Geotechnical Evaluation
Centura Parker NHC
Parker, Colorado

Prepared For:

Portercare Adventist Health System

9100 East Mineral Circle Wadsworth Boulevard
Centennial, Colorado 80112

Attention: Ms. Erin Ward

Job Number: 22-3043 **November 14, 2022**

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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 design of the proposed medical office building to be constructed at Lot H of the Parker Pointe development in Parker, Colorado. Our study was conducted in general accordance with GROUND's Proposal Number 2206-1066 Revised2 dated July 5, 2022 between Portercare Adventist Health System and GROUND.

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 on-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 Portercare Adventist Health System, 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 the provided request for proposal (PS NHC Geotech RFP 060122 revised.pdf, prepared and provided to GROUND by MedDevelopment, LLC), we understand that present plans call for a two-story office building with a footprint area of about 16,000 square feet. Additionally, paved parking areas, local landscaping, and new underground utilities are also planned for construction. Structural loads are anticipated to be relatively low to moderate, typical of this type of construction. No-below grade levels are planned at this time.

Grade changes up to about 2 to 5 feet, and retaining walls are also 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 re-evaluate the conclusions and parameters presented herein.

Performance Expectations Based on our experience with other, similar projects, we understand that post-construction, building foundation and floor movements on the order of 1 inch are acceptable to, and anticipated by Portercare Adventist Health System as are 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 and ancillary structures, although movement estimates closer to 1 inch may be preferable near the buildings. Assuming that traffic speeds will be relatively low, still greater movements (3+ inches locally) are acceptable and anticipated for the parking area, driveway, and circulation road pavements, 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 site generally consisted of undeveloped land vegetated with short to tall grasses and weeds, sparce deciduous trees, and scattered succulents. The site was bordered to the west by South Parker Road, and to the north, south, and east by undeveloped land. Additional undeveloped land, single-family residences, and commercial developments further surrounded the site.

The site was located south of an approximately 30-foot, steeply sloped hill on relatively flat ground, with approximately 6 feet of relief across the site. Prairie dog mounds were observed commonly across the project site. Construction debris and trash were scattered across the site. Older metal, wood, and silt fencing were present. Marks for underground utilities were observed on the west side of the site, adjacent to the existing roadway and overhead power lines. Wooden debris were scattered across the east side of the site, likely from the removal of an approximately 400-foot stand of deciduous trees.

Review of historical aerial imagery available on Google Earth indicated that the project site for the proposed Building H has been relatively unchanged since the earliest available images in 1993. Although not within this specific project area, a building appears to have been demolished on the northern margin of the development. Additionally, Kinney Creek traverses through the southern margin of the project site.

SUBSURFACE EXPLORATION

Subsurface exploration for the project was conducted in July 2022. A total of 5 test holes were drilled with a conventional, track-mounted drilling rig advancing 4-inch diameter, solid stem augers to evaluate the subsurface conditions and retrieve samples for laboratory testing. Two of the test holes were advanced within the approximate building footprint area to depths of about 40 and 48 feet below existing grade. One of the test holes was drilled along the proposed retaining wall alignment to a depth of about 14 feet below existing grade. The remaining two test holes were advanced within the areas proposed for paving to depths of about 5 and 9 feet below existing grade. A GROUND engineer 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 1⅜-inch inner-diameter standard penetration sampler. The samplers were driven into the substrata with blows from a 140-pound hammer falling 30 inches. This procedure is similar to 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. Logs of the test holes are presented in Figure 2. A legend and notes are provided in Figure 3. Detailed logs are provided in *Appendix A*.

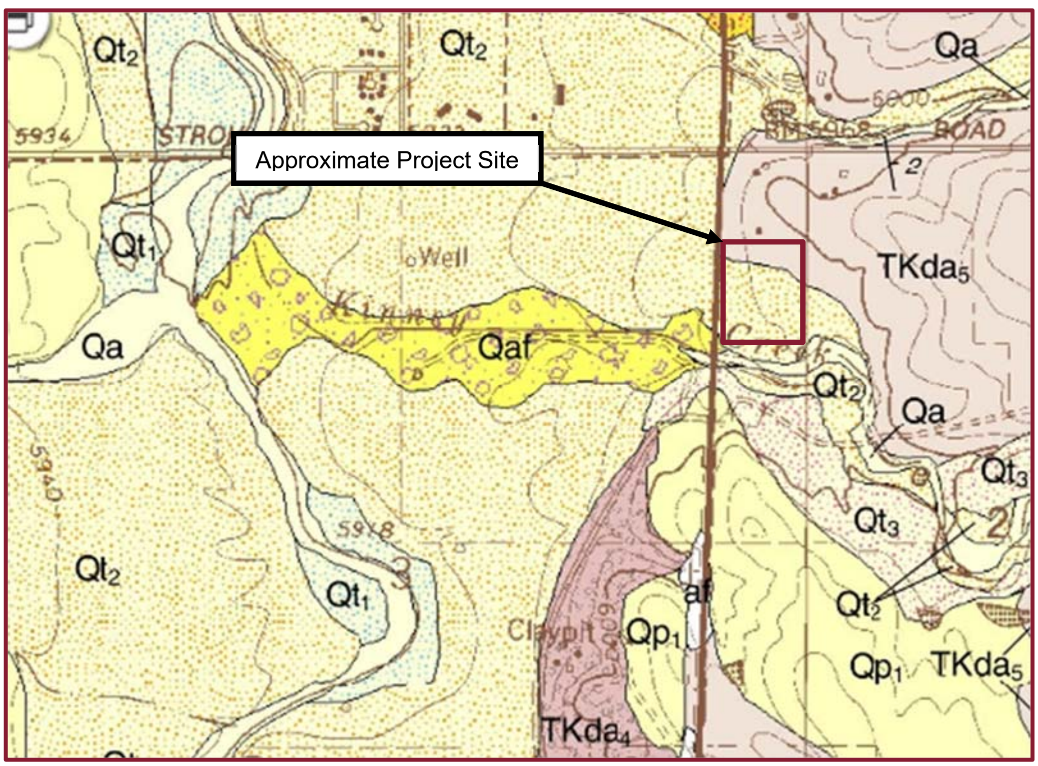
Test Hole 2 was completed as a temporary groundwater monitoring and observation hole to facilitate additional depth groundwater measurements. A diagram of the monitoring and observations hole and subsequent water readings are provided in Figure 4.

