well-cemented beds that can be very hard and difficult to excavate.

Local Conditions In general, the test holes penetrated about 3 to 6 inches of topsoil[4] before penetrating fill materials which extended to depths of about 2 to 12 feet below existing grades or elevations of about 7076 to 7093 feet. Fill soils were encountered at the ground surface in Test Holes P-1 and P-3. Native sands were encountered below the fill that extended to depths of about 5 to 8 feet below existing grade or to elevations of about 7073 to 7088 feet. Below the native soils, sandstone bedrock was encountered and extended to the depths explored. The upper several feet of the bedrock was relatively severely weathered. Additionally, it should be noted that the native sand layer was not recognized in Test Holes 4, 6 through 9, and IP-1, and bedrock materials were encountered immediately below the fill.

We interpret the fill materials to have been placed during the original development of the area. We interpret the native sands to be alluvial deposits. The underlying sandstone bedrock is interpreted to be Dawson Arkose deposits.

[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 plants that may be proposed for the project.

Fill materials were recognized at the test holes and are likely present throughout the site. (See the *Site Conditions* section of this report.) These fill soils may contain coarse gravels and cobbles, as well as similarly sized pieces of construction debris even where not recognized in the test holes. 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.

Similarly, coarse gravel and larger clasts are not well represented in small diameter liner samples collected from the test holes. Therefore, such materials may be present even where not called out in the material descriptions herein.

Fill consisted of clayey and silty, fine to coarse sands and clays with gravels. They were moist to very moist, moderately plastic, very stiff to hard or medium dense to dense, and brown to dark brown to black in color.

Sands consisted of clean to clayey and silty, fine to coarse sands with gravels. Silt and clay beds and lenses were noted locally. They were very moist to wet, non- to slightly plastic, medium dense to dense, and brown in color.

Weathered Sandstone consisted primarily of weathered, silty and clayey, fine to coarse sandstone with local weathered claystone. They were dry to moist, slightly to moderately plastic, weathered to firm, and gray to pale gray brown in color. Iron staining was noted locally.

Sandstone consisted primarily of silty and clayey, fine to coarse sandstone with local claystone. They were dry to wet, non- to moderately plastic, hard to very hard, and gray to gray-brown in color. Iron staining was noted locally.

Groundwater was encountered in the test holes at the time of drilling at depths between about 16 and 27 feet below existing grade corresponding to elevations of about 7068 to 7076 feet. When checked in Test Holes 2 and 7 at the end of drilling operations, groundwater was measured to be at depth of about 16 feet in each test hole. This depth corresponded to an elevation of about 7079 and 7078 feet respectively.

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, 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 that Meridian Service Metro District desires additional/repeated groundwater level observations, GROUND should be contacted to provide a cost estimate for this additional geotechnical evaluation.

Swell-Consolidation Testing of selected samples of site soils recovered from the test holes indicated a swell of about 1.7 percent in a sample of weathered claystone and consolidations of up to approximately 0.8 percent in the other samples of fill and native materials selected for testing. These selected samples were tested against surcharge loads approximating in-place overburden pressures. (See Table 1.)

ENGINEERING SEISMICITY

Based on extrapolation of available data to depth and our experience in the project area, we consider the site (approximate building addition footprint) likely to meet the criteria for a Seismic Site Classification of **D** 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 were 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. GROUND can provide a fee estimate for shear wave velocity testing upon request. We consider the likelihood of achieving a Site Class C to be relatively moderate to low.

Using longitude and latitude coordinates obtained from Google Earth and the ASCE Hazard Tool (https://asce7hazardtool.online) the project area is indicated to possess an S_{DS} value of **0.198** and an S_{D1} value of **0.088** for the site latitude and longitude and a Site Class of D.

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 Meridian Service Metro District, 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, Meridian Service Metro District, also must understand the risk-cost trade-offs addressed by the civil and structural engineering disciplines in order to direct his design team to the portion of the Higher Cost/Lower Risk–Lower Cost/Higher Risk spectrum in which this project should be designed. If Meridian Service Metro District, does not understand these risks, it is critical that additional information or clarification be requested so that their expectations reasonably can be met.

General Geotechnical Risk In GROUND's opinion the primary source of geotechnical risk at the site was the presence of the existing fill soils. Undocumented, fill soils must be treated as unsuitable to support new construction. However, GROUND was provided with field testing data and a summary letter[5] by CTL | THOMPSON, Incorporated (CTL) for overlot grading of the area that, we understand, included the location of the proposed field house. The summary letter indicated that CTL's, "Compaction test results indicate densities and moistures in general conformance with the requirements of our

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[5] CTL | THOMPSON, Inc., 2007, *Overlot Grading, Meridian Ranch, Filing No. 3, Londonderry Drive and Rainbow Bridge Drive, El Paso County, Colorado*, Project No. CS15869-310, prepared for Tech Contractors, dated June 29.

specifications." Therefore, based on the CTL summary letter, we consider the fill soils encountered in the test holes to be properly controlled and compacted fill. However, because of the passage of time with resultant moisture changes, freezing and thawing, plant growth, etc., can disturb compacted fills, we consider the top 3 feet of fill soils across the site to be undocumented fills that should be not be relied upon to support new construction.

Additionally, constructed fills can contain deleterious materials that may not be suitable for inclusion in improvement supporting fills and such materials may require special handling considerations.[6]

Likely Post-construction Movements Based on our data, and our experience with similar sites, we estimate improvements supported directly on the existing site soils are subject to likely, post-construction, vertical movements of 1 to 1½ inches where improvements bear directly on the existing undocumented fill soils. Lateral movements will result, as well. Foundation, slab/flatwork, and pavement movements of these magnitudes can result in significant damage. Nearly all of the proposed improvements are vulnerable in this regard.

The underlying native soils and sandstone bedrock appear to be able to provide sufficient support for proposed construction without excessive settlements. However, it should be noted that the Dawson Arkose is known to contain highly expansive claystones and siltstones. Such expansive materials were not identified in the test holes or by our laboratory testing. Regardless, due to the limitation of geotechnical subsurface exploration, there may exist expansive bedrock materials may be present beneath the proposed field house footprint. We consider this risk to be relatively low, but it is not zero. Therefore, it may be useful to perform additional geotechnical analysis if significant volumes of claystone and siltstone are encountered in project excavations. Retaining a geotechnical engineer to observed project excavations may be beneficial in this regard.

Building Foundation and Floor Types Several foundation systems and floor systems appear to be geotechnically feasible at this site and are discussed below. Each system discussed below offer different cost-risk trade-offs. Selecting the appropriate system for

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[6] Note that an environmental assessment of the site and site materials was not included in our scope of services. If an environmental assessment is needed, an environmental consultant should be retained.

the project will require the careful consideration of project performance goals, budget, schedule, and other factors affecting the project.

Deep Foundation/Structural Floor System In GROUND's opinion, supporting the proposed buildings on drilled pier foundation system will provide the lowest estimates of likely post-construction foundation movement (about ½ inch, with similar differential movements over spans of about 40 feet) and will provide the least risk of excessive foundation movements. However, deep foundations systems may not be practical because they may not be required to carry the structural loads.

As an alternative to drilled piers, helical pier foundation systems, or other similar, proprietary systems, such as micropiles could be used. These systems likely will yield similar performance to drilled piers and we anticipate that they may be more economical. These proprietary systems are designed and installed by a specialty contractor. We anticipate that this report will be provide sufficient information for a specialty foundation contractor to complete their design. Note that multiple elements may be needed to resist lateral loads.

We suggest contacting the following firms, though others may be available:

- **D&B Engineering Contractors** (Wheat Ridge, Colorado) 303–423–6834 for helical piers.
- **Keller** (Commerce City, Colorado) 303–469–1136 for helical piers, micropiles and other systems.
- **Park Range Construction** (Englewood, Colorado) 303–395–3234 for helical piles and other systems.
- **Schnabel Foundation Company** (Aurora, Colorado) 303–696–7268 for micropiles and other systems.

Constructing the lowest level building floors as a structural floor, also supported on drilled piers, will yield similarly low post-construction floor movement estimates. Exterior flatwork adjacent to the building, particularly at and near building entrances, also should be constructed as structural floors in such cases. Geotechnical parameters for structural floors, however, can be provided upon request.

Shallow Foundation and Slab-on-Grade Floor System—Fill Prism As a higher risk, although commonly used alternative, shallow foundations and a slab-on-grade floors could be used for the field house and office building, provided they bear on a remedial fill section constructed in general accordance with this report. In order to reduce estimates of post-construction movements to about 1 inch, the footings and floor slab/underslab gravel layer should be constructed on a remedial fill section which extends to a depth of at least **3 feet** below existing grade – the depth of fill soils taken to be "undocumented" for this report. Beneath field house and beneath the other, office building, a fill section of uniform depth should be constructed, based on the lowest, existing grade within each building footprint. Similarly, where compacted fill soils underlie any of a buildings footings, the same fill section should be constructed under all footings of the building.

Alternatively, where 3 feet of remedial earthwork would result, nominally, in some footings bearing on fill and others of the same building bearing on native soils or the existing, compacted fill, the footings that would bear on newly placed fill may be deepened so that they also bear on the native soils or the existing, compacted fill.

Differential movements for improvements bearing on such a fill section likely would be on the order of ½ inch over spans of about 40 feet. Additional parameters for the design and construction for shallow foundations and slab-on-grade floors are provided in the *Shallow Foundations* and *Slab-on-Grade Floor* sections of this report.

The fill section beneath a building should be laterally consistent and of uniform depth to reduce differential, post-construction foundation movements. A differential fill section will tend to increase differential movements. In general, we anticipate that the majority of the existing site soils, free of deleterious materials, will be suitable geotechnically to be reused as fill. Inert construction debris with a maximum dimension of 3 inches may be incorporated into the fill. Additional information in this regard is provided in the *Project Earthwork* section of this report.

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.

Drilled Pier Foundations Based on the results of the field exploration, laboratory testing, and experience, the design criteria presented below should be observed for a straight-shaft, drilled pier foundation system. In our experience it can be beneficial to facilitate construction to use of as few pier diameters/types as possible.

Note that the minimum dead load and the minimum pier length indicated below were developed to resist the uplift force that the expansive soil / bedrock will exert on the surface of the pier in the zone above the depth of wetting. The minimum length on that basis may or may not be sufficient to provide the necessary axial capacity. The uplift loading also should be used to develop the (minimum) reinforcing steel, as discussed below.

1) Drilled piers should bear in "relatively unweathered" bedrock underlying the site. Relatively unweathered bedrock should be taken to be <u>only</u> below **an elevation of 7,080 feet beneath the field house**, and only below an elevation of **7,085 feet beneath the office building**.

   Note that weathered claystone and firmer materials likely will be encountered at shallower depths. Bedrock penetration, however, should not be counted in any material shallower than the elevations indicated above.

2) Drilled piers should be at least **18 inches** in diameter to meet geotechnical criteria.

   The pier length / diameter ratio will be determined by the structural engineer. Useful guidance in this regard may be found in appropriate AASHTO or FHWA manuals or other publications. In our experience, drilled pier diameters are rarely modified in the field. However, for various factors as discussed herein, pier lengths commonly must be increased in the field beyond the nominal design lengths.

Actual depths to bedrock will vary. Therefore, it may be beneficial to select project pier diameter(s) based on potential pier length increases so that the minimum length / diameter ratio included in the design is not exceeded.

3) Drilled piers should have a minimum length of **26 feet** based on geotechnical considerations. The actual drilled pier lengths should be determined by the structural engineer based on loading, etc., with further increases in length possibly required by the conditions encountered during installation at each drilled pier location.

4) Drilled piers also should penetrate at least **6 feet** into relatively unweathered bedrock, or **3 drilled pier diameters**, whichever is greater. (Given the relatively shallow bedrock, the minimum length of 26 feet likely will control the pier lengths, not bedrock penetration.)

   Based on the minimum length and bedrock penetration, drilled pier lengths of 26 to 29 feet are anticipated to meet the geotechnical criteria. <u>Actual drilled pier lengths commonly will be greater due to structural considerations, conditions in the drilled pier holes, actual depths to bedrock, etc.</u>

5) Drilled piers bearing in relatively un-weathered bedrock may be designed for an allowable end bearing pressure of **30,000 psf**.

   The portion of the drilled pier penetrating relatively un-weathered bedrock below the depth of wetting may be designed for a skin friction value of **2,250 psf**. 100 percent of the skin friction may be used to resist both compressional loads and uplift. However, skin friction shallower than 20 feet should be ignored for axial load resistance.

6) Drilled piers should be designed for a minimum dead load pressure of **4,000 psf** based on drilled pier cross-section area.

   Where the minimum dead load cannot be applied, it will be necessary to increase the drilled pier **length** beyond the provided minimum, even where the minimum bedrock penetration has been achieved or exceeded. This can be accomplished by assuming that skin friction on the extended zone acts to resist uplift.

7) Drilled piers should be reinforced as determined by the structural engineer. At a minimum, each drilled pier should be reinforced for its full length to resist the tensile loading created by the uplift force exerted on the pier by the swelling soils and bedrock, and the deficit between the actual dead load applied to a pier and the indicated minimum dead load. This uplift load on a pier may be estimated using an uplift skin friction of **700 psf** acting on the surface area above a depth of 20 feet.

8) A **4-inch** or thicker continuous void should be provided beneath grade beams, drilled pier caps, and foundation walls to limit the potential of swelling soil and bedrock from exerting uplift forces on these elements and to concentrate drilled pier loadings. The void space should be protected from backfill intrusion.

9) Geotechnical parameters for resisting lateral loading of drilled piers are provided in the *Lateral Loads* section of this report.

10) Penetration of relatively unweathered bedrock in drilled pier shafts should be roughened artificially to assist the development of peripheral shear between the drilled pier and bedrock. Artificially roughening of drilled pier holes should consist of installing shear rings **3 inches high** and **2 inches deep** in the portion of each drilled pier **below 20 feet**. The shear rings should be installed **18 inches on centers**.

However, the specifications should allow a geotechnical engineer to waive the requirement for shear rings depending on the conditions actually encountered in individual drilled pier holes, if in his opinion, the installation of the shear rings will be detrimental to foundation performance.

11) Groups of closely spaced drilled piers will require an appropriate reduction of the estimated capacities. Reduction of axial capacity generally can be avoided by spacing drilled piers **at least 3 diameters center to center**. At this spacing or greater, no reduction in axial capacities or horizontal soil modulus values is required. The capacities of drilled piers spaced more closely than 3 diameters center to center should be reduced. Reduction factors can be obtained from the plot provided in Figure 12.

12) Linear arrays of drilled piers, however, must be spaced **at least 8 diameters** center to center to avoid reductions in lateral capacity when loaded in line with the array (parallel to the line connecting the drilled pier centers). The lateral capacities of piers in linear arrays spaced more closely than 8 diameters should be reduced. Reduction factors can be obtained from the plot provided in Figure 13.

Drilled Pier Construction The following should be considered during the construction of a drilled pier foundation.

13) A preconstruction meeting should be held prior to drilled pier construction to discuss the necessary lengths and bedrock embedment of the drilled piers, possible variations in pier length across the site, and potential constructability issues. The firm providing drilled pier observations as well as the contractor's responsible person must attend.

14) Drilled pier excavation may generate cuttings that contain debris and require export. The contractor should be prepared to handle such materials. Additional information regarding exporting of fill materials is provided below in the *Project Earthworks* section below.

15) The depth of relatively unweathered bedrock should be determined in the field at each drilled pier location and may differ from other information provided herein. <u>However, at no pier location should the top of the relatively unweathered bedrock be taken to be shallower the elevations indicated in item 1) above</u> .

16) Lenses of relatively soft bedrock may not be suitable for foundation support were encountered within the relatively unweathered bedrock section. Where such layers are encountered, the drilled pier should be lengthened to bear on firm bedrock (i.e., not a relatively soft layer).

17) Because of potential for individual pier holes to require deepening beyond the planned minimum lengths, it may be cost effective for the contractor not to cut the reinforcing steel prior to drilling.

18) The bedrock beneath the site was hard to very hard, with penetration resistance values as high as 50 blows for 3 inches commonly, and locally as high as 50 blows

for 0, i.e., no penetration. Local lenses of still more highly cemented sandstone have been encountered in the project area. The contractor should be prepared to core isolated lenses of highly cemented materials. The pier-drilling contractor should mobilize equipment of sufficient size and operating capability to achieve the design lengths and bedrock penetration.

If refusal is encountered in these materials, a geotechnical engineer should be retained to evaluate the conditions to establish whether true refusal has been met with adequate drilling equipment.

19) Encountering groundwater in the drilled pier holes should be anticipated near or below about Elevation 7,080 feet, and where groundwater is encountered, casing may be required in the drilled pier holes to reduce water infiltration and to reduce the impact of caving conditions. In the event that casing is seated into the bedrock, the minimum bedrock penetration should be taken from the bottom of the casing.

Seating of the casing in the upper layers of the bedrock may not create positive cutoff of water infiltration. The contractor should be prepared to address this condition.

20) In no case should concrete be placed in more than **3 inches** of water, unless placed through an approved tremie method. The proposed concrete placement method should be discussed during the pre-construction meeting by the project team.

21) Where groundwater and unconsolidated soils are encountered, the installation procedure of drilled piers can be a concern. Commonly in these conditions, the drilling contractor utilizes casing and slurry during excavation of the drilled pier holes, which may adversely affect the axial and/or lateral capacities of the completed drilled piers. During casing withdrawal, the concrete should have sufficient slump and must be maintained with sufficient head above groundwater levels to displace the water or slurry fully to prevent the creation of voids in the drilled pier.

Because of these considerations, the drilling contractor should submit a written procedure addressing the use of casing, slurry, and concrete placement prior to commencement of drilled pier installation.

22) Drilled pier holes should be cleaned properly prior to placement of concrete.

23) Concrete utilized in the drilled piers should be a fluid mix with sufficient slump so that it will fill the void between reinforcing steel and the drilled pier hole wall, and inhibit soil, water, and slurry from contaminating the concrete. The concrete should be designed with a minimum slump of no less than **5 inches**.

24) Concrete should be placed by an approved method to minimize mix segregation.

25) Concrete should be placed in a drilled pier on the same day that it is drilled. Failure to place concrete the day of drilling may result in a requirement for lengthening the drilled pier. The presence of groundwater or caving soils may require that concrete be placed immediately after the pier hole drilling is completed.

26) The contractor should take care to prevent enlargement of the excavation at the tops of drilled piers, which could result in “mushrooming” of the drilled pier top. Mushrooming of drilled pier tops can increase uplift pressures on the drilled piers.