LABORATORY TESTING

Samples retrieved from our 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, chlorides, 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. A gradation plot is provided in Figure 5.

SUBSURFACE CONDITIONS

Geologic Setting Published geologic maps, e.g., Thorson (2005),[1] depict the site as underlain by late Pleistocene Terrace Alluvium (**Qt**). These surficial deposits are mapped as being underlain by the early Paleocene Facies Unit Four of the Dawson formation (**Tkda**). A portion of the that map is reproduced below.



[1] Thorson, J.P., 2005, Geologic Map of the Castle Rock North Quadrangle, Douglas County, Colorado, U.S. Geological Survey, Open-File Report OF05-02, 1:24,000.

In the project area, alluvial (stream-laid) deposits typically consist of sands and gravels with varying fractions of silts and clays. Cobbles and boulders are present locally as well. Some of the larger clasts present in alluvial deposits may not be appropriate for reuse in project fills.

The Facies Unit 4 of the Dawson Formation, in the project area, includes arkostic conglomerate and sandstone that may include cobbles up to 8 inches in size. Layers of siltstone and claystone are occasionally interbedded. The siltstones and claystones can be moderately to highly expansive. The formation includes well-cemented beds that can be very hard and difficult to excavate.

Local Conditions In general the test holes penetrated about 2 to 5 inches of topsoil[2] before penetrating native, interbedded clays, silts, and sands that were recognized to a depth of about 42 feet below existing grade. Claystone, siltstone, and sandstone bedrock were encountered beneath the native soils and extended to the depths explored. The upper several feet of the bedrock were relatively severely weathered,

We interpret the native clays and silts to alluvial deposits, and the claystone, siltstone, and sandstone bedrock to be Dawson Formation materials.

Fill materials were not recognized in the test holes, but likely are present across the northern margin of the site, given the past demolition of existing structures. (See the *Site Conditions* section of this report.) These fill soils may contain coarse gravels and cobbles, as well as similarly sized or larger pieces of construction, debris even though these items 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.

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

Clays, Silts, and Sands consisted of clays, silts, and fine to medium sands. They were non- to highly plastic, soft to stiff and very loose to medium dense, dry to wet, and pale brown to dark brown to gray in color. Caliche and iron staining were noted locally.

Claystone, Siltstone, and Sandstone Bedrock consisted of interbedded claystones, siltstones, and sandstones. They were slightly to highly plastic, medium hard to hard, moist to very moist, and pale brown to dark gray in color. Iron staining and lignite were noted locally.

Groundwater was encountered at the time of drilling at depths of about 19 to 20 feet below existing grades. The test holes were backfilled upon drilling completion per Code of Colorado Regulations (2 CCR 402-2). Groundwater levels can be expected to fluctuate, however, in response to annual and longer-term cycles of precipitation, irrigation, surface drainage, nearby rivers and creeks, land use, and the development of transient, perched water conditions. At this site we anticipate that groundwater levels will tend to rise and fall in conjunction with nearby waterways, such as the Cherry Creek and Kinney Creek.

A monitoring and observation hole was installed in Test Hole 2 in order to provide subsequent groundwater measurements, at the request of Portercare Adventist Health System. During the time of drilling, the water level was measured at 19 feet below existing grade. An additional measurement, recorded one day after drilling, depicted the groundwater level as being 15 feet below existing grade. Another measurement was recorded on September 1, 2022 depicting the groundwater level as being 14 feet below existing grade.

Swell-Consolidation Testing of a selected sample of on-site soils recovered from the test holes indicated a swell of about 0.4 percent and consolidations of about 0.5 and 0.6 percent under surcharge load approximating in-place overburden pressure. (See Table 1.)

SEISMIC CLASSIFICATION

Based on extrapolation of available data to depth and our experience in the project area, we consider the area of the proposed addition likely to meet the criteria for a Seismic Site Classification of **E** according to the ASCE 7-10 (Table 1613.5.5). (Exploration and/or shear wave velocity testing to a depth of 100 feet or more was not part of our present scope of services.) If, however, a quantitative assessment of the site seismic properties is desired, then shear wave velocity testing should be performed. GROUND can provide a fee estimate for shear wave velocity testing upon request. We consider the likelihood of achieving a Site Class D to be low.

Using longitude and latitude coordinates obtained from Google Earth and the ASCE 7 Hazard Tool (https://asce7hazardtool.online/), the project area is indicated to possess an S_{DS} value of **0.290** and an S_{D1} value of **0.134** for the site latitude and longitude and a Site Class of E.

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 Portercare Adventist Health System, upon receipt of this report, to discuss the risks and the geotechnical information presented in this report with us. In addition to the risks and remedial approaches presented in this report, Portercare Adventist Health

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

General Geotechnical Risk In GROUND's opinion, there are several sources of geotechnical risk at this site. The primary geotechnical risk is the relatively low penetration resistance values in the native soils. Blow counts ranging from 0 blows for 12 inches to 20 blows for 12 inches were recorded in the soils at depths of 5 to 40 feet below existing grades. Materials with such low blow counts typically do not provide adequate bearing support for improvements like the planned building without excessive settlements.

Another potential source of geotechnical risk at this site is the risk of heave in the clayey site soils and bedrock. Significant swells and consolidations have been measured in similar soils, but swell-consolidation testing of the on-site materials were relatively low, less than 1 percent. This suggests that the risk of heave and/or consolidation at this site is lower than other similar sites nearby.

As the risk of heave at this site appears to be relatively low, it is GROUND's opinion that the underlying native soils appeared to be geotechnically suitable to support relatively lightly load improvements without unusual risk. As discussed above, the native site soils, in their current state, appear to be unable to support the anticipated loads for the proposed medical office building. We estimate that post-construction movements on the order of 1 to 3 inches are likely with differential movements of similar magnitudes, where improvements are supported directly on the existing site soils. The amount of movement experienced will depend significantly on the foundation loads applied.

Approaches that can reduce the estimates of post-construction movements are presented below.

Building Foundation and Floor Types In GROUND's opinion, supporting the proposed building on drilled pier or driven pile foundation systems 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 foundation systems may not be practical because they are not needed to support the structural loads and bedrock was encountered at relatively great depths, about 42 feet below existing grade in Test Hole 1 and greater than 40 feet below existing grade in Test Hole 2. Geotechnical parameters for drilled pier or driven pile foundations can be provided upon request.

Constructing the building floor as a structural floor, also supported on drilled piers or driven piles, 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 also can be provided upon request.

As a higher risk, but commonly used alternative, shallow foundations and slab-on-grade floors or a mat foundation could be used at this site. Shallow foundations could bear directly on the existing site soils, a remedial fill section, or soils improved by rammed aggregate piers, as discussed below. Allowable bearing capacity for each bearing condition are tabulated below, as well.

ALLOWABLE BEARING CAPACITIES FOR SHALLOW FOUNDATIONS

Bearing Condition	***Allowable Bearing Capacity***	***Maximum Footing Width***	***Estimated Post-Construction Movements***
Existing Native Soils	1,000 psf	6 feet	1 inch
Remedial Fill Section (*14 feet of Select Granular Fill*)	1,500 psf	6 feet	1 inch
Soils Improved By Rammed Aggregate Piers	> 1,500 psf	Determined By Designer Installer	Determined By Designer Installer

Due to the relatively low penetration resistance values, the site soils likely will not be able to provide sufficient bearing for the proposed construction. Where a greater bearing capacity is needed, a remedial fill section could be constructed. Such a remedial fill section should extend to a depth of at least 14 feet below existing grade and consist of select, granular fill.

We understand, however, that constructing such a remedial fill section may not be practical. As an alternative to a remedial fill section, rammed aggregate piers could be used to improve the site soils and increase the allowable bearing capacity while reducing estimates of post-construction movements. The allowable bearing capacity, number, and depth/length of the individual elements, are determined by the specialty designer installer. The data in this report should be sufficient for the designer/installer to provide their design, but GROUND should be contacted if additional geotechnical data is needed. Rammed aggregate piers have been used successfully in the greater project area to support similar structures and appear to be the most efficient option for supporting the proposed medical office building.

For additional information, we suggest contacting a qualified and experienced designer/installer of these systems. We suggest contacting the following firms for additional information, though others may be available:

- Ground Improvement Engineering 816 – 421 – 4334
- Keller (Hayward Baker) 303 – 469 – 1136

Additional, geotechnical parameters for shallow foundation design are presented in the *Shallow Foundations* section of this report. Geotechnical parameters for mat foundation design are presented in the *Mat Foundation* section of this report. Likely post-construction movements for the building foundations described in the *Shallow Foundation* and *Mat Foundation* sections are estimated to be on the order of 1 inch. Differential movements, likely will be on the order of ½ inch over spans of 40 feet. More detailed geotechnical parameters for design of shallow foundations and slab-on-grade floors are provided in subsequent sections of this report.

In general, we anticipate that the majority of the existing site soils will be geotechnically suitable to be reused as fill. Additional parameters and considerations regarding the suitability of the existing site soils are provided in the *Project Earthwork* section of this report.

SHALLOW FOUNDATIONS

The geotechnical parameters below may be used for design of shallow, footing type foundations for the proposed building. Parameters and considerations for shallow, mat type foundations are provided in the *Mat Foundations* section of this report.

Geotechnical Parameters for Shallow Foundation Design

1) Footings should bear on the existing native soils, a remedial fill section. or soils improved by rammed aggregate piers as discussed in the *Geotechnical Considerations for Design* section of this report.

2) The allowable bearing capacity should be selected based on the bearing condition as discussed in the *Geotechnical Considerations for Design* section of this report.

   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 above 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 buildings at this site. 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.

   Where rammed aggregate piers are used to improve the site soils, an allowable bearing pressure provided by the rammed aggregate pier designer/installer should be used.

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.

Shallow Foundation Construction

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

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

10) 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.

11) 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.