27) Sonic integrity testing (sonic echo or cross-hole sonic) should be performed for an appropriate percentage (e.g., 10 percent) of the drilled piers to assess the effectiveness of the drilled pier construction methods. Additional information on sonic integrity testing can be provided upon request.

Shallow Foundations

Geotechnical Parameters for Shallow Foundation Design

1) Footings should bear on properly compacted fill or on firm native soils as discussed in the *Geotechnical Considerations for Design* section of this report.

The fill section beneath the building addition should extend at full thickness across the building addition footprint and at least **10 feet** laterally beyond the footing margins.

The contractor should provide survey data of the excavation beneath the building addition indicating the depth and lateral extents of the remedial excavation.

2) Footings bearing on firm native soils or on compacted fill may be designed for an allowable soil bearing pressure of **2,500 psf** for footings up to **8 feet in width**.

This value may be increased by ⅓ for transient loads such as wind or seismic loading. For larger footings, 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 ¾ inch, based on an assumption of drained foundation conditions. 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 herein may result.

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 building at this site. (See the *Geotechnical Considerations for Design Section* of this report.) It is in addition to movements from post-construction volume change in the native soils underlying the site and from densification of the fill section constructed beneath the building, as discussed above.

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

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

4) 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.

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

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

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

8) To the extent possible, utility lines should not be routed under shallow foundations, particularly isolated pad foundations, nor in the soils supporting the 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

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

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

11) 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 properly compacted fill or the foundation deepened.

12) Foundation-supporting soils 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.

13) All foundation subgrade should be compacted prior to placement of concrete.

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

FLOOR SYSTEMS

The floor system 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 floor system 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 risks and post-construction movement estimates are acceptable for the project.

Structural Floors A structural floor should be utilized for either or both of the proposed structure as the floor system resulting in the least potential for post-construction floor movement, including differential movement. Entryway floor slabs and other exterior flatwork also should be constructed as structural floors where differential movements between the building and exterior flatwork is a concern.

A structural floor should be supported on grade beams and straight-shaft drilled piers in the same manner as the building structure. Requirements for the number and position of additional piers to support the floors will depend upon the span, design load, and structural design, and should be developed by the structural engineer. Geotechnical information for design and installation of drilled piers are provided above. Additionally, it should be noted that structural floors tend to be flexible and move elastically under live loads. Building design also should account for floor movements of this type.

A structural floor should be constructed to spanning above a well-ventilated crawl space. The crawl space should be adequately sized to allow access to and maintenance of utility piping if placed beneath the floor. Piping connections through floors, grade beams, or foundation walls should allow for differential movement between the piping and the floor.

Alternatively, a structural floor may be constructed over an **4-inch** or thicker continuous void form; however, in our experience this entails a higher risk relative to constructing the floor over a crawl space, related primarily to effective construction and void collapse.

Where a void form is used, appropriate protections should be provided to prevent soil from intruding laterally into the void.

A vapor barrier, water proofing, other means of preventing moisture intrusion should be utilized where appropriate.

All utility lines entering the building should be carefully tested before operation. Where lines enter through the floor, positive bond breaks should be provided. Lines can be displaced by soils and bedrock movements, which are not reflected in the building. Design and installation of associated fixtures should accommodate this potential differential movement.

Slab-on-Grade Floors The geotechnical parameters below may be used for design of slab-on-grade floors for the proposed building and interior turf field. ACI Sections 301/302/360 provide guidance regarding concrete slab-on-grade design and construction.

Geotechnical Parameters for Design of Slab-on-Grade Floors

1) A slab-on-grade floor system should bear on a section of properly compacted fill soils as discussed in the *Geotechnical Considerations for Design* section of this report. (If the sub-slab gravel section is omitted, the depth of fill beneath the floor slab should be thickened correspondingly.)

   A fill section constructed beneath a slab-on-grade floor should extend at full thickness across the building footprint and at least **3 feet** laterally beyond the building perimeter. Any flatwork intended to perform in the same manner as the building floor should be prepared in the same manor.

The fill section beneath the building should be laterally consistent and of uniform depth to reduce differential, post-construction floor movements. A differential fill section will tend to increase differential movements. Criteria for fill placement and compaction are provided in the *Project Earthwork* section of this report.

The contractor should provide survey data of the excavation beneath each building indicating the depth and lateral extents of the remedial excavation.

2) Floor slabs 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 **70 tcf** (81 pci) may be used for design of a concrete, slab-on-grade floor bearing on a remedial fill section.

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

4) Floor slabs 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) Concrete slabs-on-grade 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 floor slabs 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 slab-on-grade floors and utility lines in the soils beneath them 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 sub-slab 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, **more than 50 percent** retained on the No. 4 Sieve, and a maximum particle size of **2 inches**.

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.

We understand, however, that professional experience and opinion differ with regard to inclusion of a free-draining gravel layer beneath slab-on-grade floors. If these issues are understood by the owner and appropriate measures are implemented to address potential concerns including slab curling and moisture fluxes, then the gravel layer may be deleted.

9) A vapor barrier beneath a building floor slab is beneficial with regard to reducing sub-slab moisture vapor transmission through the floor slab and into the building, but can retard downward drainage of construction moisture. Elevated vapor fluxes can be detrimental to the adhesion and performance of many floor coverings and can also contribute to other moisture-induced concerns. Thus, an effective sub-slab vapor barrier is a published industry requirement for most slab-on-ground construction (i.e., IBC, ASTM), regardless of project location, soil conditions, and water table depth.

Per ACI 302.2R-15, a vapor barrier should be placed 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. Moreover, ACI recommends a vapor barrier for any building which will be humidity or climate controlled, including exposed slabs (such as industrial warehouse). ACI 302 provides further guidance on the location of the vapor barrier beneath the slab.

However, if a slab were cast directly on the vapor barrier, considerations and steps may be needed to help reduce uneven drying/shrinkage concerns and potential slab curling.

Therefore, the owner, the architect, and/or contractor should weigh many considerations when designing and implementing the sub-slab vapor barrier system, including building use and operating conditions, flooring products, sub-base (gravel layer) type, size, and thickness, expected construction traffic, etc.

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. (Note that Polyethylene ("poly") sheeting (even if 15 mils in thickness which polyethylene sheeting commonly is not) does not meet the ASTM E1745 criteria and generally should not be used as a vapor barrier material.)

Construction Considerations for Slab-on-Grade Floors

10) Loose, soft, or otherwise unsuitable materials exposed on the prepared surface on which the floor slab will be cast should be excavated and replaced with properly compacted fill.

11) The fill section beneath a slab should be of uniform thickness. The use of rammed aggregate piers may alter this requirement.

12) Concrete floor slabs should be constructed and cured in accordance with applicable industry standards and slab design specifications.

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

MECHANICAL ROOMS, MECHANICAL PADS, AND TRASH ENCLOSURES

Often, slab-bearing mechanical rooms/mechanical equipment/trash enclosures are incorporated into projects that are attached to, or adjacent to the primary structure. Commonly, they also are partially below grade. These elements should be founded on the same type of foundation systems as the main structure. Furthermore, mechanical connections must allow for potential differential movements.

BUILDING RETAINING WALLS/FOUNDATION WALLS

We understand that the west wall and a portion of the north wall of the field house will retain several feet of soil. The following parameters and considerations should be used for the below-grade walls. Groundwater considerations are provided in the *Subsurface Drainage* section of this report.

Wall Design Parameters Equivalent fluid pressures for use in design of retaining walls are provided in the *Lateral Earth Pressures* section of this report. The data on those tables should be evaluated by an experienced engineer for use in wall design calculations.

Parameters for backfill placement and compaction are provided in the *Project Earthwork* section of this report.

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, such as mechanical pads as discussed in the next section, 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

Deep Foundations Resisting Lateral Loads Based on the data obtained for this study and our experience with similar sites and conditions, lateral load analysis using the "L-Pile" or a similar computer program should use the following geotechnical parameters for input. The parameters are based on a simplified soil / bedrock profile. These include, moist unit weights (γ'), angle of internal friction (ϕ) and horizonal soil modulus (**k**), for the earth materials. Resistance to lateral loads should be neglected in the upper **3 foot** of soils, whether fill or native.

GEOTECHNICAL PARAMETERS FOR LATERAL LOAD ANALYSIS USING L-PILE

Soil / Bedrock Material	Approximate Depth Range	Parameter	Value
Existing Fill and Native Soils *(model as Sand without Free Water)*	3 feet below FFE to Elevation 7,077 feet	γ'	130 pcf (0.075 pci)
		ϕ	28 degrees
		k	0.216×10^6 pcf (125 pci)
Sandstone Bedrock (*model as Sand with Free Water*)	Elevation < 7,077 feet	γ'	63 pcf (0.036 pci)
		ϕ	25 degrees
		k	0.259×10^6 pcf (150 pci)

Values for equivalent fluid pressures for the drained condition and the coefficient for frictional resistance to sliding are provided below. These values were based on moist unit weight (γ') of 130 pcf and an angle of internal friction (ϕ) of 28 degrees for site soils reworked as properly compacted fill and are unfactored. Appropriate factors of safety should be included in design calculations

Shallow Elements Resisting Lateral Loads A friction coefficient of **0.35** between a foundation element and the site soils may be used for design of shallow foundations and thrust blocks resisting lateral loads.

Passive soil pressure at this site may be estimated using an equivalent fluid pressure of **320 pcf** for drained conditions, to a **maximum of 3,200 psf**.

The upper 1 foot of embedment should be neglected for passive resistance, however. Where 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. A reduced passive pressure can be used for reduced anticipated strains, however.

At-Rest and Active Lateral Earth Pressures Site soils placed as backfill against a structure in an **at-rest** condition may be considered to exert an equivalent fluid unit weight of **69 pcf** for the drained condition.

Site soils placed as backfill where the full, **active** earth pressure condition applies may be considered to exert an equivalent fluid unit weight of **47 pcf** for the drained condition may be used.

WATER-SOLUBLE SULFATES

The concentration of water-soluble sulfates measured in selected samples of site soils during this evaluation indicated values of approximately 0.03 percent by weight. (See Table 2.) Such concentrations of soluble sulfates represent 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 should be used in all concrete exposed to site soils:

<u>Class 0 (Negligible)</u>

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 of approximately -6 and -26 millivolts. Such low potentials typically create a more corrosive environment.

Sulfide Reactivity testing indicated "positive" results in the local soils. The presence of sulfides in the soils suggests a more corrosive environment.

Soil Resistivity In order to assess the "worst case" for mitigation planning, samples of materials retrieved from the 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. Measurement of electrical resistivity indicated values of approximately 1,812 and 2,707 ohm-centimeters in 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] Our testing indicated pH values of about 7.0 and 7.6.

[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 (16½ 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. <u>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.</u>

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. Collecting survey data of fill areas after initial earthwork is complete can be beneficial for monitoring the progress of fill settlements.

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 present on site should not be incorporated into ordinary fills. Instead, it 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 present throughout the site. Because not all site 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 approved prior to reuse on the site.

Use of Existing Native Soils Based on the samples retrieved from the test holes, we anticipate that the existing site soils that are free of organic materials, coarse cobbles, boulders, or other deleterious materials will be suitable, in general, for reuse as compacted fill.

Fragments of rock and cobbles, (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 **20 percent or less** passing the No. 200 Sieve and a plasticity index of **10 or less**. Materials proposed for import should be approved prior to transport 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 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.

Wet, Soft, or Unstable Subgrades We anticipate that wet, soft, or unstable subgrades could be encountered in project excavations. Where wet, soft, or unstable subgrades are encountered, the contractor should establish a stable platform for fill placement and achieving compaction in the overlying fill soils. Therefore, excavation of the unstable soils and replacing them with relatively dry or granular material, possibly together with the use of stabilization geo-textile or geo-grid, may be necessary to achieve stability. Whereas the stabilization approach should be determined by the contractor, GROUND offers the alternatives below for consideration. Proof-rolling can be beneficial for identifying unstable areas.

- Replacement of the existing subgrade soils with clean, coarse, aggregate (e.g., crushed rock) or road base. Excavation and replacement to a depth of 1 to 2 feet commonly is sufficient, but greater depths may be necessary to establish a stable surface.

  On very weak subgrades, an 18- to 24-inch “pioneer” lift that is not well compacted may be beneficial to stabilize the subgrade. Where this approach is employed, however, additional settlements of up to ½ inch may result.

- Where coarse, aggregate alone does not appear sufficient to provide stable conditions, it can be beneficial to place a layer of stabilization geotextile or geogrid (e.g., Tencate Mirafi® RS 580*i*, or Tensar® BX 1100) at the base of the aggregate section.

  The stabilization geotextile or geogrid should be selected based on the aggregate proposed for use. It should be placed and lapped in accordance with the manufacturer’s recommendations.

  Geotextile or geogrid products can be disturbed by the wheels or tracks of construction vehicles. We suggest that appropriate care be taken to maintain the effectiveness of the system. Placement of a layer of aggregate over the geo-textile / geo-grid prior to allowing vehicle traffic over it can be beneficial in this regard.

When a given remedial approach has been selected, the contractor should construct a test section to evaluate the effectiveness of the approach prior to use over a larger area.

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.

Excavated bedrock materials, including those present in the existing fill, will require a well-coordinated effort to moisture treat, process, place, and compact properly. In-place bedrock deposits were hard to very hard, and should be broken down in to a soil-like mass. Greater than typical watering, and compaction equipment that aids in breaking down such material (e.g., a Caterpillar 825 compactor-roller), likely will be needed. Crushing or other methods should be anticipated to sufficiently reduce well cemented sandstone bedrock where encountered. 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 Soils that classify as **GP, GW, GM, GC, SP, SW, SM, or SC** in accordance with the USCS classification system (granular materials) 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 job-sites.

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 and in addition to settlement of any existing soils left in place. For a 12-foot fill, for example, that would correspond to a total settlement of about 2 inches. 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. 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.

Cut and Filled Slopes Permanent (final grading), unretained, graded slopes supported by local soils up to **15 feet** in height should be constructed no steeper than **3 : 1** (horizontal : vertical). Minor raveling or surficial sloughing should be anticipated on slopes cut at this angle until vegetation is well reestablished. Given the sandy nature of the site soils, the risk of these features developing appears to be somewhat greater than other, more clayey sites. 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. Caving conditions may be encountered where excavations extend into poorly cemented sandstones, particularly below the water table. Additionally, excavations into bedrock may encounter very hard and resistant beds and lenses. Materials much harder than those identified in the test holes could be encountered locally, as the Dawson Akrose is known to include well cemented beds and lenses that can require greater than typical efforts to excavate, handle, and process. Specialized breaking equipment may be needed and greater than typical equipment wear may result.

Undocumented fill soils were also identified at the site. Given the inherent nature of undocumented fill soils and demolition fill, materials that may be awkward or otherwise difficult to handle (e.g., relatively large pieces of construction debris) may be encountered. Relatively large pieces of construction debris, including concrete, plastic, and metal, as well as relatively large cobbles and boulders were identified on the ground surface. Therefore, we anticipate that the likelihood of encountered similar materials at depth with in the fill is greater than on other similar sites. The contractor and the project team should be prepared to handle such materials.

Groundwater Groundwater was encountered in test holes at the time of drilling at depths between about 16 and 27 feet below existing grade corresponding to elevations of about 7068 to 7076 feet. When checked in Test Holes 2 and 7 at the end of drilling operations, groundwater was measured to be at depth of about 16 feet in each test hole. This depth corresponded to an elevation of about 7079 and 7078 feet respectively.

Therefore, we anticipate that project excavations shallower than a relative elevation of 7083 feet will be unlikely to encounter shallow groundwater except for limited volumes of perched groundwater. Dewatering measures could be necessary where project excavations approach the existing channel at the site, though.

Should seepage or flowing groundwater be encountered in project excavations, the slopes should be flattened as necessary to maintain stability or a geotechnical engineer should be retained to evaluate the conditions. The risk of slope instability will be significantly increased in areas of seepage along excavation slopes.

Groundwater also should be anticipated at similar elevations in drilled pier holes.

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.

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 Meridian Service Metro District, 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.

Temporary, unshored excavation slopes up to **10 feet** in height, in general, should be cut no steeper than **2 : 1** (horizontal : vertical) in the on-site soils in the absence of seepage. Some surface sloughing may occur on the slope faces at these angles. Should site constraints prohibit the use of the above-indicated slope angle, temporary shoring should be used. GROUND is available to provide shoring design upon request. Stockpiling of materials should not be permitted closer to the tops of temporary slopes than 5 feet or a distance equal to the depth of the excavation, whichever is greater. Additionally, shallow granular soils should be cleared back from the tops of slopes.

UTILITY LATERAL 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.

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, the entire depth of the trench (both bedding and common backfill zones) should be backfilled 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 In areas that area tolerant of surface settlements, conventional soil backfilling may be used. 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 Meridian Service Metro District.

We anticipate that the 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 a granular bedding material is specified, with regard to potential migration of fines into the 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, and we don't anticipate that it will, 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 cut-off 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.

If granular bedding is specified, the contractor should not anticipate that significant volumes of shallow site soils will be suitable for that use. Materials proposed for use as pipe bedding should be tested for suitability prior to use.

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 structure, 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 foundation or floor elevations. Therefore, local saturation of project foundation 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. <u>Maintenance should be anticipated to include removal and replacement of sidewalk stones, curb and gutter, sections of pavement, etc., to restore effective drainage</u>. 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 underslab 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) Measures for positive surface drainage away from the building 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 sidewalks and other hardscaping as well as utility 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.

4) In GROUND's experience, it is common during construction that in areas of partially completed paving or hardscaping, bare soil behind curbs and gutters, and utility trenches, water is allowed to pond after rain or snow-melt events. Wetting of the subgrade can result in loss of subgrade support and increased settlements. By the time final grading has been completed, significant volumes of water can already have entered the subgrade, leading to subsequent distress and failures. The contractor should maintain effective site drainage throughout construction so that water is directed into appropriate drainage structures.

5) In no case should water be permitted to pond adjacent to or on sidewalks, hardscaping, or other improvements as well as utility trench alignments, which are likely to be adversely affected by moisture-volume changes in the underlying soils or flow of infiltrating water.

6) Roof downspouts and drains, if used, should discharge well beyond the perimeter of the structure foundation, or be provided with positive conveyance off-site for collected waters. Downspouts should not be routed to discharge into an underdrain system.

If roof downspouts and drains are not used, then surface drainage design should anticipate concentrated volumes of water adjacent to the buildings.

7) Irrigation water—both that applied to landscaped areas and over-spray—commonly is a significant cause of distress to improvements. Where (near-) saturated soil conditions are sustained, distress to nearby improvements should be anticipated.

To reduce to potential for such distress, vegetation requiring watering should be located **10 or more feet** from the building perimeter, flatwork, or other improvements. Irrigation sprinkler heads should be deployed so that applied water is not introduced near or into foundation/subgrade soils. Landscape irrigation should be limited to the minimum quantities necessary to sustain healthy plant growth.

Use of drip irrigation systems can be beneficial for reducing over-spray beyond planters. Drip irrigation also can be beneficial for reducing the amounts of water introduced to building foundation soils, but only if the total volumes of applied water are controlled with regard to limiting that introduction. Controlling rates of moisture increase beneath the foundations, floors and other improvements should take higher priority than minimizing landscape plant losses.

Where plantings are desired within 10 feet of the building, plants should be placed in water-tight planters, constructed either in-ground or above-grade, to reduce moisture infiltration in the surrounding subgrade soils. Planters should be provided with positive drainage and landscape underdrains.

As an alternative involving only a limited increase in risk, the use of water-tight planters may be replaced by local, shallow underdrains beneath the planter beds.

8) Plastic membranes should not be used to cover the ground surface near the building without careful consideration of other components of project drainage. Plastic membranes can be beneficial to directing surface waters away from the building and toward drainage structures. However, they effectively preclude evaporation and transpiration of shallow soil moisture. Therefore, soil moisture

tends to increase beneath a continuous membrane. Where plastic membranes are used, additional shallow, subsurface drains should be installed.

Perforated "weed barrier" membranes that allow ready evaporation from the underlying soils may be used.

SUBSURFACE DRAINAGE

As a component of project civil design, properly functioning, subsurface drain systems ("underdrains") which, in principle, could include both perimeter underdrains and sub-floor lateral 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 heave or 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 a perimeter underdrain should be installed along the western and norther sides of the building where the walls will retain soil. Installing underdrains in other portions of the building could be beneficial, but may be excluded if the owner prefers. If other underdrains are 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. 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.

Damp-proofing should be applied to the exteriors of below-grade elements. The provision of Tencate Mirafi® G-Series backing (or comparable wall drain provisions) on the exteriors of below-grade elements may be appropriate, depending on the intended use.

GROUND is available to discuss the above options and as well as other underdrain alternatives upon request.

Geotechnical Parameters for Underdrain Design Underdrain design should incorporate the parameters below. The actual underdrain layouts, outlets, and locations should be developed by a civil engineer. Typical, cross-section details of underdrains that may be implemented for this project are provided in Figures 14 and 15.

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 a building 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 at least **30 gallons per minute** 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 as near as practical to collection and discharge pipe elbows of **60 degrees or more**.

6) If a below-grade level is included, the underdrain system should include both a perimeter drain and lateral drains. Lateral drains should be spaced such that no point of the basement floor is more than **50 feet** horizontally from a perimeter or lateral drain collection pipe.

7) 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.

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

PAVEMENT SECTIONS

A pavement section is a layered system designed to distribute concentrated traffic loads to the subgrade. Performance of the pavement structure is directly related to the physical properties of the subgrade soils and traffic loadings.

Standard practice in pavement design describes a typical flexible pavement section as a "20-year" design pavement. However, a pavement should not be anticipated to remain in satisfactory condition without routine maintenance and rehabilitation procedures performed throughout the life of the pavement.

Pavement sections for the private pavements at the subject facility were developed in general accordance with the guidelines and procedures of the American Association of State Highway and Transportation Officials (AASHTO) and local pavement construction practice.

Subgrade Materials Our data indicate that the shallow soils at the site classify primarily as A-1-b and A-2-4 soils with group index values of up to 0 in accordance with the AASHTO classification system. Such soils generally provide fair to good subgrade support.

A resilient modulus value of 8,000 psi was estimated to be representative of the soils at the project site and was used in the development of the pavement sections. It is important to note that significant decreases in soil support have been observed as the moisture content increases above the optimum. Additionally, the performance of the pavements will be significantly decreased if the subgrade is not prepared in general accordance with the *Subgrade Preparation* section below due to the relatively shallow soft and wet soils. Pavements that are not properly drained may experience a loss of the soil support and subsequent reduction in pavement life.

Anticipated Traffic Project-specific traffic loads had not been provided to GROUND at the time of preparation of this report. Therefore, assumed traffic loadings were used to develop the pavement section alternatives based on our experience with similar facilities.

An ESAL value of 22,000 (corresponding to an EDLA value of 3 for a 20-year design life) was assumed for parking stalls for light vehicles (automobiles and similar). An ESAL value of 73,000 (corresponding to an EDLA value of EDLA of 10 for a 20-year design life) was assumed for the facility driveways. An ESAL value of 365,000 (corresponding to an EDLA value of 50 for a 20-year design life) was assumed for the truck loading and unloading areas, trash collection routes, fire truck routes, and other pavement areas subject to heavy vehicle traffic. If design traffic loadings differ significantly from these assumed values, GROUND should be notified to reevaluate the pavement sections below.

Pavement Sections The soil resilient modulus and the ESAL values were used to determine the required structural number for the project pavements which then was then used to develop the pavement sections based on the DARWin™ computer program that solves the 1993 AASHTO pavement equations. A reliability level of 85 percent and a terminal serviceability of 2.0 were utilized for design of the pavement sections. A structural coefficient of 0.44 was used for hot bituminous asphalt and 0.12 was used for aggregate base course. The minimum pavement sections for a 20-year design are tabulated below.

Minimum Pavement Sections

Location	***Full Depth Asphalt*** *(inches Asphalt)*	***Composite Section*** *(inches Asphalt / inches Aggregate Base)*	***Rigid Section*** *(inches Concrete / inches Aggregate Base)*
Parking Stalls	**4**	**3 / 6**	**6 / 6**
Drive Lanes	**5**	**4 / 6**	**6 / 6**
Heavy Truck Traffic and Fire Lanes	**-**	**-**	**6½ / 6**

For best performance, truck loading and unloading areas, fire lanes, trash collection areas, as well as other pavement areas subjected to high turning stresses or heavy truck traffic should be provided with rigid pavements consisting of **6½ or more inches** of portland cement concrete underlain by **6 inches** of properly compacted CDOT Class 6 Aggregate Base Course. A mathematically equivalent flexible pavement section for these areas would be 4 inches of asphalt over 8 inches of aggregate base course. However, in our experience, asphalt pavements will not perform as well as rigid pavement in areas of high turning stresses or prolonged static loading, and additional maintenance costs (repairing tearing by pushing and tearing) should be anticipated if either of these sections were selected.

Pavement Materials Asphalt pavement should consist of a bituminous plant mix composed of a mixture of aggregate and bituminous material. Asphalt mixture(s) should meet the requirements of a job-mix formula established by a qualified engineer as well as applicable municipal design requirements, which may differ from those presented below. We anticipate that the following binders and gradations will be appropriate for the project, although others could be used.

Aggregate gradation **S** (nominal ¾-inch) and binder type **PG58-28** should be used for the lower lift(s), and gradation **SX** (nominal ½-inch) and binder type **PG64-22** for the top lift. Other binders may be appropriate, based on other considerations, however.

For the lower (S) lift(s), lift thicknesses generally should be between **2¼ and 3½ inches**. The top (SX) lift generally should be between **2 and 3 inches** in thickness.

Aggregate base material should meet the criteria of CDOT Class 6 Aggregate Base Course. Base course should be placed in and compacted in accordance with the standards in the *Project Earthwork* section of this report.

Pavement concrete should consist of a plant mix composed of a mixture of aggregate, portland cement and appropriate admixtures meeting the requirements of a job-mix formula established by a qualified engineer as well as applicable municipal design requirements design requirements. Concrete should have a minimum modulus of rupture of third point loading of **650 psi**. Normally, concrete with a 28-day compressive strength of **4,500 psi** should develop this modulus of rupture value. The concrete should be air-entrained with approximately 6 percent air and should have a minimum cement content of **6 sacks per cubic yard**. Maximum allowable slump should be **4 inches**.

These concrete mix design criteria should be coordinated with other project requirements including any criteria for sulfate resistance presented in the *Water-Soluble Sulfates* section of this report. To reduce surficial spalling resulting from freeze-thaw cycling, we suggest that pavement concrete meet the requirements of CDOT Class P concrete. In addition, the use of de-icing salts on concrete pavements during the first winter after construction will increase the likelihood of the development of scaling. Placement of flatwork concrete during cold weather so that it is exposed to freeze-thaw cycling before it is fully cured also increases its vulnerability to scaling. Concrete placing during cold weather conditions should be blanketed or tented to allow full curing. Depending on the weather conditions, this may result in 3 to 4 weeks of curing, and possibly more.

Concrete pavements should contain sawed or formed joints. CDOT and various industry groups provide guidelines for proper design and concrete construction and associated jointing. In areas of repeated turning stresses, such as truck loading and unloading areas, the concrete pavement joints should be fully tied and doweled. Example layouts for joints, as well as ties and dowels, which may be applicable, can be found in CDOT's M standards, found at the CDOT website: PCA, ACI, and ACPA publications also provide useful guidance in these regards. Joint spacings less than the 15-foot maximum indicated in in CDOT's M standards, e.g., 10 feet or 12 feet, may be beneficial to reduce concrete cracking.

Subgrade Preparation Remedial earthwork to any depth will not prevent pavement distress on these soils, but will tend to reduce it and improve perceived rideability. At this site, maintenance measures should be expected to include the removal, regrading, and replacement of distressed pavement areas.

<u>*Remedial Earthwork*</u> Based on the plasticity of the soils and CDOT guidelines, the pavements should be constructed, in general, on a section of properly moisture-conditioned and compacted to a depth of **at least 12 inches or a depth that removes and replaces all undocumented fill soils, whichever is greater.** We anticipate that excavations of up to about 3 feet will be necessary to remove and replace the undocumented fill soils. This section assumes that a) traffic speeds in the parking areas and driveways will be relatively slow, and b) the facility owner will be tolerant of significant total and differential pavement post-construction movements (on the order of several inches) and the associated maintenance costs that that are necessary to re-establish effective drainage, replace distressed pavement, etc.

We understand, however, that it may not be practical to remove and replace all the undocumented fill soils as properly compacted fill. Therefore, if the owner opts to reduce the fill section beneath the pavements, additional post-construction movements, accelerated pavement distress, and additional maintenance should be anticipated. We suggest remedial earthwork should be performed to no less than 12 inches in such a case. Similarly, where existing utility lines or other site constraints limit the depth to which remedial earthwork can be accomplished, additional maintenance should be anticipated.

In general, increasing the depth of fill beneath the pavements will decrease the risk of post-construction movements. If performance like project floors is desired, then project pavements should be constructed in a similar manner as project floors.

Criteria for placement and compaction of fill are provided in the *Project Earthwork* section of this report.

Subgrade preparation of the selected depth should extend the full width of the pavement from back-of-curb to back-of-curb. The subgrade for any sidewalks and other project hardscaping also should be prepared in the same manner.

Geotechnical criteria for fill placement and compaction are provided in the *Project Earthwork* section of this report. Where existing pavement and flatwork are removed, the contractor should anticipate that the shallowest soils may be wet. The contractor should be prepared to either dry the subgrade materials or moisten them, as needed, prior to compaction.

Proof Rolling Immediately prior to paving, the subgrade should be proof rolled with a heavily loaded, pneumatic tired vehicle. Areas that show excessive deflection during proof rolling should be excavated and replaced and/or stabilized. Areas allowed to pond prior to paving will require significant re-working prior to proof-rolling. Establishment of a firm paving platform (as indicated by proof rolling) is an additional requirement beyond proper fill placement and compaction. It is possible for soils to be compacted within the limits indicated in the *Project Earthwork* section of this report and fail proof rolling, particularly in the upper range of moisture content.

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

Landscape irrigation in planters adjacent to pavements and in “island” planters within paved areas should be carefully controlled or differential heave and/or rutting of the nearby pavements will result. Drip irrigation systems are suggested for such planters to reduce over-spray and water infiltration beyond the planters. Enclosing the soil in the planters with plastic liners and providing them with positive drainage also will reduce differential moisture increases in the surrounding subgrade soils.

In our experience, infiltration from planters adjacent to pavements is a principal source of moisture increase beneath those pavements. This wetting of the subgrade soils from infiltrating irrigation commonly leads to loss of subgrade support for the pavement with resultant accelerating distress, loss of pavement life, and increased maintenance costs. This is particularly the case in the later stages of project construction after landscaping has been emplaced but heavy construction traffic has not ended. Heavy vehicle traffic over wetted subgrade commonly results in rutting and pushing of flexible pavements, and cracking of rigid pavements. In relatively flat areas where design drainage gradients necessarily are small, subgrade settlement can obstruct proper drainage and yield increased infiltration, exaggerated distress, etc. (These considerations apply to project flatwork, as well.)

Also, GROUND's experience indicates that longitudinal cracking is common in asphalt-pavements generally parallel to the interface between the asphalt and concrete structures such as curbs, gutters, or drain pans. Distress of this type is likely to occur even where the subgrade has been prepared properly and the asphalt has been compacted properly.

The anticipated traffic loading does not include excess loading conditions imposed by heavy construction vehicles. Consequently, heavily loaded concrete, lumber, and building material trucks can have a detrimental effect on the pavement.

Most pavements will not remain in satisfactory condition and achieve their "design lives" without regular maintenance and rehabilitation procedures performed throughout the life of the pavement. Maintenance and rehabilitation measures preserve, rather than improve, the structural capacity of the pavement structure. Therefore, an effective program of regular maintenance should be developed and implemented to seal cracks, repair distressed areas, and perform thin overlays throughout the lives of the pavements. The greatest benefit of pavement overlaying will be achieved by overlaying sound pavements that exhibit little or no distress.

Crack sealing should be performed at least annually and a fog seal/chip seal program should be performed on the pavements every 3 to 4 years. After approximately 8 to 10 years after construction, patching, additional crack sealing, and asphalt overlay may be required. Prior to overlays, it is important that all cracks be sealed with a flexible, rubberized crack sealant in order to reduce the potential for propagation of the crack through the overlay. If actual traffic loadings exceed the values used for development of the pavement sections, however, pavement maintenance measures will be needed on an accelerated schedule.

Temporary Fire Access Routes Commonly, construction sites are required by local fire departments to provide temporary access for emergency response. It has been GROUND's experience these access drives are to provide support for trucks weighing up to 90,000 pounds and are typically desired to be gravel/aggregate-surfaced.

Based on our experience, a temporary section consisting of **at least 12 inches** of material meeting the requirements of CDOT Class 5 or Class 6 Aggregate Base Course or **at least 8 inches** of CDOT Class 5 or Class 6 Aggregate Base Course over **a layer of stabilization geotextile/geofabric**, such as Mirafi® RS380*i* or the equivalent, could be utilized provided the owner understands that this section is for temporary access during construction only and is not a replacement or an equal alternate to the pavement section(s) that was indicated previously. The aggregate base course placed for this purpose should be placed and compacted in accordance with the criteria in the *Project Earthwork* section of this report. It should be noted that the aggregate base course sections indicated above are not intended to support fire truck outriggers without cribbing or similar measures.

The aggregate comprising such a wearing course will be displaced and rutted under the loads imposed by heavy vehicles. Therefore, regular maintenance including regrading and application of additional aggregate should be implemented to ensure proper drainage, repair distressed/damaged areas, and reestablish grades. Additionally, the ability of a temporary aggregate-surfaced route to accommodate loads as indicated above is directly related to the quality of the subgrade materials on which the aggregate is placed, not only on the aggregate section. If water infiltrates these areas, additional rutting and other distress, including a reduction in capacity, will result, requiring additional maintenance.

EXTERIOR FLATWORK

We anticipate that the exterior of the proposed building and other portions of the site will be provided with concrete flatwork. Like other site improvements, flatwork will experience post-construction movements as soil moisture contents increase after construction and distress likely will result. The following measures will help to reduce damages to these improvements, but will not prevent all movements. Critical flatwork, which may include flatwork at entrances and exits, should be constructed as a floor in a similar manner to the associated building. Such areas should be identified by the owner.

1) Remedial earthwork to prepare flatwork subgrades is subject to the same factors discussed in the *Pavement Sections* section of this report, and should be undertaken to the same depth.

2) Prior to placement of flatwork, a proof roll should be performed to identify areas that exhibit instability and deflection. The deleterious soils in these areas should be removed and replaced with properly compacted fill. The contractor should take care to achieve and maintain compaction behind curbs to reduce differential sidewalk settlements. Passing a proof roll is an additional requirement to placing and compacting the subgrade fill soils within the specified ranges of moisture content and relative compaction in the *Project Earthwork* section of this report. Subgrade stabilization may be cost-effective in this regard.

3) Flatwork should be provided with control joints extending to an effective depth and spaced no more than **10 feet** apart, both ways. Narrow flatwork, such as sidewalks, likely will require more closely spaced joints.

4) In no case should exterior flatwork extend to under any portion of the building where there is less than **2 inches** of vertical clearance between the flatwork and any element of the building to allow for frost heave, larger movement may result from post-construction movements of flatwork due to heave. Exterior flatwork in contact with brick, rock facades, or any other element of the building can cause damage to the structure if the flatwork experiences movements.

Construction and Drainage Between Buildings and Pavements Proper design, drainage, construction and maintenance of the areas between individual buildings and parking/driveway areas are critical to the satisfactory performance of the project. Sidewalks, entranceway slabs and roofs, fountains, raised planters and other highly visible improvements commonly are installed within these zones, and distress in or near these improvements is common. Commonly, proper soil preparation in these areas receives little attention during overlot construction because they fall between the building and pavement areas which typically are built with heavy equipment. Subsequent landscaping and hardscape installation often is performed by multiple sub-contractors with light or hand equipment, and necessary over-excavation and soil processing is not performed. Consequently, subgrade soil conditions commonly deviate significantly from specified ranges. Therefore, the contractor should take particular care with regard to proper subgrade preparation in the immediate building exteriors.

Concrete Scaling Climatic conditions in the project area including relatively low humidity, large temperature changes and repeated freeze-thaw cycles, make it likely that project sidewalks and other exterior concrete will experience surficial scaling or spalling. The likelihood of concrete scaling can be increased by poor workmanship during construction, such as 'over-finishing' the surfaces. In addition, the use of de-icing salts on exterior concrete flatwork, particularly during the first winter after construction, will increase the likelihood of scaling. Even use of de-icing salts on nearby roadways, from where vehicle traffic can transfer them to newly placed concrete, can be sufficient to induce scaling. Typical quality control / quality assurance tests that are performed during construction for concrete strength, air content, etc., do not provide information with regard to the properties and conditions that give rise to scaling.