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

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

MAT FOUNDATION

The geotechnical parameters below may be used for design of a mat foundation for the proposed building. For shallow, footing type foundations, see the *Shallow Foundations* section of this report.

Geotechnical Parameters for Mat Foundation Design

1) A shallow mat foundation should bear on a section of the existing native soils or soils improved by rammed aggregate piers.

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

2) A mat bearing on firm native soils or properly compacted fill may be designed for an allowable soil bearing pressure of **500 psf** for a mat up to **200 feet in width**. GROUND estimates that post-construction movements associated with this bearing capacity will be approximately 1 inch.

   The selected bearing pressure value may be increased by ⅓ for transient loads such as wind or seismic loading.

   Where rammed aggregate piers are used to improve the site soils, an allowable bearing capacity provided by the rammed aggregate pier designer/installer should be used.

   To reduce differential settlements across the foundation, foundation loads should be as uniform as possible. Differentially loaded footings will settle differentially.

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

3) An allowable vertical modulus of subgrade reaction (**Kv**) of **30 tcf** (35 pci) may be used for design of a concrete, mat foundation bearing on firm native soils or a properly compacted fill section as discussed in the *Geotechnical Considerations for Design* section of this report.

Where rammed aggregate piers are used to improve the site soils, an allowable vertical modulus of subgrade reaction provided by the rammed aggregate pier designer/installer should be used.

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

4) The mat 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.

Interior partitions resting on mat 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 1½ inches or more of vertical, differential movement. Accommodation for differential movement also should be made where partitions meet bearing walls.

SLAB-ON-GRADE FLOOR

The geotechnical parameters below may be used for design of slab-on-grade floors for the proposed buildings. 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 the existing native soils or soils improved by rammed aggregate piers.

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 **30 tcf** (35 pci) may be used for design of a concrete, slab-on-grade floor bearing on firm, native soils or a fill section of site derived soils. Where 3 or more feet of select, granular fill are present beneath a slab-on-grade floor, then a modulus of subgrade reaction (Kv) of 175 tcf (202 pci) may be used.

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

   Where rammed aggregate piers are used to improve the site soils, an allowable vertical modulus of subgrade reaction provided by the rammed aggregate pier designer/installer should be used.

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 **1½ inches or more** of vertical, differential movement. Accommodation for differential movement also should be made where partitions meet bearing walls.

7) Post-construction heave 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**. CDOT Class 5 or 6 Aggregate Base Course, or similar material may be substituted for the free-draining gravel.

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

Per the 2006 ACI *Location Guideline*, a vapor barrier is required under concrete floors when that floor is to receive moisture-sensitive floor covering and/or adhesives, or the room above that floor has humidity control.

Therefore, in light of the several, potentially conflicting effects of the use vapor-barriers, the owner and the architect and/or contractor should weigh the performance of the slab and appropriate flooring products in light of the intended building use, etc., during the floor system design process and the selection of flooring materials. Use of a plastic vapor-barrier membrane may be appropriate for some building areas and not for others.

In the event a vapor barrier is utilized, 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 E-96 or ASTM F-1249, and comply with ASTM E-1745 (Class “A”). Vapor barriers should be installed in accordance with ASTM E-1643.

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

Construction Considerations for Slab-on-Grade Floors

10) 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.

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.

LATERAL LOADS

Values for equivalent fluid pressures and the coefficient for frictional resistance to sliding are provided below. These values were based on moist unit weight (γ') of 125 pcf and an angle of internal friction (ϕ) of 24 degrees for the site soils and are unfactored. Appropriate factors of safety should be included in design calculations.

Shallow Elements Resisting Lateral Loads A friction coefficient of **0.30** 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 **260 pcf** for drained conditions, to a **maximum of 2,600 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 **75 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 **53 pcf** for the drained condition may be used.

WATER-SOLUBLE SULFATES

The concentration of water-soluble sulfates measured in a selected sample of site soils was approximately less than 0.03 percent by weight. (See Table 2.) Such a concentration of soluble sulfates represents a **negligible** environment for sulfate attack on concrete exposed to these materials. Degrees of attack are based on the scale of "negligible," "moderate," "severe" and "very severe" as described in the "Design and Control of Concrete Mixtures," published by the Portland Cement Association (PCA). The Colorado Department of Transportation (CDOT) utilizes a corresponding scale with four classes of severity of sulfate exposure (Class 0 to Class 3) as described in the table below.

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

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

Based on our test results and PCA and CDOT guidelines, appropriate cement conforming to one of the following requirements should be used in all concrete exposed to site soils and bedrock:

<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 a red-ox potential of approximately -37 millivolts. Such a low potential typically creates a more corrosive environment.

Sulfide Reactivity testing indicated "negative" result in the local soils. The absence of sulfides in the soils suggests a less 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 a value of approximately 3,187 ohm-centimeters in a sample of site soils.

Chlorides testing indicated a chloride concentration of less than 10 parts per million.

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.[3] Our testing indicated a pH value of about 7.9.

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[3] 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, reproduced below, used to predict corrosivity. The scale is intended for protection of ductile iron pipe but is valuable for project steel selection. At 10 points or higher, protective measures for ductile iron pipe are indicated. 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 moderately corrosive environment for ferrous metals (6 points).

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 and low or negative redox-potential results (< 50 mV) are obtained, add three (3) points for this range.