We understand that some municipalities require removal and replacement of concrete that exhibits scaling, even if the material was within specification and placed correctly. The contractor should be aware of the local requirements and be prepared to take measures to reduce the potential for scaling and/or replace concrete that scales.

In GROUND's experience, the measures below can be beneficial for reducing the likelihood of concrete scaling. Which measures, if any, used should be based on cost and the owner's tolerance for risk and maintenance. It must be understood, however, that because of the other factors involved, including weather conditions and workmanship, surface damage to concrete can develop, even where all of these measures were followed. Also, the mix design criteria should be coordinated with other project requirements including criteria for sulfate resistance presented in the *Water-Soluble Sulfates* section of this report.

1) Maintaining a maximum water/cement ratio of 0.45 by weight for exterior concrete mixes.

2) Include Type F fly ash in exterior concrete mixes as 20 percent of the cementitious material.

3) Specify a minimum, 28-day, compressive strength of 4,500 psi for all exterior concrete.

4) Including 'fibermesh' in the concrete mix also may be beneficial for reducing surficial scaling.

5) Cure the concrete effectively at uniform temperature and humidity. This commonly will require fogging, blanketing and/or tenting, depending on the weather conditions. As long as 3 to 4 weeks of curing may be required, and possibly more.

6) Avoid placement of concrete during cold weather so that it is not exposed to freeze-thaw cycling before it is fully cured.

7) Avoid the use of de-icing salts on given reaches of flatwork through the first winter after construction.

We understand that sometimes it is not practical to implement some of these measures for reducing scaling due to safety considerations, project scheduling, etc. In such cases, where these measures are not implemented, additional costs for flatwork maintenance or reconstruction should be incorporated into project budgets.

Frost and Ice Considerations Nearly all soils other than relatively coarse, clean, granular materials are susceptible to loss of density if allowed to become saturated and exposed to freezing temperatures and repeated freeze-thaw cycling. The formation of ice in the underlying soils can result in heaving of pavements, flatwork, and other hardscaping ("ice jacking") in sustained cold weather up to 2 inches or more. This heaving can develop relatively rapidly. A portion of this movement typically is recovered when the soils thaw, but due to loss of soil density, some degree of displacement will remain. This can result even where the subgrade soils were prepared properly.

Where hardscape movements are a design concern, e.g., at doorways, replacement of the subgrade soils with 3 or more feet of clean, coarse sand or gravel should be considered or supporting the element on foundations similar to the building and spanning over a void. Detailed guidance in this regard can be provided upon request. It should be noted that where such open graded granular soils are placed, water can infiltrate and accumulate in the subsurface relatively easily, which can lead to increased settlement or heave from factors unrelated to ice formation. Therefore, where a section of open graded granular soils is placed, a local underdrain system should be provided to discharge collected water. GROUND will be available to discuss these concerns upon request.

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 earthworks 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 Meridian Service Metro District, 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 Meridian Service Metro District, as it pertains to design and construction of the proposed field house building and related improvements as described herein. It may not contain sufficient information for other parties or other purposes.

In addition, GROUND has assumed that project construction will commence by summer 2024. Any changes in project plans or schedule should be brought to the attention of the 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 in this report relied upon subsurface exploration at 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.

If during construction, surface, soil, bedrock, or groundwater conditions appear to be at variance with those described herein, a geotechnical engineer should be retained at once, so that reevaluation of the conclusions for this site may be made in a timely manner. In addition, a contractor who obtains information from this report for development of his scope of work or cost estimates 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 is responsible for obtaining the additional geotechnical information that is necessary to develop his workscope and cost estimates with sufficient precision. This includes current depths to groundwater, etc.

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 Meridian Service Metro District. Utilizing these criteria 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, then Meridian Service Metro District, or other members of the design team, should 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.

GROUND makes no warranties, either expressed or implied, as to the professional data, opinions or conclusions contained herein. 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 Meridian Service Metro District, with a proposal for construction observation and materials testing.

Sincerely,

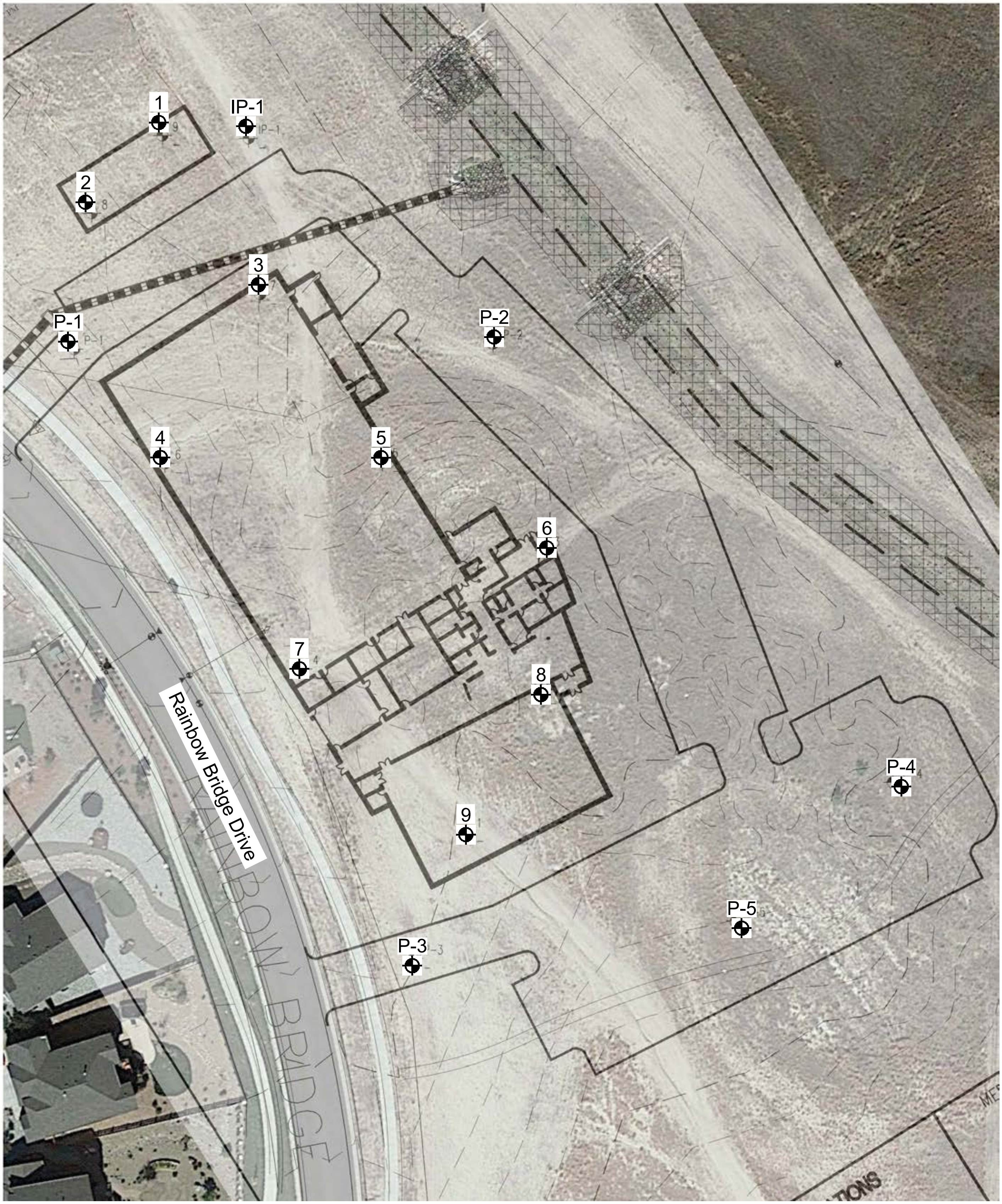
GROUND Engineering Consultants, Inc.

COLORADO LICENSED
BRIAN H. RECK
48265
PROFESSIONAL ENGINEER

1/10/2024

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

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



GOOGLE EARTH AERIAL IMAGE (10/06/2019)
SITE PLAN PROVIDED BY CLIENT

9 ⌖ Indicates test hole numbers and approximate locations.

NOT TO SCALE

GROUND ENGINEERING

JOB NO.:	23-8008
FIGURE:	1

GENERAL PAD LOCATIONS

LOGS OF THE TEST HOLES

GROUND ENGINEERING

PROJECT: Meridian Ranch Field House

CLIENT: Meridian Service Metro District

JOB NO: 23-8008

SITE LOCATION: Falcon, CO

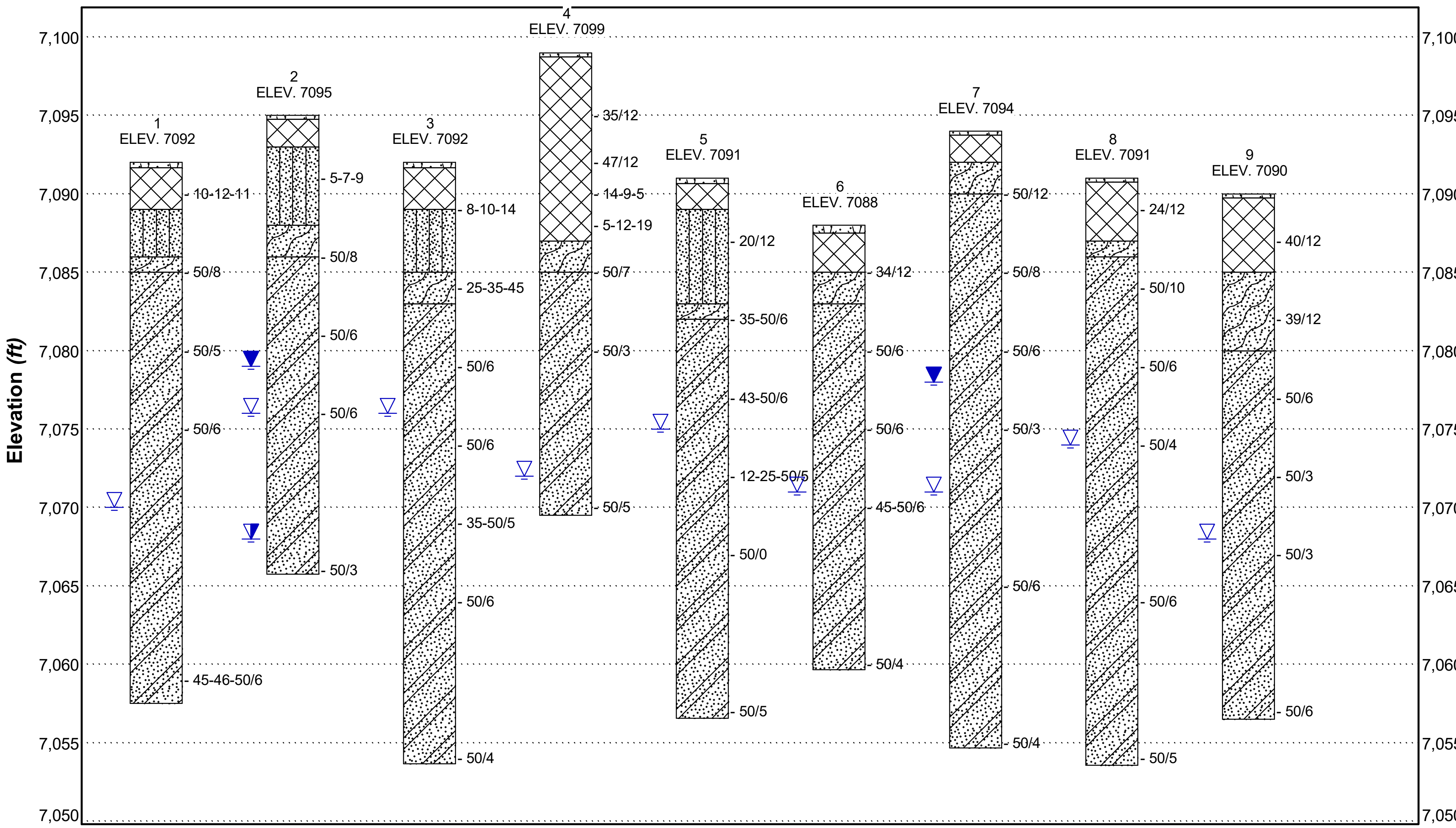


Figure 2

LOGS OF THE TEST HOLES

GROUND ENGINEERING

PROJECT: Meridian Ranch Field House

CLIENT: Meridian Service Metro District

JOB NO: 23-8008

SITE LOCATION: Falcon, CO

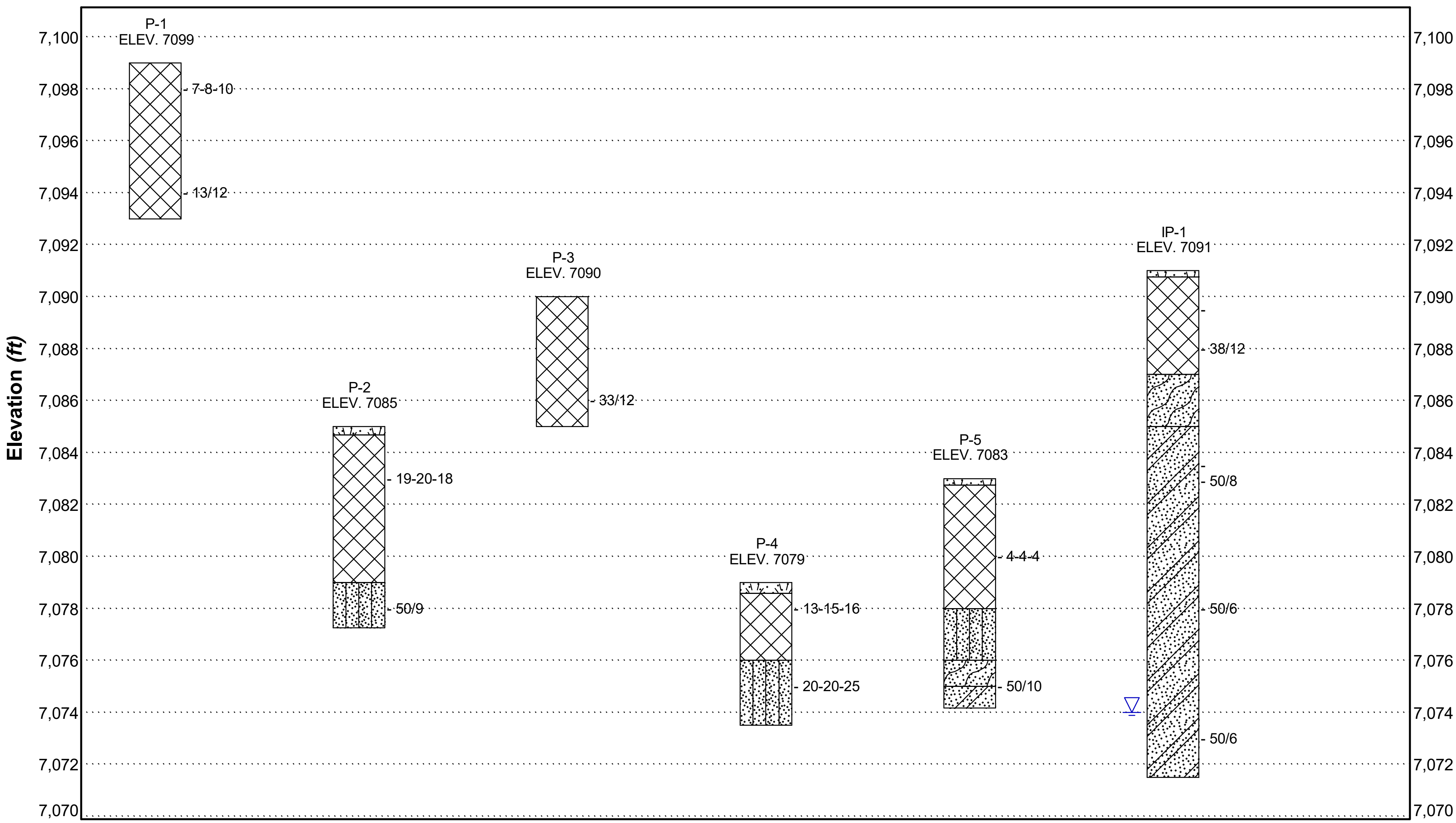


Figure 3

LEGEND AND NOTES

GROUND ENGINEERING

PROJECT: Meridian Ranch Field House

JOB NO: 23-8008

CLIENT: Meridian Service Metro District

SITE LOCATION: Falcon, CO

MATERIAL SYMBOLS

TOPSOIL

FILL

SAND

WEATHERED SANDSTONE

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.

Large Disturbed Sample

No Recovery

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 10/11/2023 with 4" Solid Stem Auger.

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

3. Elevations of the test holes were not measured and the logs of the test holes are drawn to depth. Nominal elevation of "100 feet" indicates existing ground level at the test hole at the time of drilling.