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

PROJECT EARTHWORK

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

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

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

Topsoil and other organic materials present on-site should not be incorporated into ordinary fills. Instead, topsoil should be stockpiled during initial grading operations for placement in areas to be landscaped or for other approved uses. These materials should be removed and replaced where fill will be placed above them or where they will be beneath a proposed improvement.

The animal burrows observed on the site should be collapsed and backfilled during project grading. This may entail excavation and backfilling to greater depths than otherwise called for. Where burrows are not remediated effectively, additional local settlements in excess of those discussed herein can develop.

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 re-use as compacted fill.

Fragments of crystalline 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. Fragments of claystone, siltstone, and sandstone must be broken down to a soil like mass, however.

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 **40 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.

Select, Granular Fill Material to be imported to the site as select, granular fill should meet the criteria for CDOT Class 1 Structure Backfill (tabulated below).

CDOT CLASS 1 STRUCTURE BACKFILL

Sieve Size or Parameter	Acceptable Range
2-inch	100% passing
No. 4	30% to 100% passing
No. 50	10% to 60% passing
No. 200	5% to 20% passing
Liquid Limit	$\leq$ 35
Plasticity Index	$\leq$ 6

Materials proposed for retaining wall backfill and/or select granular fill should be tested and approved for use a retaining wall backfill prior to import to the site.

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

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

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

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 re-work areas affected by the formation of ice to re-establish 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 to 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 settlement of the existing soils left in place. For a 6-foot fill, for example, that corresponds to a total settlement of about 1 inch. 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, unretained, graded slopes supported by local soils up to **10 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 re-established. 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 slope-specific stability analyses 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. Therefore, in general, we anticipate no unusual excavation difficulties in these materials, in general, for the proposed construction with conventional, heavy duty, excavating equipment.

Although not encountered in the test holes, in our experience in the project vicinity, beds and lenses of well cemented sandstones may be present locally within the bedrock. These beds and lenses can be very hard and resistant to excavation or, for example, to advance drilled pier holes through. The contractor should be prepared to excavate, handle, process, and, if necessary, export such materials. The use of very heavy-duty excavation equipment, e.g., a Caterpilar D10 or other larger dozer with a single-shank ripper, may be necessary, if highly cemented bedrock is encountered. We understand, however, that project excavations are not anticipated to be advanced into site bedrock at this time.

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 Portercare Adventist Health System, 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, un-shored excavation slopes up to **10 feet** in height, in general, should be cut no steeper than **1½ : 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.

Groundwater Groundwater was encountered in the test holes at depths of approximately 19 to 20 feet below existing grades at the time of drilling. Upon further measurements, taken one day after drilling, the water was measured at 16 feet below existing grade. The groundwater level was measured a third time at 14 feet below existing grade on September 1, 2022. Therefore, based on conditions at the time of this subsurface exploration, relatively shallow excavations at the site appear unlikely to encounter groundwater except, limited volumes of perched groundwater.

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.

Surface Water The contractor should take pro-active 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.

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 + 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 re-excavation 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 the facility owner.

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

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

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

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

If 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 the shallow on-site soils may 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. Maintenance should be anticipated to include removal and replacement of sidewalk stones, curb and gutter, sections of pavement, etc., to restore effective drainage. 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 and maintained 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") can be beneficial for collecting and discharging saturated subsurface waters. Although the subsurface drainage system anticipated for this project may consist of perimeter underdrains along the building perimeter and underdrains constructed beneath floor system, they are addressed as underdrains herein.

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 it will be beneficial to include a perimeter underdrain system to help limit wetting of the foundation bearing soils. However, we understand that the owner and project team may consider that the reduction of risk provided by a properly constructed and maintained underdrain system does not justify the costs associated with including an underdrain. In such a case, an underdrain system can be excluded. If an underdrain system is excluded, then there will be an increased risk of the likely post-construction movements estimated in this report being exceeded. GROUND considers this increase in risk to be low, but it is not zero. Where an underdrain system is excluded, additional 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.

Where a below grade level is added, an underdrain system should be included. If a below-grade level will underlie only a portion of the building footprint, then the underdrain system could be local to that area. 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 (some) below-grade elements may be appropriate, depending on the intended use.

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

Geotechnical Parameters for Underdrain Design Where underdrains are included as a part of facility drainage design, underdrain design should incorporate the parameters below. The actual underdrain layout, 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 6 & 7.

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 **10 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-2-4 and A-2-6 soils with group index values of 0 in accordance with the AASHTO classification system. Such soils generally provide fair to good subgrade support.

A resilient modulus value of 4,500 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. 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 36,500 (corresponding to an EDLA value of 5 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 10 for a 20-year design life) was assumed for the parking lot and individual building driveways. An ESAL of 365,000 (corresponding to an EDLA value of 50 for a 20-year design life) was estimated for the heavy-duty pavements (i.e., heavy truck routes, loading and unloading areas, trash collection routes, etc.). If design traffic loadings differ significantly from these assumed values, GROUND should be notified to re-evaluate 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)*
Light Vehicle Parking Stalls	**5½**	**4 / 6**	**6 / 6**
Light Vehicle Drive Lanes	**6**	**4 / 8**	**6 / 6**
Heavy Truck Traffic	**-**	**-**	**7 / 6**

Truck routes, truck loading and unloading areas, trash collection areas, fire access roads, and other pavement areas subjected to high turning stresses or heavy truck traffic should be provided with rigid pavements consisting of **7 or more inches** of portland cement concrete underlain by **6 inches** of properly compacted CDOT Class 6 Aggregate Base Course. A theoretically equivalent flexible pavement section for these areas would be 5 inches of asphalt over 10 inches of aggregate base course. However, in our experience, asphalt pavements will not perform as well as rigid pavement in areas of repeated turning stresses or static loading. Pushing, rutting, and tearing of the asphalt should be anticipated, with local ponding of water, and additional maintenance costs should be anticipated if this section 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. We consider the following gradations and binders to be appropriate, though 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.