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

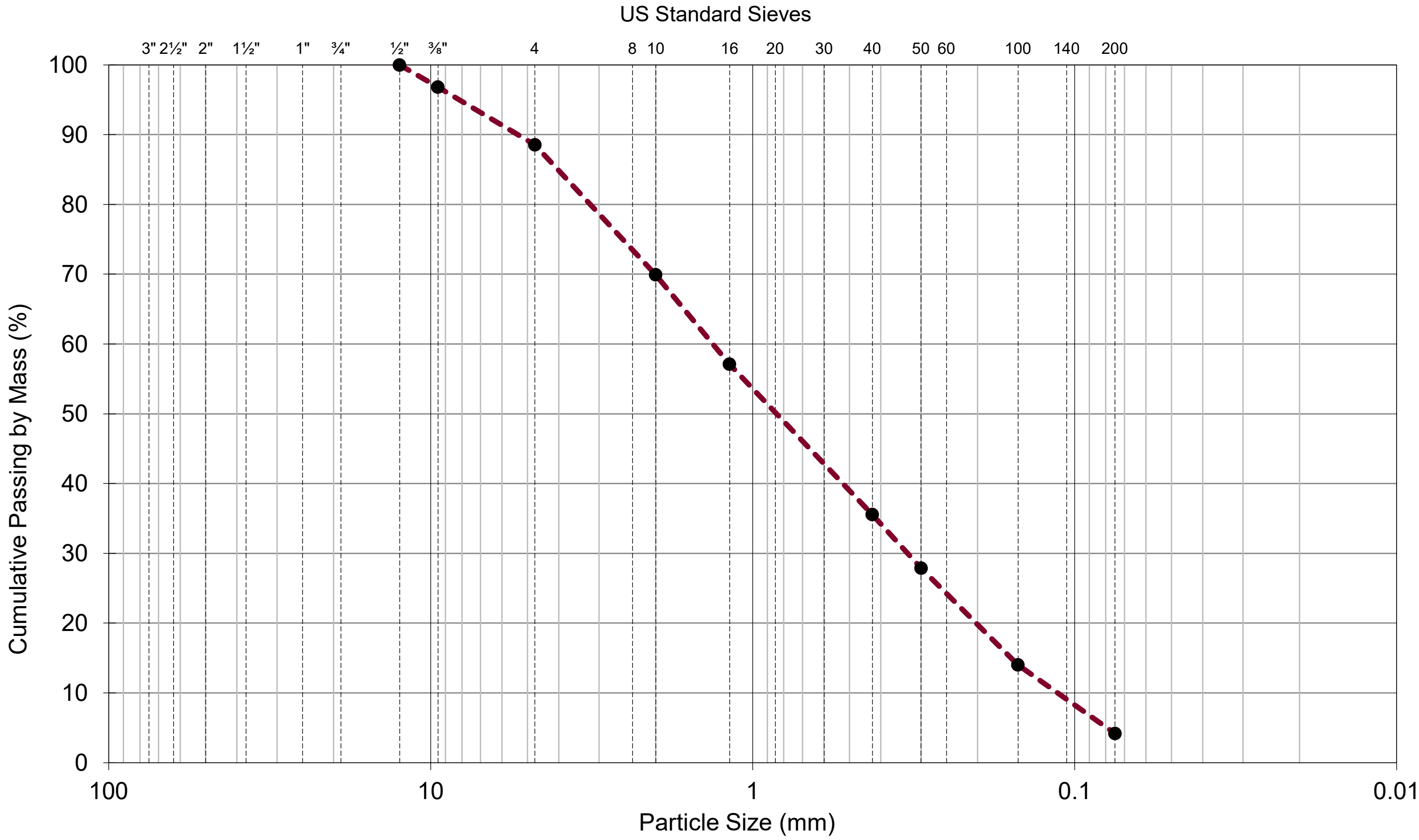
NV	No Value
NP	Non-Plastic

Figure 4

Project No.: 23-8008

Meridian Ranch Field House

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	89	D90	5.368
5 in	125	-	No. 8	2.36	-	D85	4.029
4 in	100	-	No. 10	2.00	70	D80	3.193
3 in	75	-	No. 16	1.18	57	D60	1.328
2.5 in	63	-	No. 20	0.85	-	D50	0.842
2 in	50	-	No. 30	0.60	-	D40	0.525
1.5 in	37.5	-	No. 40	0.425	36	D30	0.330
1 in	25.0	-	No. 50	0.300	28	D15	0.157
3/4 in	19.0	-	No. 60	0.250	-	D10	0.113
1/2 in	12.5	100	No. 100	0.150	14	D05	0.079
3/8 in	9.5	97	No. 140	0.106	-	Cu	11.759
No. 4	4.75	89	No. 200	0.075	4.2	Cc	0.728

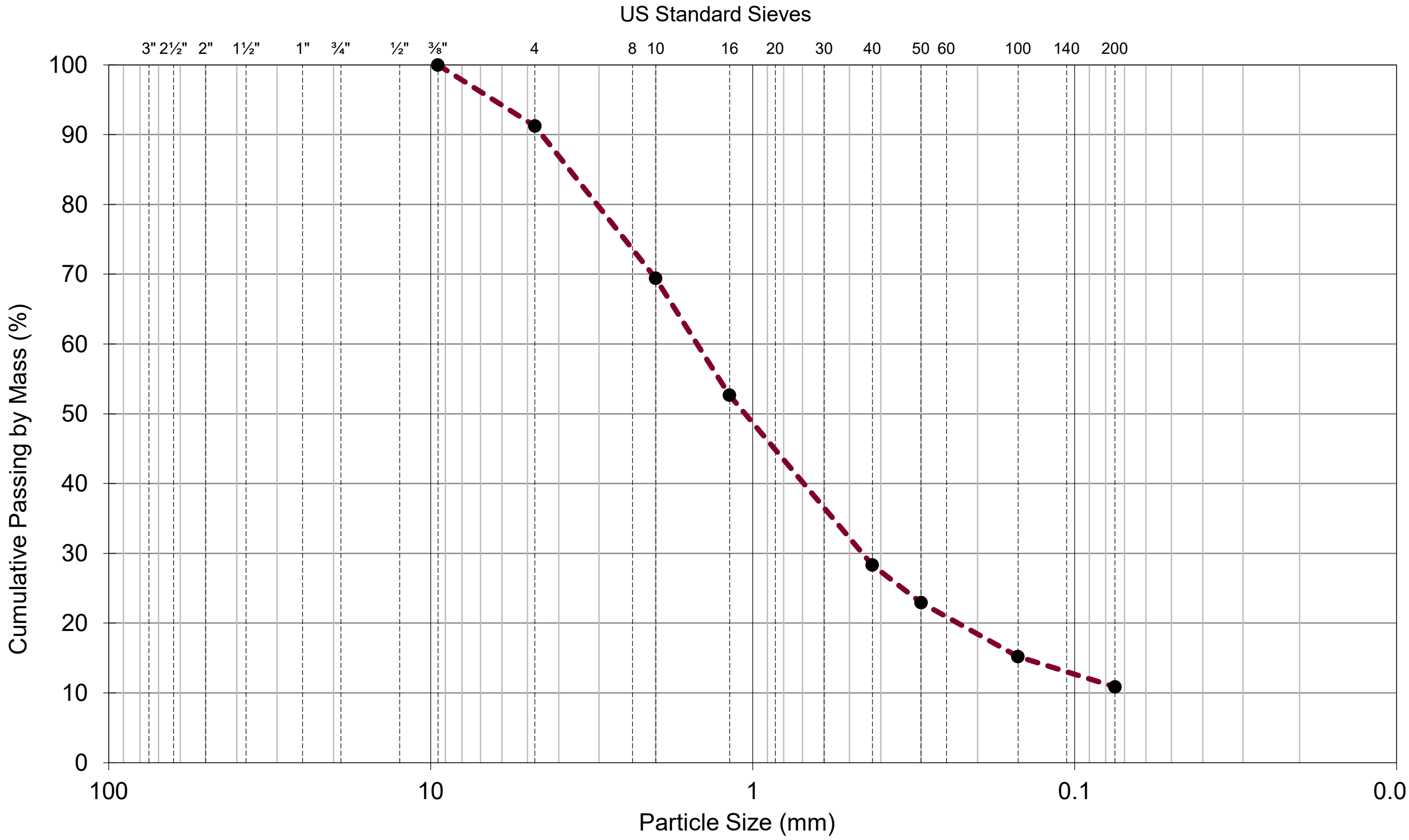
Location: 1 at 2 feet
Description: FILL: Clayey Sand

Classification: SP / A-1-b (0)
Liquid Limit: 21
Plasticity Index: 1

Gravel (%): 11
Sand (%): 85
Silt/Clay (%): 4

Meridian Ranch Field House

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	91	D90	4.518
5 in	125	-	No. 8	2.36	-	D85	3.707
4 in	100	-	No. 10	2.00	69	D80	3.041
3 in	75	-	No. 16	1.18	53	D60	1.487
2.5 in	63	-	No. 20	0.85	-	D50	1.055
2 in	50	-	No. 30	0.60	-	D40	0.693
1.5 in	37.5	-	No. 40	0.425	28	D30	0.455
1 in	25.0	-	No. 50	0.300	23	D15	0.144
3/4 in	19.0	-	No. 60	0.250	-	D10	-
1/2 in	12.5	-	No. 100	0.150	15	D05	-
3/8 in	9.5	100	No. 140	0.106	-	Cu	-
No. 4	4.75	91	No. 200	0.075	10.9	Cc	-

Location: 3 at 8 feet
Description: Weathered SANDSTONE

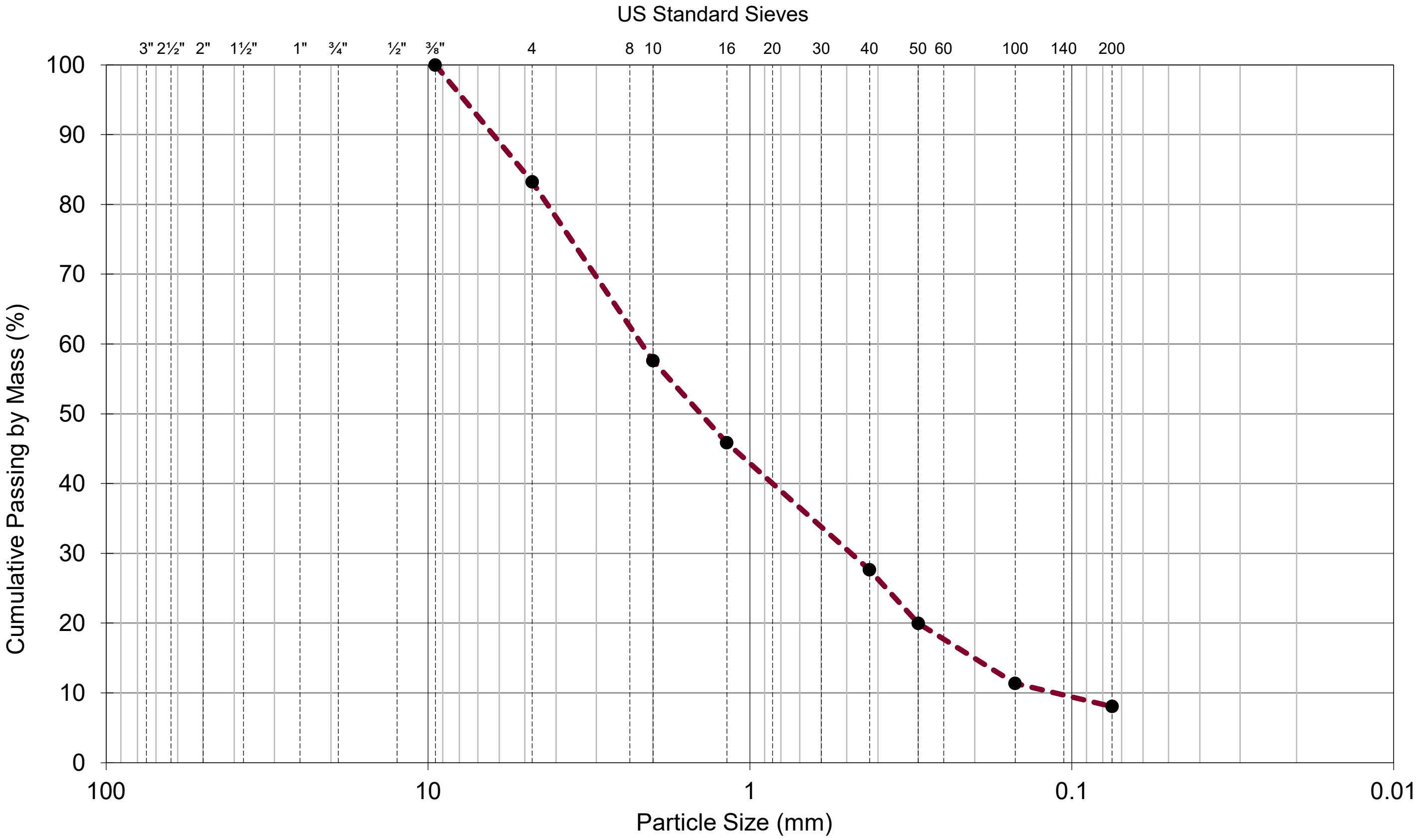
Classification: SP-SC / A-1-b (0)
Liquid Limit: 25
Plasticity Index: 4

Gravel (%): 9
Sand (%): 80
Silt/Clay (%): 11

Project No.: 23-8008

Meridian Ranch Field House

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	83	D90	6.281
5 in	125	-	No. 8	2.36	-	D85	5.107
4 in	100	-	No. 10	2.00	58	D80	4.257
3 in	75	-	No. 16	1.18	46	D60	2.166
2.5 in	63	-	No. 20	0.85	-	D50	1.420
2 in	50	-	No. 30	0.60	-	D40	0.848
1.5 in	37.5	-	No. 40	0.425	28	D30	0.484
1 in	25.0	-	No. 50	0.300	20	D15	0.201
3/4 in	19.0	-	No. 60	0.250	-	D10	0.112
1/2 in	12.5	-	No. 100	0.150	11	D05	-
3/8 in	9.5	100	No. 140	0.106	-	Cu	19.264
No. 4	4.75	83	No. 200	0.075	8.1	Cc	0.962

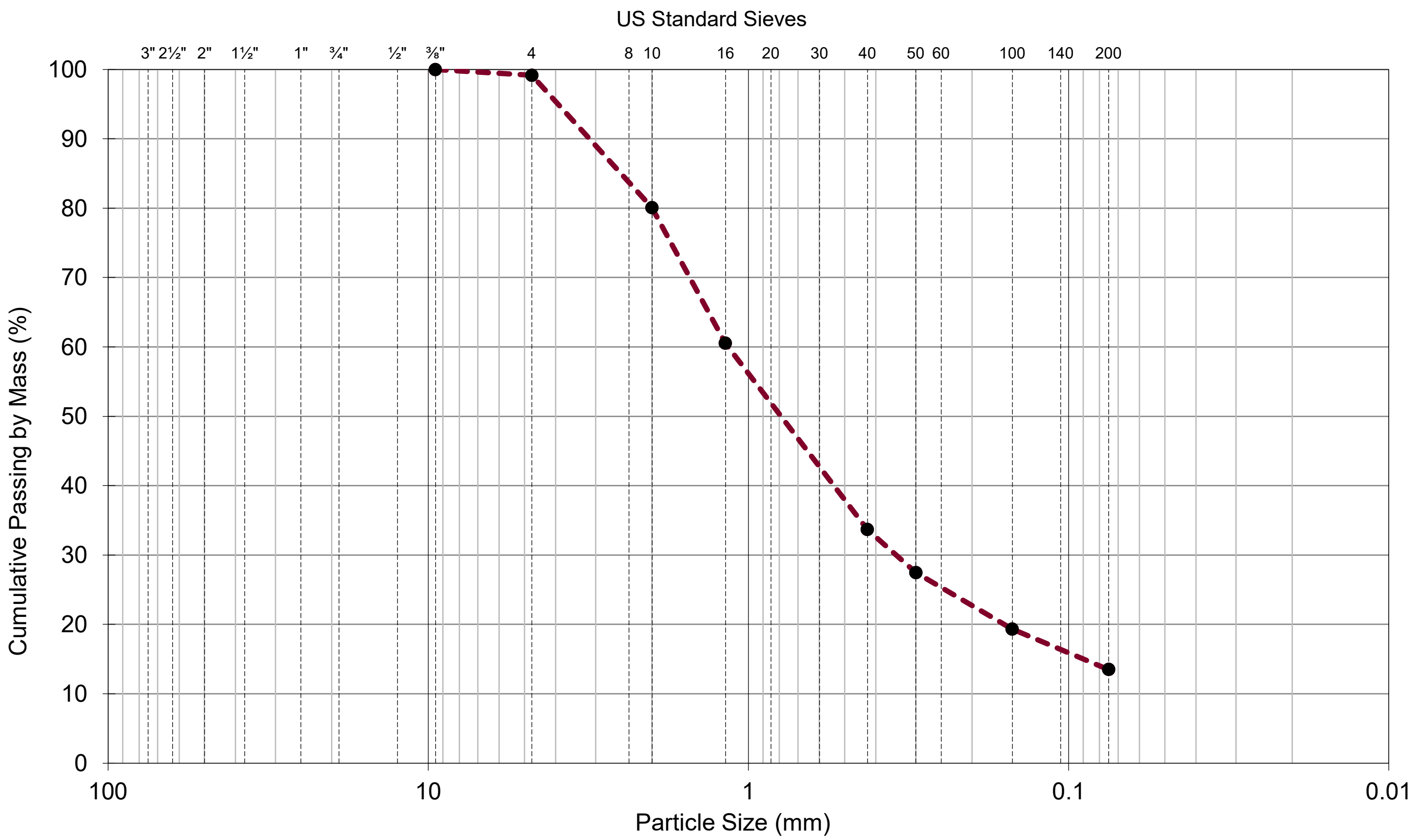
Location: 5 at 14 feet
Description: SANDSTONE Bedrock

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

Gravel (%): 17
Sand (%): 75
Silt/Clay (%): 8

Meridian Ranch Field House

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	99	D90	3.136
5 in	125	-	No. 8	2.36	-	D85	2.500
4 in	100	-	No. 10	2.00	80	D80	1.995
3 in	75	-	No. 16	1.18	61	D60	1.155
2.5 in	63	-	No. 20	0.85	-	D50	0.790
2 in	50	-	No. 30	0.60	-	D40	0.540
1.5 in	37.5	-	No. 40	0.425	34	D30	0.345
1 in	25.0	-	No. 50	0.300	27	D15	0.090
3/4 in	19.0	-	No. 60	0.250	-	D10	-
1/2 in	12.5	-	No. 100	0.150	19	D05	-
3/8 in	9.5	100	No. 140	0.106	-	Cu	-
No. 4	4.75	99	No. 200	0.075	13.5	Cc	-

Location: 6 at 13 feet
Description: SANDSTONE Bedrock

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

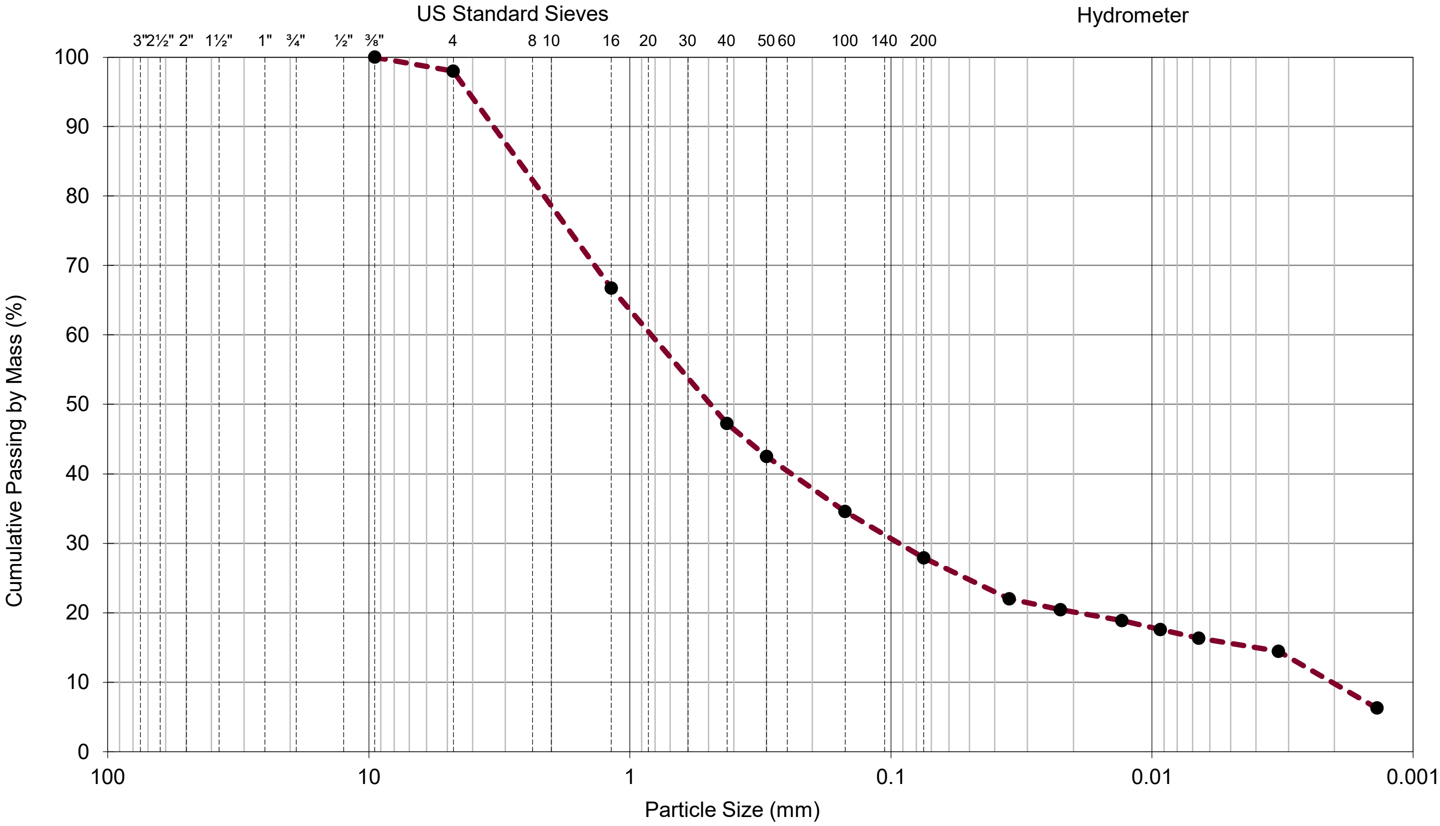
Gravel (%): 1
Sand (%): 85
Silt/Clay (%): 14

GROUND ENGINEERING

Project No.: 23-8008

Meridian Ranch Field House

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	98	0.035	22	D90	3.330
5 in	125	-	No. 8	2.36	-	0.022	20	D85	2.664
4 in	100	-	No. 10	2.00	-	0.013	19	D80	2.131
3 in	75	-	No. 16	1.18	67	0.009	18	D60	0.828
2.5 in	63	-	No. 20	0.85	-	0.007	16	D50	0.490
2 in	50	-	No. 30	0.60	-	0.003	14	D40	0.241
1.5 in	37.5	-	No. 40	0.425	47	0.001	6	D30	0.093
1 in	25.0	-	No. 50	0.300	42	-	-	D15	-
3/4 in	19.0	-	No. 60	0.250	-	-	-	D10	-
1/2 in	12.5	-	No. 100	0.150	35	-	-	D05	-
3/8 in	9.5	100	No. 140	0.106	-	-	-	Cu	-
No. 4	4.75	98	No. 200	0.075	27.9	-	-	Cc	-

Location: IP-1 at 0 to 3 feet
Description: FILL: Clayey Sand

Classification: SC / A-2-4 (0)
Liquid Limit: 28
Plasticity Index: 9
Activity: 0.9

Gravel (%): 2
Sand (%): 70
Silt/Clay (%): 28
< .002 mm (%): 10

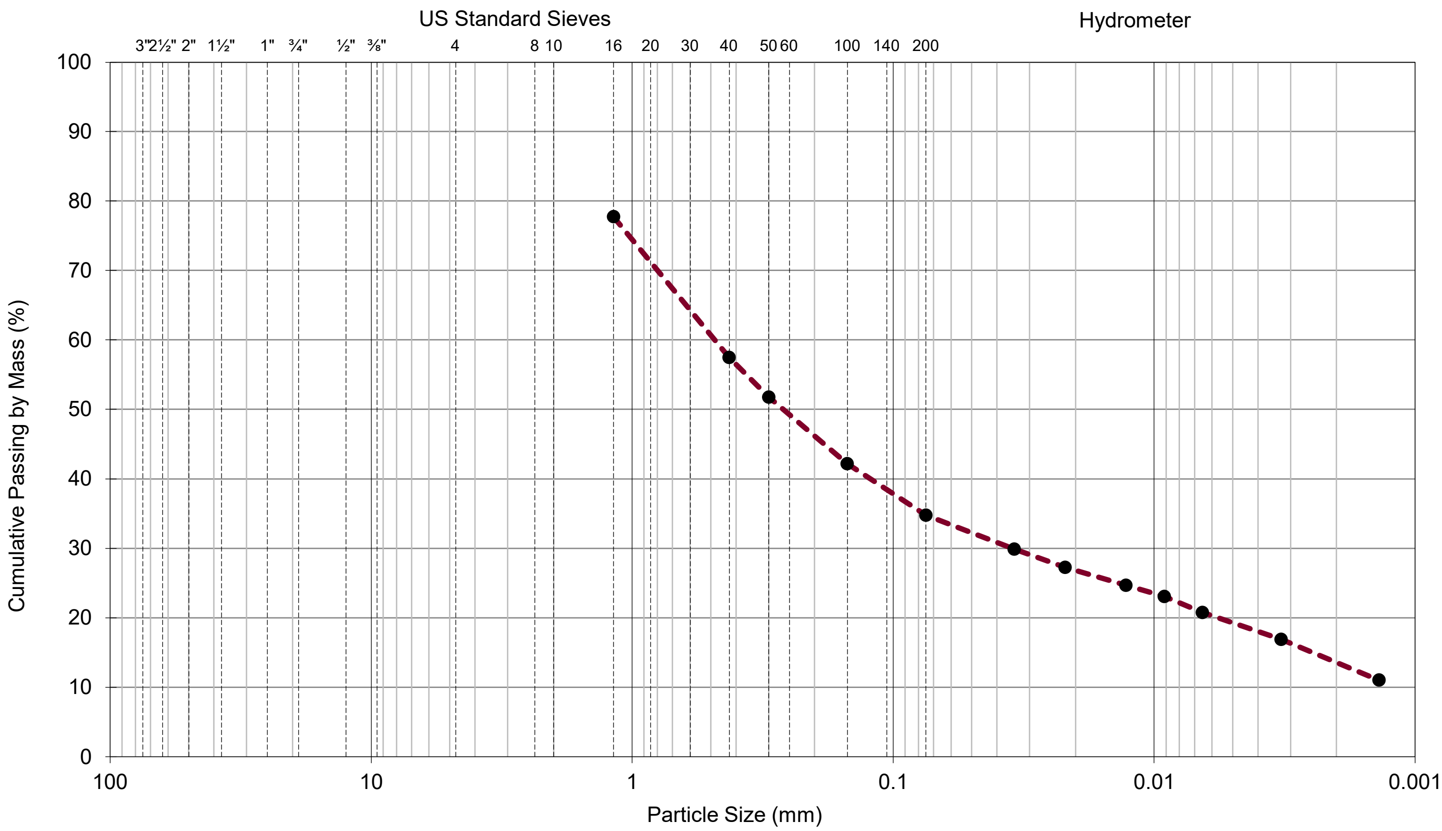
Results apply only to the specific items and locations referenced and at the time of testing. For the hydrometer portion of the test, a composite temperature correction and meniscus correction were applied to each reading. This report should not be reproduced, except in full, without the written permission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

GROUND ENGINEERING

Project No.: 23-8008

Meridian Ranch Field House

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	-	0.034	30	D90	#N/A
5 in	125	-	No. 8	2.36	-	0.022	27	D85	#N/A
4 in	100	-	No. 10	2.00	-	0.013	25	D80	#N/A
3 in	75	-	No. 16	1.18	78	0.009	23	D60	0.482
2.5 in	63	-	No. 20	0.85	-	0.007	21	D50	0.264
2 in	50	-	No. 30	0.60	-	0.003	17	D40	0.122
1.5 in	37.5	-	No. 40	0.425	57	0.001	11	D30	-
1 in	25.0	-	No. 50	0.300	52	-	-	D15	-
3/4 in	19.0	-	No. 60	0.250	-	-	-	D10	-
1/2 in	12.5	-	No. 100	0.150	42	-	-	D05	-
3/8 in	9.5	-	No. 140	0.106	-	-	-	Cu	-
No. 4	4.75	-	No. 200	0.075	34.8	-	-	Cc	-

Location: IP-1 at 6 to 8 feet
Description: SANDSTONE Bedrock

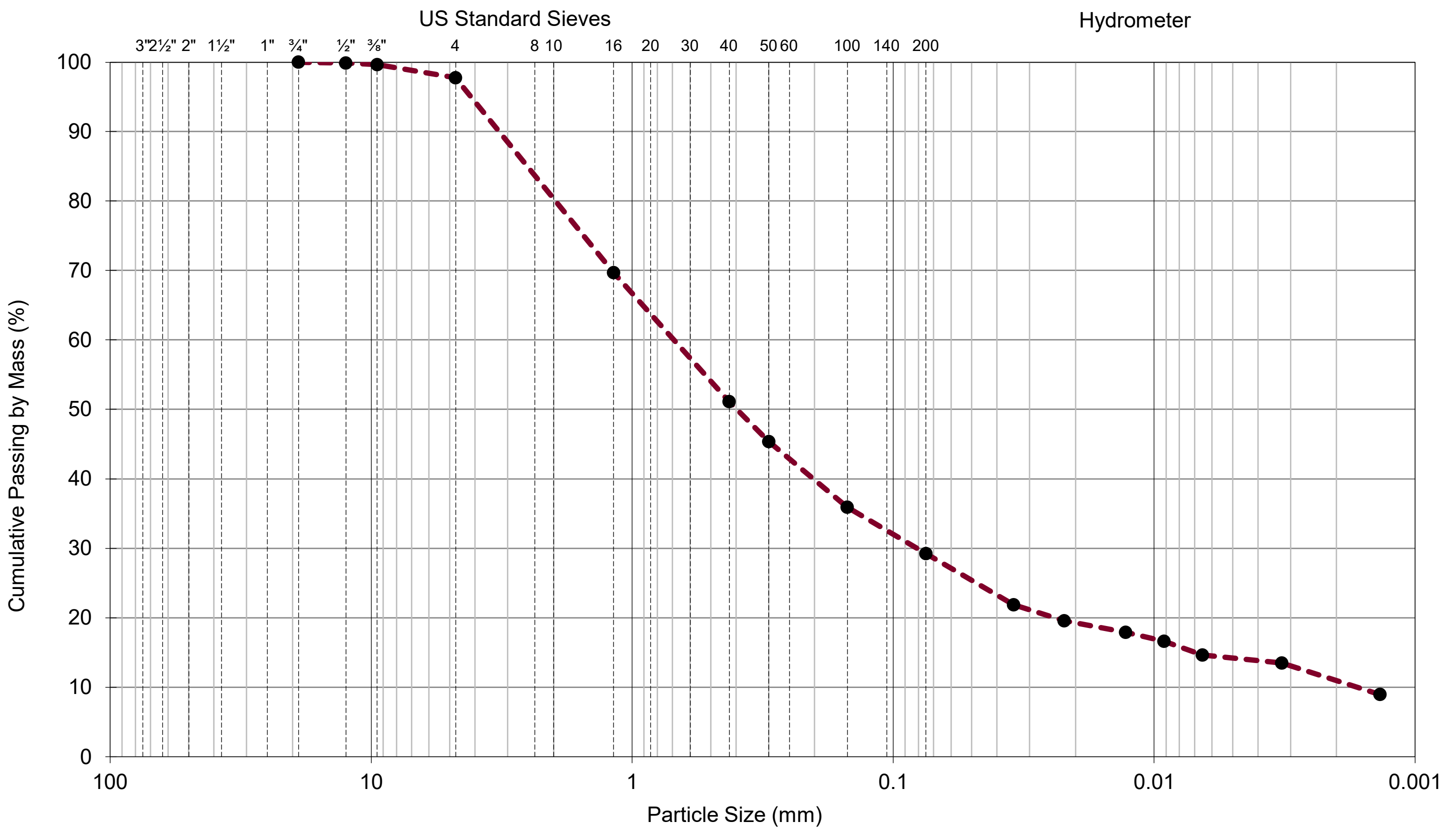
Classification: SC / A-2-4 (0)
Liquid Limit: 23
Plasticity Index: 8
Activity: 0.6

Gravel (%): 0
Sand (%): 65
Silt/Clay (%): 35
< .002 mm (%): 14

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.

Meridian Ranch Field House

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

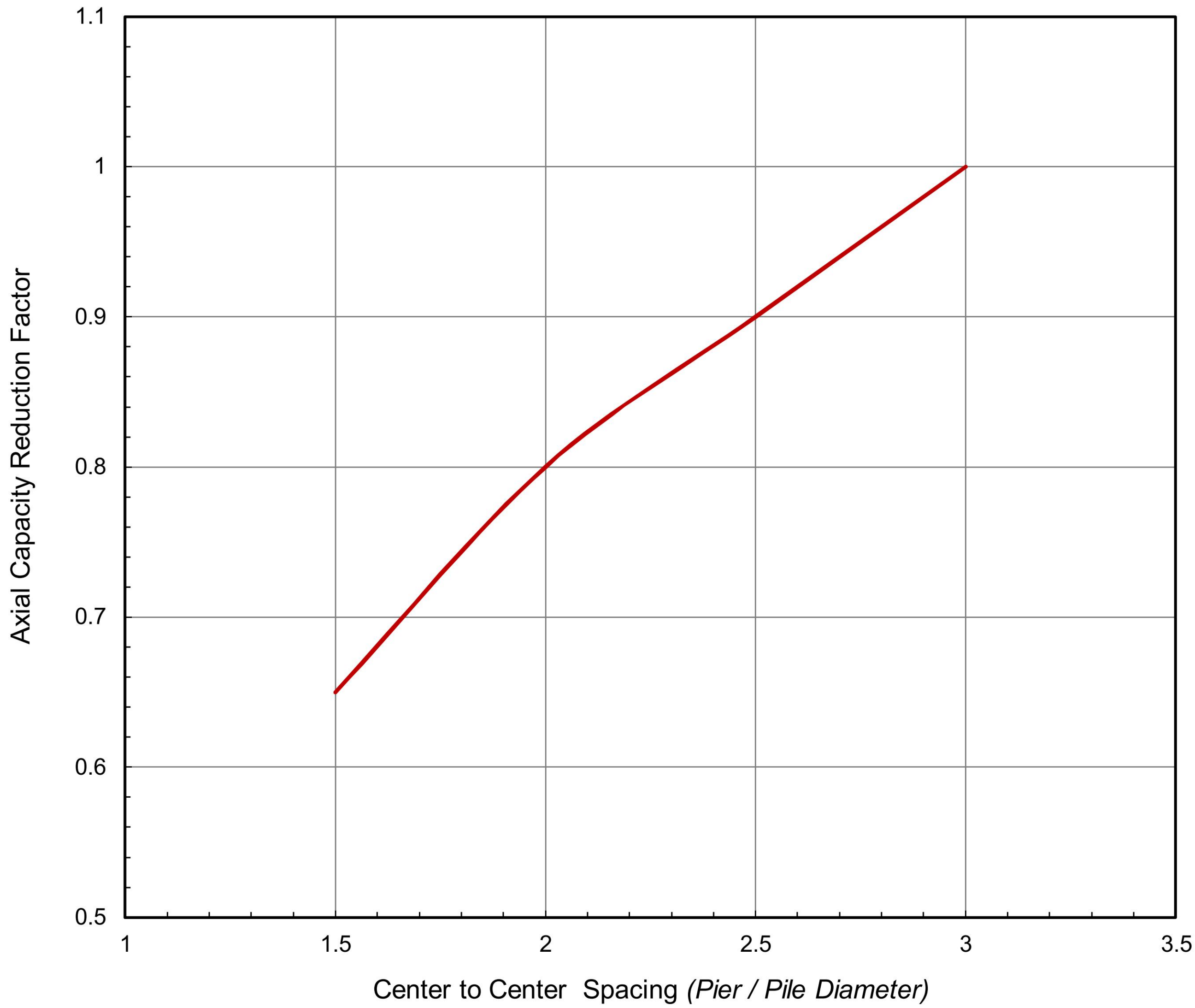


Coarse Gradation			Fine Gradation			Hydrometer		Grading	
US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	Particle Size (mm)	Passing by Mass (%)	Coefficient	Value
6 in	150	-	No. 4	4.75	98	0.035	22	D90	3.234
5 in	125	-	No. 8	2.36	-	0.022	20	D85	2.523
4 in	100	-	No. 10	2.00	-	0.013	18	D80	1.969
3 in	75	-	No. 16	1.18	70	0.009	17	D60	0.693
2.5 in	63	-	No. 20	0.85	-	0.007	15	D50	0.397
2 in	50	-	No. 30	0.60	-	0.003	13	D40	0.202
1.5 in	37.5	-	No. 40	0.425	51	0.001	9	D30	0.081
1 in	25.0	-	No. 50	0.300	45	-	-	D15	-
3/4 in	19.0	100	No. 60	0.250	-	-	-	D10	-
1/2 in	12.5	100	No. 100	0.150	36	-	-	D05	-
3/8 in	9.5	100	No. 140	0.106	-	-	-	Cu	-
No. 4	4.75	98	No. 200	0.075	29.3	-	-	Cc	-

Location: Composite
Description: Clayey SAND

Classification: SC / A-2-4 (0)
Liquid Limit: 27
Plasticity Index: 8
Activity: 0.7