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. Aggregate composed of recycled asphalt should not be expected to provide the same support for the wearing course as native, Class 6 material and should not be considered as an equivalent for it. Our experience suggests that recycled asphalt is difficult to compact properly when placed and can hold water after the wearing course is placed on it. Additionally, a base course like material was identified in the test holes. This material may need processing to meet the requirements of CDOT Class 6 or other applicable aggregate base course criteria.

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. 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 distress pavement areas.

Remedial Earthwork 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 24 inches or a depth that removes and replaces all soft, wet, otherwise unsuitable soils, whichever is greater.** 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 remove and replace all the undocumented fill soils or soft, yielding, or otherwise deleterious soils as properly compacted fill due the presence of utility lines and the proximity of existing improvements. 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 the building's floor is desired, then project pavements should be constructed in a similar manner as the buildings' floor.

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. The contractor should be prepared to either dry the subgrade materials or moisten them, as needed, prior to compaction.

<u>Proof Rolling</u> 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. <u>Establishment of a firm paving platform (as indicated by proof rolling) is an additional requirement beyond proper fill placement and compaction</u>. 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 compacted to at least 95 percent of the maximum modified Proctor dry density. 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 re-grading and application of additional aggregate should be implemented to ensure proper drainage, repair distressed/damaged areas, and re-establish 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 slab-on-grade floor in a similar manner to project floors. 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.

   Regardless of the depth of subgrade preparation, due to the potentials for settlement and heave in the undocumented fill soils at this site, greater than typical maintenance, including the removal and replacement of portions of flatwork, should be anticipated for project exterior flatwork. Greater depths of subgrade preparation will tend to reduce the extent and frequency of extra maintenance, however.

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

Limitations This report has been prepared for Portercare Adventist Health System, as it pertains to design and construction of the proposed dentist office 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 2023. Any changes in project plans or schedule should be brought to the attention of a geotechnical engineer, in order that the geotechnical conclusions in this report may be re-evaluated and, as necessary, modified.

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.

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 re-evaluation 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 Portercare Adventist Health System 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.

If any information referred to herein is not well understood, then Portercare Adventist Health System, or other members of the design team, or the facility owner, 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. Re-use 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 Portercare Adventist Health System with a proposal for construction observation and materials testing.

Sincerely,

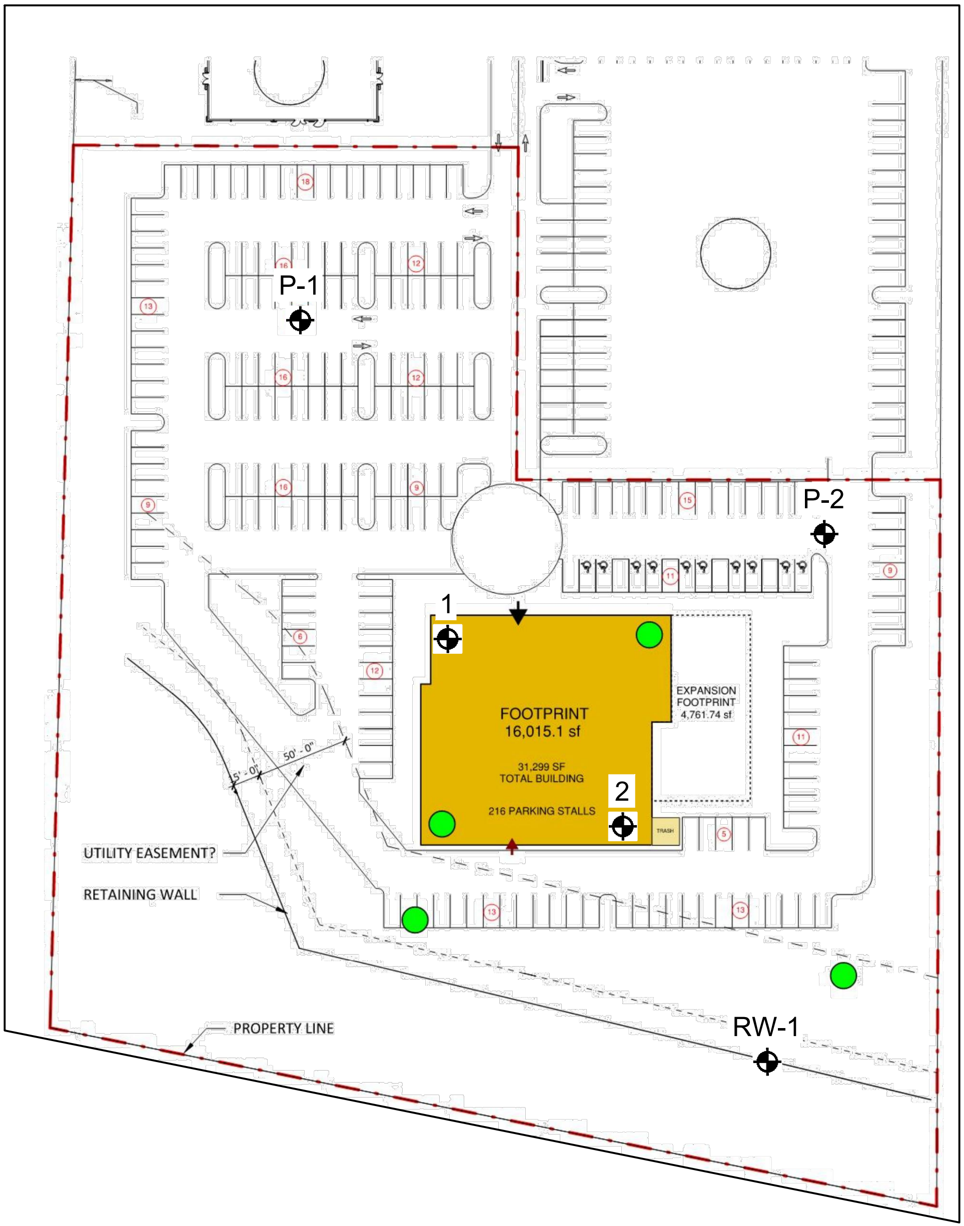
GROUND Engineering Consultants, Inc.

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

COLORADO LICENSED BRIAN H. RECK 48265 PROFESSIONAL ENGINEER

11/14/2022

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



1

Indicates test hole number and approximate location.

NOT TO SCALE

SITE PLAN PROVIDED BY CLIENT

GROUND ENGINEERING

JOB NO.: 22-3043

FIGURE: 1

LOCATION OF TEST HOLES

LOGS OF THE TEST HOLES

GROUND ENGINEERING

CLIENT: Portercare Adventist Health Systems

JOB NO: 22-3043

PROJECT NAME: Centura Parker NHC

PROJECT LOCATION: Parker, CO

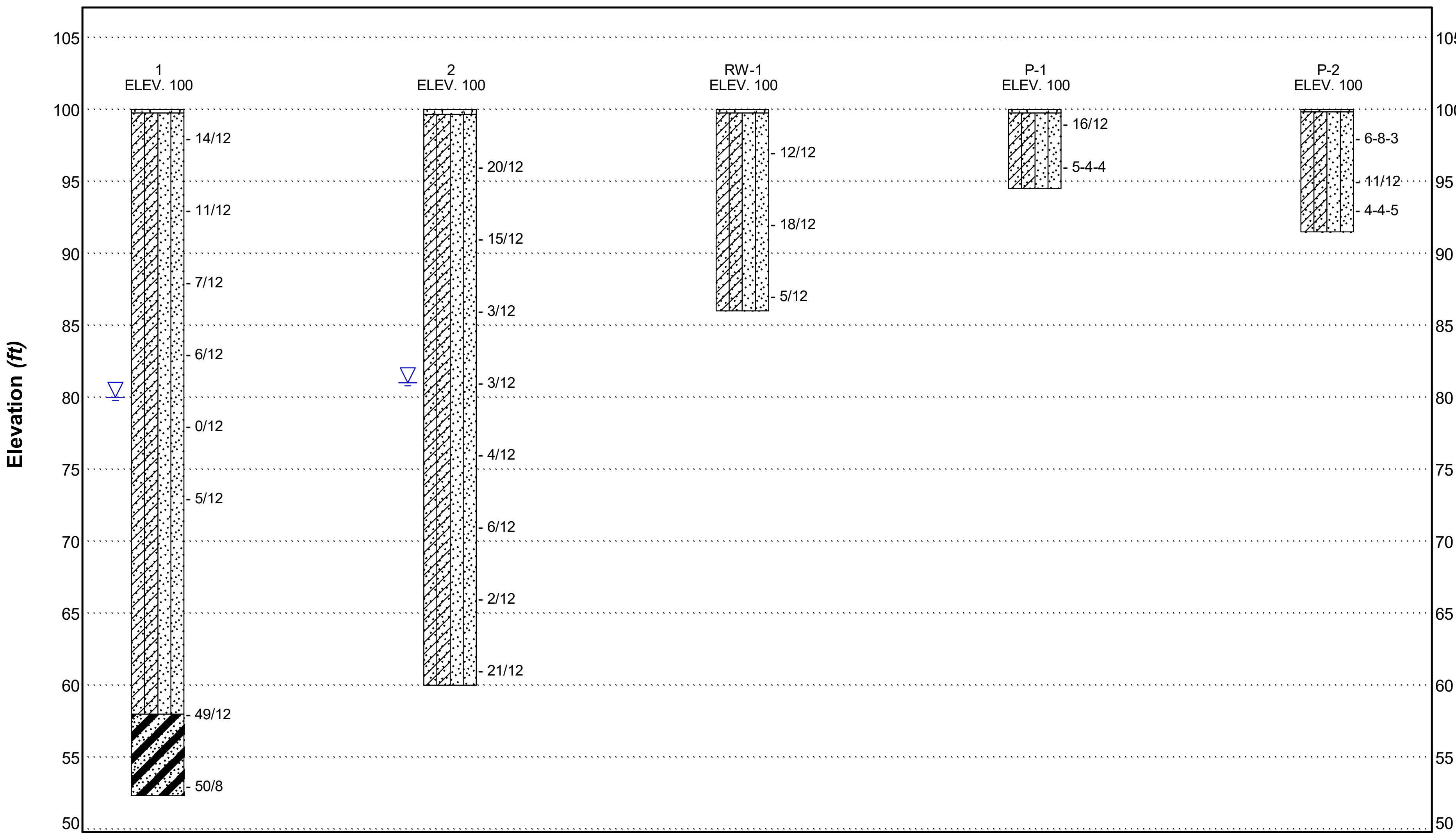


Figure 2

LEGEND AND NOTES

GROUND ENGINEERING

CLIENT: Portercare Adventist Health Systems
PROJECT NAME: Centura Parker NHC
JOB NO: 22-3043
PROJECT LOCATION: Parker, CO

MATERIAL SYMBOLS

TOPSOIL

SAND, CLAY, and SILT

SILTSTONE, CLAYSTONE, and SANDSTONE BEDROCK

NOTE: See Detailed Logs for Material descriptions.