Gravel (%): 2
Sand (%): 69
Silt/Clay (%): 29
< .002 mm (%): 11



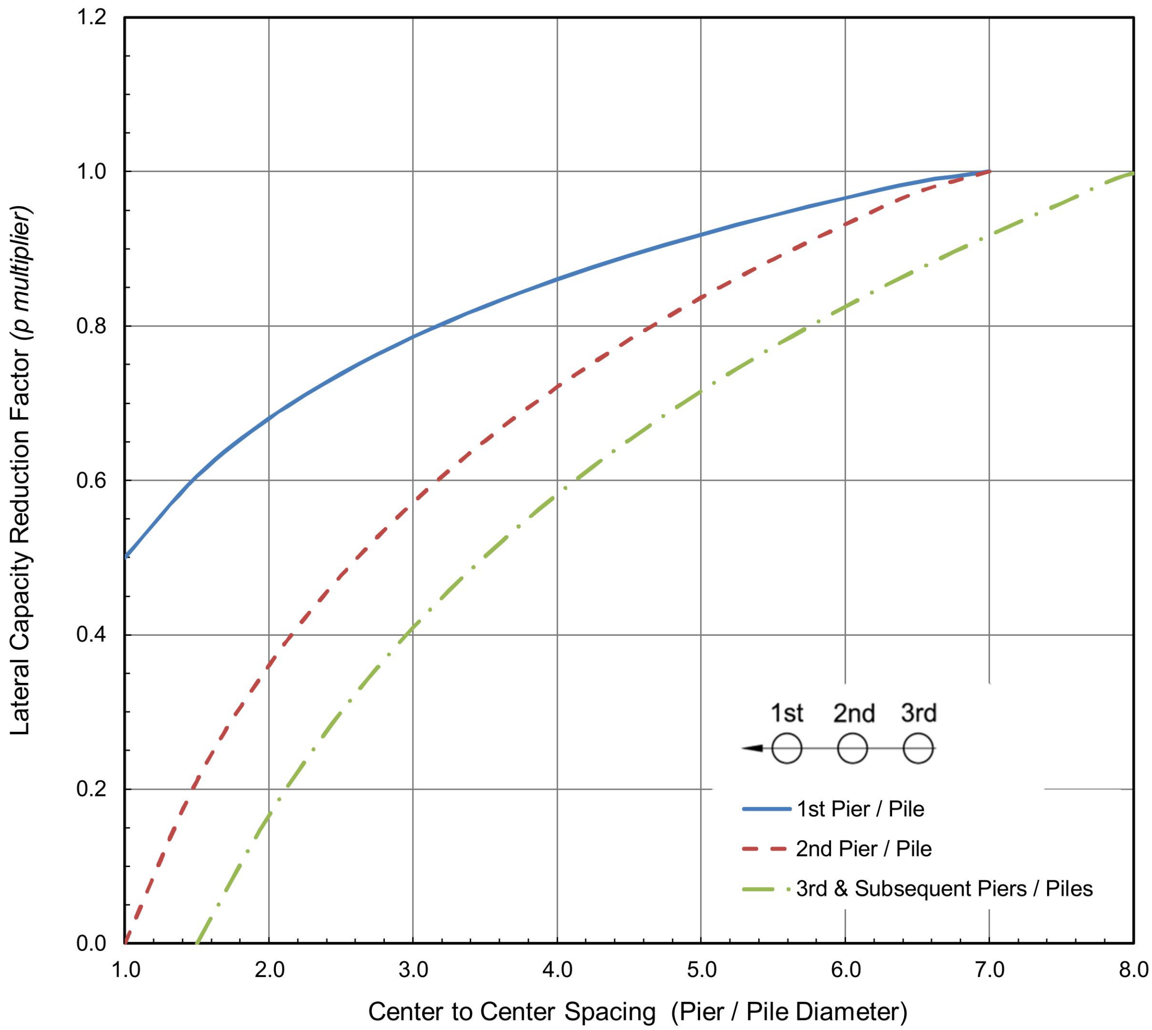
Axial Capacity Reductions as Functions of Closely Spaced Pier / Pile Elements.

The graph above provides estimated reductions in total axial capacity for closely spaced piers.

Pier / Pile reductions should be interpolated from the graph above.

GROUND ENGINEERING	JOB NO.: 23-8008
	FIGURE: 12

AXIAL CAPACITY
REDUCTION FACTORS
FOR CLOSELY SPACED PIERS / PILES



Lateral Capacity Reduction (*p multipliers*) as Functions of Closely Spaced Pier / Pile Elements

The "1st" or "lead" pier / pile is the element that leads movement in the direction that the lateral load will cause the piers to deflect, as shown.

For lateral loads oriented perpendicular to the row of piers / piles, use the 1st pier / pile p-multiplier.

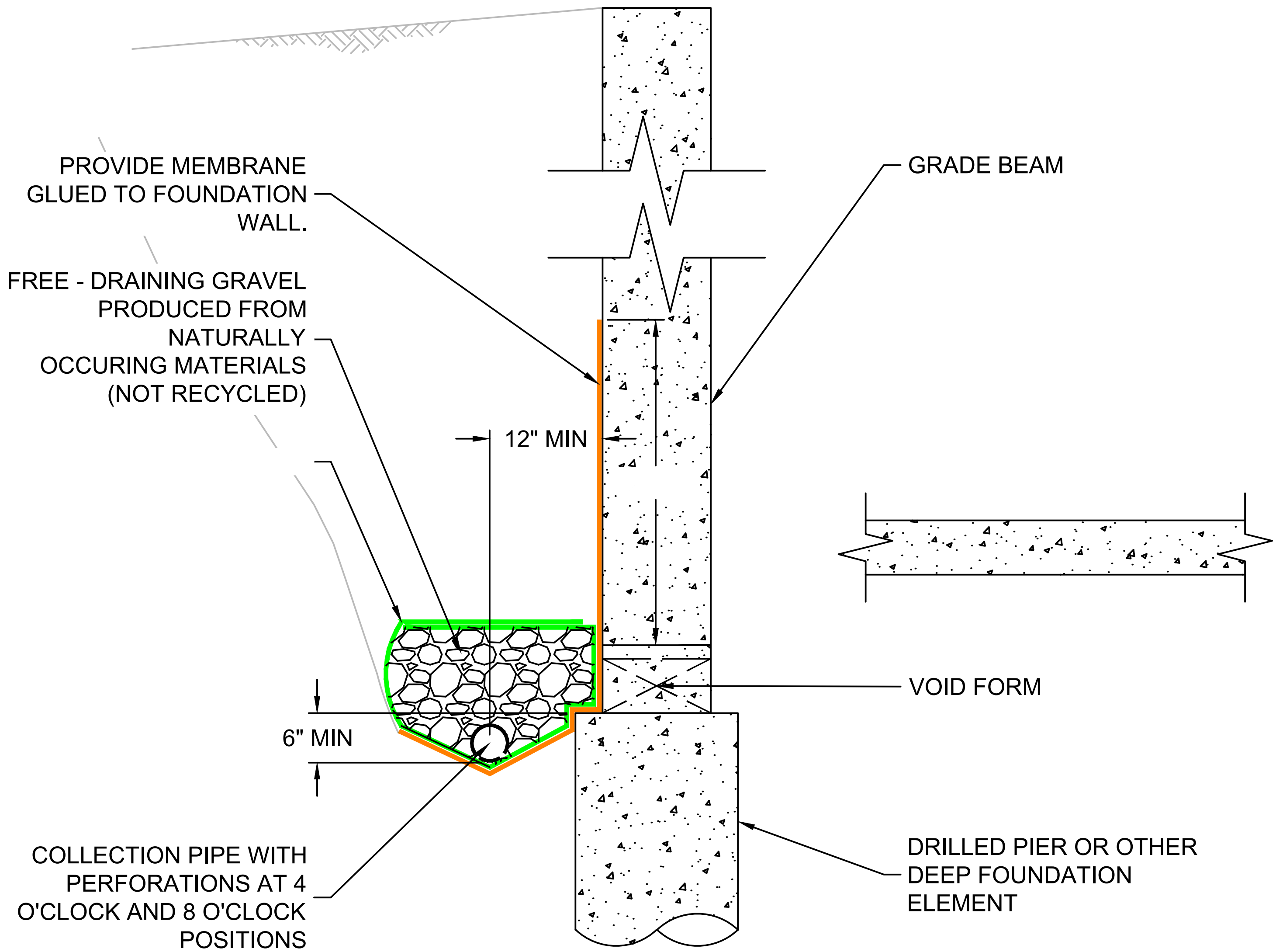
Pier / pile reductions should be interpolated from the graph above.

Figure to be reproduced in color for clarity.

GROUND ENGINEERING

JOB NO.:	23-8008
FIGURE:	13

LATERAL CAPACITY
REDUCTION FACTORS
FOR CLOSELY SPACED PIERS / PILES



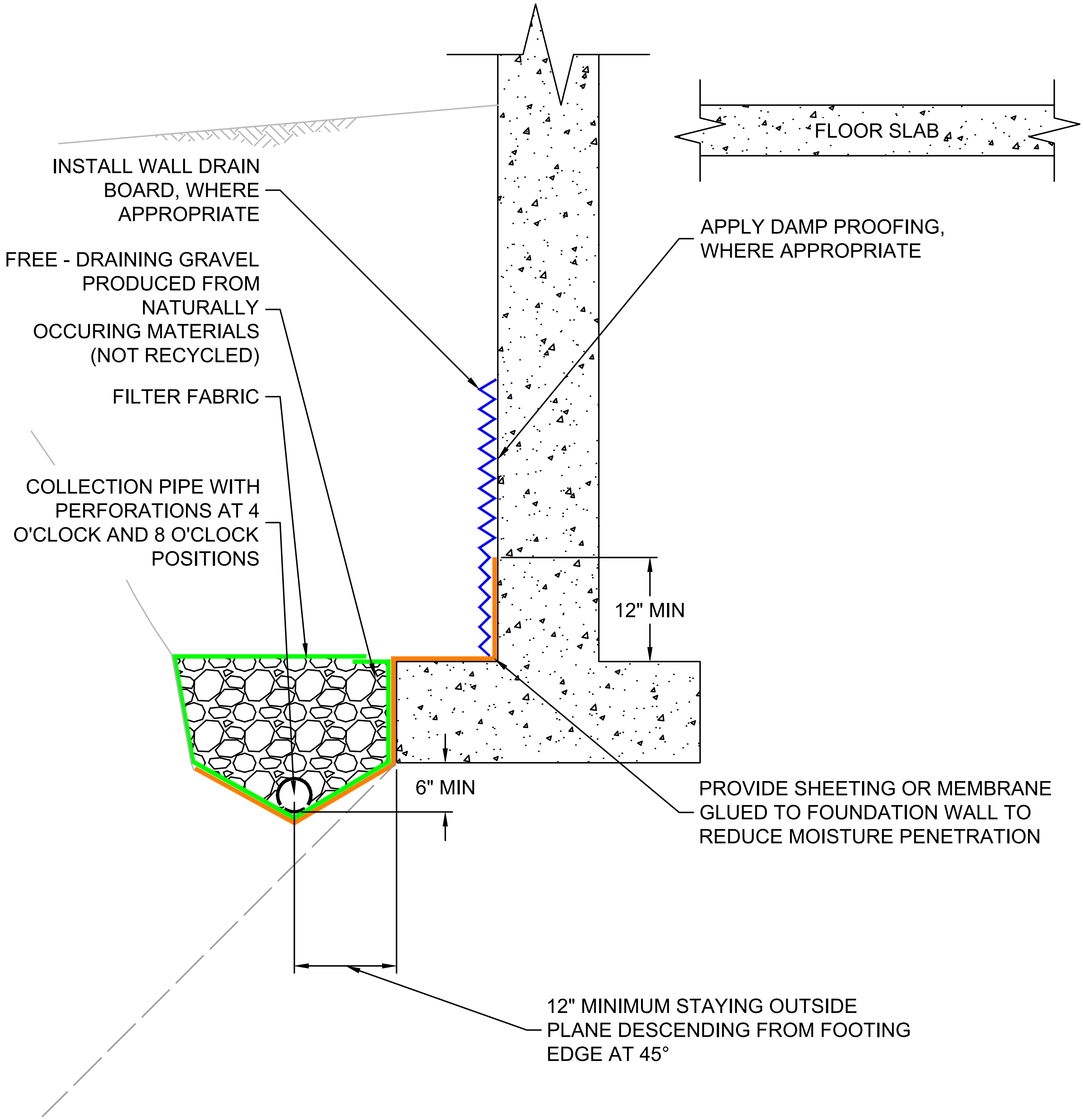
NOTES:

1. This is NOT a design - level drawing. it should be used solely for general information purposes only. Actual Underdrain design should be completed by others.
2. The underdrain system must be tested by the contractor after installation and backfilling to verify that it functions properly.
3. Inclusion of this figure in construction documents is done so at the document preparer's risk.
4. Reproduction of this document should be in color.

NOT TO SCALE
SEE TEXT FOR ADDITIONAL INFORMATION

GROUND ENGINEERING	JOB NO.: 23-8008
	FIGURE: 14

TYPICAL UNDERDRAIN DETAIL



NOTES:

1. This is NOT a design - level drawing. it should be used solely for general information purposes only. Actual Underdrain design should be completed by others.
2. The underdrain system must be tested by the contractor after installation and backfilling to verify that it functions properly.
3. Inclusion of this figure in construction documents is done so at the document preparer's risk.
4. Reproduction of this document should be in color.

NOT TO SCALE
SEE TEXT FOR ADDITIONAL INFORMATION

GROUND ENGINEERING	JOB NO.: 23-8008
	FIGURE: 15

TYPICAL UNDERDRAIN DETAIL

GROUND ENGINEERING

Meridian Ranch Field House

TABLE 1: SUMMARY OF LABORATORY TEST RESULTS

Sample Location		Natural	Natural	Gradation			Atterberg Limits		Swell/Consolidation		Unconfined Compressive Strength		USCS	AASHTO	
Test Hole No.	Depth (feet)	Moisture Content (%)	Dry Density (pcf)	Gravel (%)	Sand (%)	Fines (%)	Liquid Limit	Plasticity Index	Volume Change (%)	Surcharge Pressure (psf)	(psi)	(ksf)	Equivalent Classification	Equivalent Classification (Group Index)	Sample Description
1	2	7.2	-	11	85	4.2	21	1	-	-	-	-	SP	A-1-b (0)	FILL: Clayey Sand
1	17	10.8	115.2	0	53	47.0	26	9	-0.7	2,125	-	-	SC	A-4 (1)	SANDSTONE Bedrock
2	7	6.1	112.5	3	85	12.1	26	5	-	-	-	-	SC-SM	A-2-4 (0)	SANDSTONE Bedrock
2	9	7.8	120.2	3	81	15.6	31	9	-	-	-	-	SC	A-2-4 (0)	SANDSTONE Bedrock
2	19	-	-	-	-	-	-	-	-	-	31.5	4.54	-	-	SANDSTONE Bedrock
3	3	7.2	-	7	69	24.0	30	12	-	-	-	-	SC	A-2-6 (0)	Clayey SAND
3	8	5.3	-	9	80	10.9	25	4	-	-	-	-	SP-SC	A-1-b (0)	Weathered SANDSTONE
4	4	8.6	121.7	13	58	29.0	27	10	-0.6	500	-	-	SC	A-2-4 (0)	FILL: Clayey Sand
4	7	8.1	125.0	8	70	22.3	28	9	-	-	-	-	SC	A-2-4 (0)	FILL: Clayey Sand
5	4	9.6	SD	2	90	8.3	NV	NP	-	-	-	-	SP-SM	A-2-4 (0)	SAND with Silt
5	14	10.9	-	17	75	8.1	NV	NP	-	-	-	-	(SP-SM)g	A-1-b (0)	SANDSTONE Bedrock
5	19	9.2	-	0	61	38.6	24	7	-	-	-	-	SC	A-4 (0)	SANDSTONE Bedrock
6	3	11.0	117.5	6	61	33.0	28	9	-0.2	375	-	-	SC	A-2-4 (0)	Weathered SANDSTONE
6	13	10.1	-	1	86	13.5	NV	NP	-	-	-	-	SM	A-1-b (0)	SANDSTONE Bedrock
7	4	7.1	115.8	15	64	21.2	25	5	-0.5	500	-	-	(SC-SM)g	A-2-4 (0)	SANDSTONE Bedrock
7	29	12.3	-	1	78	21.3	31	11	-	-	-	-	SC	A-2-6 (0)	SANDSTONE Bedrock
8	2	9.3	117.3	4	73	22.8	25	3	-	-	-	-	SM	A-2-4 (0)	FILL: Silty Sand
8	12	12.5	122.3	0	35	64.6	34	13	-0.3	1,500	-	-	s(CL)	A-6 (7)	CLAYSOTNE Bedrock
9	8	12.8	117.2	0	32	67.7	33	14	1.7	1,000	-	-	s(CL)	A-6 (8)	Weathered CLAYSTONE
9	13	12.9	117.1	0	38	62.3	35	14	-0.4	1,625	-	-	s(CL)	A-6 (7)	CLAYSTONE Bedrock
IP-1	3	7.6	123.0	2	70	27.9	28	9	-	-	-	-	SC	A-2-4 (0)	FILL: Clayey Sand
IP-1	8	8.2	114.5	0	65	34.8	23	8	-	-	-	-	SC	A-2-4 (0)	SANDSTONE Bedrock
P-1	1	7.0	-	9	68	23.5	26	7	-	-	-	-	SC-SM	A-2-4 (0)	FILL: Silty, Claye Sand
P-2	2	4.8	-	6	85	9.3	25	8	-	-	-	-	SP-SC	A-2-4 (0)	FILL: Sand with Clay
P-3	4	6.8	123.1	14	59	27.2	27	8	-0.8	500	-	-	SC	A-2-4 (0)	FILL: Clayey Sand
P-4	4	6.2	-	8	74	18.4	29	9	-	-	-	-	SC	A-2-4 (0)	Clayey SAND
P-5	3	7.8	-	3	82	14.8	NV	NP	-	-	-	-	SM	A-2-4 (0)	FILL: Silty Sand
P-5	8	-	-	-	-	-	-	-	0.0	1,000	-	-	-	-	SANDSTONE Bedrock
Composite		-	-	2	69	29.3	27	8	-	-	-	-	SC	A-2-4 (0)	Clayey SAND

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

Job No. 23-8008

TABLE 2: SUMMARY OF SOIL CORROSION TEST RESULTS

Sample Location		Water Soluble Sulfates	pH	Redox Potential	Sulfide Reactivity	Resistivity	USCS Equivalent Classification	AASHTO Equivalent Classification (Group Index)	Sample Description
Test Hole No.	Depth (feet)	(%)		(mv)		(ohm-cm)			
9	3	0.03	7.0	- 6	Positive	1,812	SC	-	FILL: Clayey Sand
Composite		0.03	7.6	- 26	Positive	2,707	SC	A-2-4 (0)	Clayey SAND

Appendix A

Detailed Logs of the Test Holes

GROUND ENGINEERING

TEST HOLE 1

PROJECT: Meridian Ranch Field House **JOB NO:** 23-8008

CLIENT: Meridian Service Metro District **SITE LOCATION:** Falcon, CO

Elevation (ft)	Depth (ft)	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
7092	0		**TOPSOIL:** Approximately 4 inches of topsoil.										
			FILL: consisted of clayey and silty, fine to coarse sands and clays with gravels. They were moist to very moist, moderately plastic, very stiff to hard or medium dense to dense, and brown to dark brown to black in color.		10-12-11								
7087	5		**SANDS:** consisted of clean to clayey and silty, fine to coarse sands with gravels. Silt and clay beds and lenses were noted locally. They were very moist to wet, non- to slightly plastic, medium dense to dense, and brown in color.		50/8								
7082	10		**WEATHERED SANDSTONE:** consisted primarily of weathered, silty and clayey, fine to coarse sandstone with local weathered claystone. They were dry to moist, slightly to moderately plastic, weathered to firm, and gray to pale gray brown in color. Iron staining was noted locally.		50/5								
7077	15		**SANDSTONE BEDROCK:** consisted primarily of silty and clayey, fine to coarse sandstone with local claystone. They were dry to wet, non- to moderately plastic, hard to very hard, and gray to gray-brown in color. Iron staining was noted locally.		50/6								
7072	20		Groundwater encountered at 22 feet at the time of drilling.										
7067	25												
7062	30				45-46-50/6								

Bottom of test hole at approx. 34.5 feet.

TEST HOLE 2

GROUND ENGINEERING

PROJECT: Meridian Ranch Field House **JOB NO:** 23-8008

CLIENT: Meridian Service Metro District **SITE LOCATION:** Falcon, CO

Elevation (ft)	Depth (ft)	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
7095	0		**TOPSOIL:** Approximately 3 inches of topsoil.										
			FILL: consisted of clayey and silty, fine to coarse sands and clays with gravels. They were moist to very moist, moderately plastic, very stiff to hard or medium dense to dense, and brown to dark brown to black in color.										
7090	5				5-7-9								
			SANDS: consisted of clean to clayey and silty, fine to coarse sands with gravels. Silt and clay beds and lenses were noted locally. They were very moist to wet, non- to slightly plastic, medium dense to dense, and brown in color.										
			WEATHERED SANDSTONE: consisted primarily of weathered, silty and clayey, fine to coarse sandstone with local weathered claystone. They were dry to moist, slightly to moderately plastic, weathered to firm, and gray to pale gray brown in color.		50/8								
7085	10												
			SANDSTONE BEDROCK: consisted primarily of silty and clayey, fine to coarse sandstone with local claystone. They were dry to wet, non- to moderately plastic, hard to very hard, and gray to gray-brown in color. Iron staining was noted locally.		50/6								
7080	15												
			Groundwater encountered at 16 feet approximately 4 hours after drilling.										
			Groundwater encountered at 19 feet at the time of drilling.		50/6								
7075	20												
7070	25												
			Groundwater encountered at 27 feet approximately 1 days after drilling.										
					50/3								

Bottom of test hole at approx. 29.25 feet.

TEST HOLE 3

GROUND ENGINEERING

PROJECT: Meridian Ranch Field House **JOB NO:** 23-8008

CLIENT: Meridian Service Metro District **SITE LOCATION:** Falcon, CO

Elevation (ft)	Depth (ft)	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
7092	0		**TOPSOIL:** Approximately 4 inches of topsoil.										
			FILL: consisted of clayey and silty, fine to coarse sands and clays with gravels. They were moist to very moist, moderately plastic, very stiff to hard or medium dense to dense, and brown to dark brown to black in color.		8-10-14								
7087	5		**SANDS:** consisted of clean to clayey and silty, fine to coarse sands with gravels. Silt and clay beds and lenses were noted locally. They were very moist to wet, non- to slightly plastic, medium dense to dense, and brown in color.		25-35-45								
7082	10		**WEATHERED SANDSTONE:** consisted primarily of weathered, silty and clayey, fine to coarse sandstone with local weathered claystone. They were dry to moist, slightly to moderately plastic, weathered to firm, and gray to pale gray brown in color.										
			SANDSTONE BEDROCK: consisted primarily of silty and clayey, fine to coarse sandstone with local claystone. They were dry to wet, non- to moderately plastic, hard to very hard, and gray to gray-brown in color. Iron staining was noted locally.		50/6								
7077	15		Groundwater encountered at 16 feet at the time of drilling.										
					50/6								
7072	20												
					35-50/5								
7067	25												
					50/6								
7062	30												
7057	35												
					50/4								

Bottom of test hole at approx. 38.33 feet.

TEST HOLE 4

GROUND ENGINEERING

PROJECT: Meridian Ranch Field House **JOB NO:** 23-8008

CLIENT: Meridian Service Metro District **SITE LOCATION:** Falcon, CO

Elevation (ft)	Depth (ft)	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
7099	0		**TOPSOIL:** Approximately 3 inches of topsoil.										
			FILL: consisted of clayey and silty, fine to coarse sands and clays with gravels. They were moist to very moist, moderately plastic, very stiff to hard or medium dense to dense, and brown to dark brown to black in color.										
7094	5				35/12								
					47/12								
7089	10				14-9-5								
					5-12-19								
			WEATHERED SANDSTONE: consisted primarily of weathered, silty and clayey, fine to coarse sandstone with local weathered claystone. They were dry to moist, slightly to moderately plastic, weathered to firm, and gray to pale gray brown in color.										
7084	15				50/7								
			SANDSTONE BEDROCK: consisted primarily of silty and clayey, fine to coarse sandstone with local claystone. They were dry to wet, non- to moderately plastic, hard to very hard, and gray to gray-brown in color. Iron staining was noted locally.										
7079	20				50/3								
7074	25												
			Groundwater encountered at 27 feet at the time of drilling.										
					50/5								

Bottom of test hole at approx. 29.5 feet.

TEST HOLE 5

GROUND ENGINEERING

PROJECT: Meridian Ranch Field House **JOB NO:** 23-8008

CLIENT: Meridian Service Metro District **SITE LOCATION:** Falcon, CO

Elevation (ft)	Depth (ft)	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
7091	0		**TOPSOIL:** Approximately 4 inches of topsoil.										
			FILL: consisted of clayey and silty, fine to coarse sands and clays with gravels. They were moist to very moist, moderately plastic, very stiff to hard or medium dense to dense, and brown to dark brown to black in color.										
7086	5				20/12								
			SANDS: consisted of clean to clayey and silty, fine to coarse sands with gravels. Silt and clay beds and lenses were noted locally. They were very moist to wet, non- to slightly plastic, medium dense to dense, and brown in color.										
			WEATHERED SANDSTONE: consisted primarily of weathered, silty and clayey, fine to coarse sandstone with local weathered claystone. They were dry to moist, slightly to moderately plastic, weathered to firm, and gray to pale gray brown in color.		35-50/6								
7081	10												
			SANDSTONE BEDROCK: consisted primarily of silty and clayey, fine to coarse sandstone with local claystone. They were dry to wet, non- to moderately plastic, hard to very hard, and gray to gray-brown in color. Iron staining was noted locally.		43-50/6								
7076	15												
			Groundwater encountered at 16 feet at the time of drilling.										
7071	20				12-25-50/5								
7066	25				50/0								
7061	30												
					50/5								

Bottom of test hole at approx. 34.42 feet.

GROUND ENGINEERING

TEST HOLE 6

PROJECT: Meridian Ranch Field House **JOB NO:** 23-8008

CLIENT: Meridian Service Metro District **SITE LOCATION:** Falcon, CO

Elevation (ft)	Depth (ft)	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
7088	0		**TOPSOIL:** Approximately 6 inches of topsoil.										
			FILL: consisted of clayey and silty, fine to coarse sands and clays with gravels. They were moist to very moist, moderately plastic, very stiff to hard or medium dense to dense, and brown to dark brown to black in color.		34/12								
7083	5		**WEATHERED SANDSTONE:** consisted primarily of weathered, silty and clayey, fine to coarse sandstone with local weathered claystone. They were dry to moist, slightly to moderately plastic, weathered to firm, and gray to pale gray brown in color.		50/6								
7078	10		**SANDSTONE BEDROCK:** consisted primarily of silty and clayey, fine to coarse sandstone with local claystone. They were dry to wet, non- to moderately plastic, hard to very hard, and gray to gray-brown in color. Iron staining was noted locally.		50/6								
7073	15		Groundwater encountered at 17 feet approximately 4 hours after drilling.		45-50/6								
7068	20												
7063	25				50/4								

Bottom of test hole at approx. 28.33 feet.

GROUND ENGINEERING

TEST HOLE 7

PROJECT: Meridian Ranch Field House **JOB NO:** 23-8008

CLIENT: Meridian Service Metro District **SITE LOCATION:** Falcon, CO

Elevation (ft)	Depth (ft)	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
7094	0	**TOPSOIL:** Approximately 3 inches of topsoil.										
		FILL: consisted of clayey and silty, fine to coarse sands and clays with gravels. They were moist to very moist, moderately plastic, very stiff to hard or medium dense to dense, and brown to dark brown to black in color.										
7089	5			50/12								
		WEATHERED SANDSTONE: consisted primarily of weathered, silty and clayey, fine to coarse sandstone with local weathered claystone. They were dry to moist, slightly to moderately plastic, weathered to firm, and gray to pale gray brown in color.										
		SANDSTONE BEDROCK: consisted primarily of silty and clayey, fine to coarse sandstone with local claystone. They were dry to wet, non- to moderately plastic, hard to very hard, and gray to gray-brown in color. Iron staining was noted locally.		50/8								
7084	10											
7079	15			50/6								
		Groundwater encountered at 16. feet approximately 6 hours after drilling.										
7074	20			50/3								
		Groundwater encountered at 23 feet at the time of drilling.										
7069	25											
7064	30			50/6								
7059	35											
				50/4								

Bottom of test hole at approx. 39.33 feet.

TEST HOLE 8

GROUND ENGINEERING

PROJECT: Meridian Ranch Field House **JOB NO:** 23-8008

CLIENT: Meridian Service Metro District **SITE LOCATION:** Falcon, CO

Elevation (ft)	Depth (ft)	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
7091	0		**TOPSOIL:** Approximately 3 inches of topsoil.										
			FILL: consisted of clayey and silty, fine to coarse sands and clays with gravels. They were moist to very moist, moderately plastic, very stiff to hard or medium dense to dense, and brown to dark brown to black in color.		24/12								
7086	5		**WEATHERED SANDSTONE:** consisted primarily of weathered, silty and clayey, fine to coarse sandstone with local weathered claystone. They were dry to moist, slightly to moderately plastic, weathered to firm, and gray to pale gray brown in color.		50/10								
7081	10		**SANDSTONE BEDROCK:** consisted primarily of silty and clayey, fine to coarse sandstone with local claystone. They were dry to wet, non- to moderately plastic, hard to very hard, and gray to gray-brown in color. Iron staining was noted locally.		50/6								
7076	15		Groundwater encountered at 17 feet at the time of drilling.		50/4								
7071	20												
7066	25				50/6								
7061	30												
7056	35				50/5								

Bottom of test hole at approx. 37.42 feet.

GROUND ENGINEERING

TEST HOLE 9

PROJECT: Meridian Ranch Field House **JOB NO:** 23-8008

CLIENT: Meridian Service Metro District **SITE LOCATION:** Falcon, CO

Elevation (ft)	Depth (ft)	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
7090	0		**TOPSOIL:** Approximately 3 inches of topsoil.										
			FILL: consisted of clayey and silty, fine to coarse sands and clays with gravels. They were moist to very moist, moderately plastic, very stiff to hard or medium dense to dense, and brown to dark brown to black in color.		40/12								
7085	5		**WEATHERED SANDSTONE:** consisted primarily of weathered, silty and clayey, fine to coarse sandstone with local weathered claystone. They were dry to moist, slightly to moderately plastic, weathered to firm, and gray to pale gray brown in color.		39/12								
7080	10		**SANDSTONE BEDROCK:** consisted primarily of silty and clayey, fine to coarse sandstone with local claystone. They were dry to wet, non- to moderately plastic, hard to very hard, and gray to gray-brown in color. Iron staining was noted locally.		50/6								
7075	15				50/3								
7070	20		Groundwater encountered at 22 feet at the time of drilling.		50/3								
7065	25												
7060	30				50/6								

Bottom of test hole at approx. 33.5 feet.

TEST HOLE IP-1

PROJECT: Meridian Ranch Field House **JOB NO:** 23-8008

CLIENT: Meridian Service Metro District **SITE LOCATION:** Falcon, CO

Elevation *(ft)*	Depth *(ft)*	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content *(%)*	Natural Dry Density *(pcf)*	Percent Passing No. 200 Sieve	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	Swell/Consolidation (%) at Surcharge Pressure *(psf)*	Unconfined Compressive Strength *(ksf)*	USCS Equivalent Classification
7091	0		**TOPSOIL:** Approximately 3 inches of topsoil.										
			FILL: consisted of clayey and silty, fine to coarse sands and clays with gravels. They were moist to very moist, moderately plastic, very stiff to hard or medium dense to dense, and brown to dark brown to black in color.		38/12								
7086	5		**WEATHERED SANDSTONE:** consisted primarily of weathered, silty and clayey, fine to coarse sandstone with local weathered claystone. They were dry to moist, slightly to moderately plastic, weathered to firm, and gray to pale gray brown in color.		50/8								
7081	10		**SANDSTONE BEDROCK:** consisted primarily of silty and clayey, fine to coarse sandstone with local claystone. They were dry to wet, non- to moderately plastic, hard to very hard, and gray to gray-brown in color. Iron staining was noted locally.		50/6								
7076	15		Groundwater encountered at 17 feet at the time of drilling.		50/6								

Bottom of test hole at approx. 19.5 feet.

TEST HOLE P-1

GROUND ENGINEERING

PROJECT: Meridian Ranch Field House **JOB NO:** 23-8008

CLIENT: Meridian Service Metro District **SITE LOCATION:** Falcon, CO

Elevation *(ft)*	Depth *(ft)*	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content *(%)*	Natural Dry Density *(pcf)*	Percent Passing No. 200 Sieve	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	Swell/Consolidation (%) at Surcharge Pressure *(psf)*	Unconfined Compressive Strength *(ksf)*	USCS Equivalent Classification
7099	0												
			FILL: consisted of clayey and silty, fine to coarse sands and clays with gravels. They were moist to very moist, moderately plastic, very stiff to hard or medium dense to dense, and brown to dark brown to black in color.		7-8-10								
7094	5				13/12								

Bottom of test hole at approx. 6 feet.

GROUND ENGINEERING

TEST HOLE P-2

PROJECT: Meridian Ranch Field House **JOB NO:** 23-8008

CLIENT: Meridian Service Metro District **SITE LOCATION:** Falcon, CO

Elevation (ft)	Depth (ft)	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
7085	0		**TOPSOIL:** Approximately 4 inches of topsoil.										
			FILL: consisted of clayey and silty, fine to coarse sands and clays with gravels. They were moist to very moist, moderately plastic, very stiff to hard or medium dense to dense, and brown to dark brown to black in color.		19-20-18								
7080	5												
			SANDS: consisted of clean to clayey and silty, fine to coarse sands with gravels. Silt and clay beds and lenses were noted locally. They were very moist to wet, non- to slightly plastic, medium dense to dense, and brown in color.		50/9								

Bottom of test hole at approx. 7.75 feet.

TEST HOLE P-3

GROUND ENGINEERING

PROJECT: Meridian Ranch Field House **JOB NO:** 23-8008

CLIENT: Meridian Service Metro District **SITE LOCATION:** Falcon, CO

Elevation (ft)	Depth (ft)	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
7090	0												
			FILL: consisted of clayey and silty, fine to coarse sands and clays with gravels. They were moist to very moist, moderately plastic, very stiff to hard or medium dense to dense, and brown to dark brown to black in color.										
7085	5				33/12								

Bottom of test hole at approx. 5 feet.

GROUND ENGINEERING

TEST HOLE P-4

PROJECT: Meridian Ranch Field House **JOB NO:** 23-8008

CLIENT: Meridian Service Metro District **SITE LOCATION:** Falcon, CO

Elevation (ft)	Depth (ft)	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
7079	0		**TOPSOIL:** Approximately 5 inches of topsoil.										
			FILL: consisted of clayey and silty, fine to coarse sands and clays with gravels. They were moist to very moist, moderately plastic, very stiff to hard or medium dense to dense, and brown to dark brown to black in color.		13-15-16								
7074	5		**SANDS:** consisted of clean to clayey and silty, fine to coarse sands with gravels. Silt and clay beds and lenses were noted locally. They were very moist to wet, non- to slightly plastic, medium dense to dense, and brown in color.		20-20-25								

Bottom of test hole at approx. 5.5 feet.

TEST HOLE P-5

GROUND ENGINEERING

PROJECT: Meridian Ranch Field House **JOB NO:** 23-8008

CLIENT: Meridian Service Metro District **SITE LOCATION:** Falcon, CO

Elevation (ft)	Depth (ft)	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits		Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
									Liquid Limit	Plasticity Index			
7083	0		**TOPSOIL:** Approximately 3 inches of topsoil.										
			FILL: consisted of clayey and silty, fine to coarse sands and clays with gravels. They were moist to very moist, moderately plastic, very stiff to hard or medium dense to dense, and brown to dark brown to black in color.		4-4-4								
7078	5		**SANDS:** consisted of clean to clayey and silty, fine to coarse sands with gravels. Silt and clay beds and lenses were noted locally. They were very moist to wet, non- to slightly plastic, medium dense to dense, and brown in color.		50/10								
			WEATHERED SANDSTONE: consisted primarily of weathered, silty and clayey, fine to coarse sandstone with local weathered claystone. They were dry to moist, slightly to moderately plastic, weathered to firm, and gray to pale gray brown in color.										
			SANDSTONE BEDROCK: consisted primarily of silty and clayey, fine to coarse sandstone with local claystone. They were dry to wet, non- to moderately plastic, hard to very hard, and gray to gray-brown in color. Iron staining was noted locally.										

Bottom of test hole at approx. 8.83 feet.