SAMPLER SYMBOLS

Modified California Liner Sampler
23 / 12 Drive sample blow count indicates 23 blows of a 140 pound hammer falling 30 inches were required to drive the sampler 12 inches.

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

NOTES

1. Test holes were drilled on 7/27/2022 with 4" solid stem auger.

2. Locations of the test holes were determined approximately by pacing from features shown on the site plan provided.

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

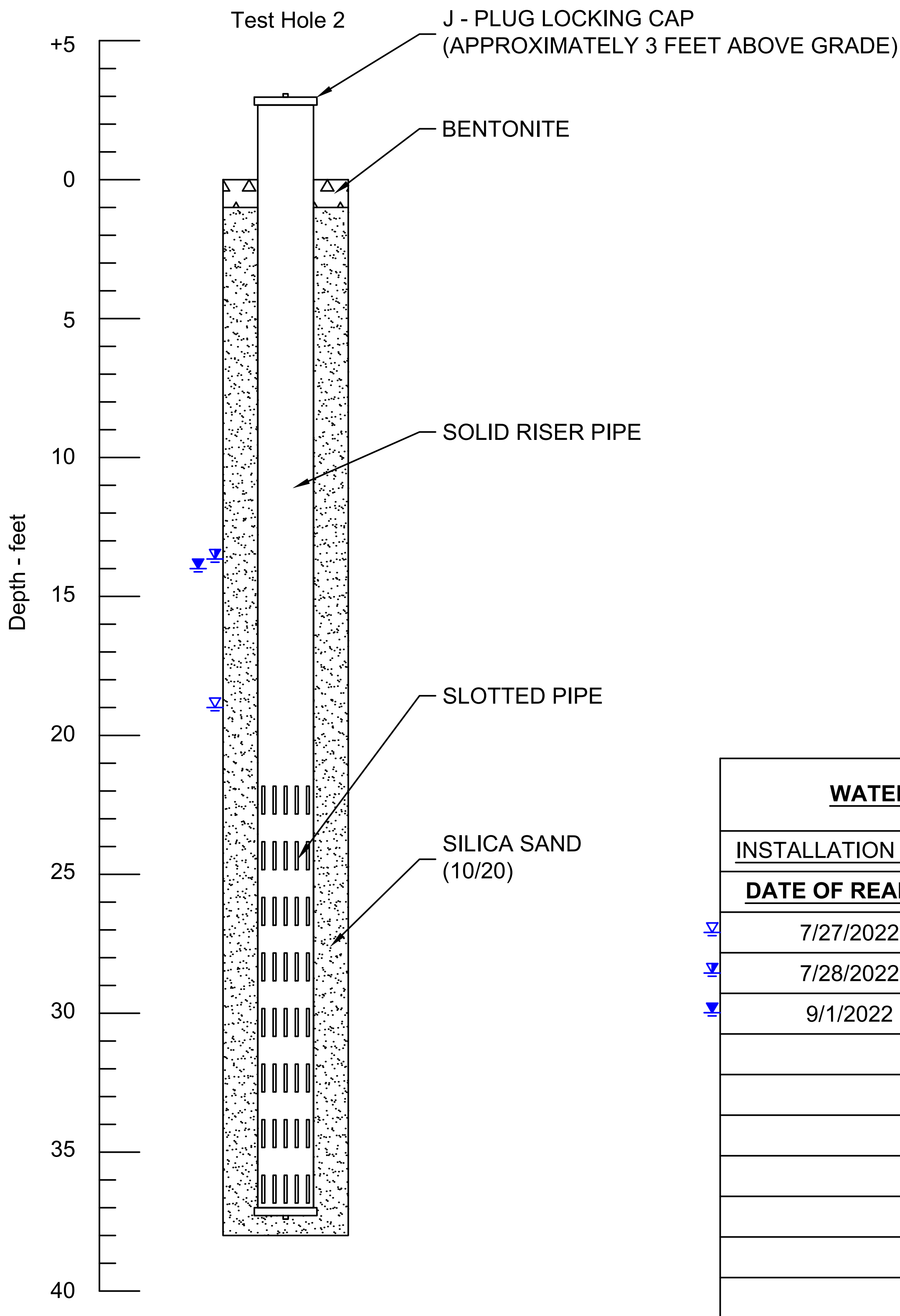
▽ Water Level at Time of Drilling, or as Shown

▼ Water Level at End of Drilling, or as Shown

▼ Water Level After 24 Hours, or as Shown

NV	No Value
NP	Non-Plastic

Figure 3



WATER READINGS	
INSTALLATION DATE:	7/27/2022
DATE OF READING	**DEPTH**
7/27/2022	19'
7/28/2022	13.7'
9/1/2022	14'

NOTES:

1. Bore hole is approximately 4 inches in diameter.
2. PVC is schedule 40 and 2 inches in diameter slots for slotted PVC are 0.010" wide.

GROUND ENGINEERING

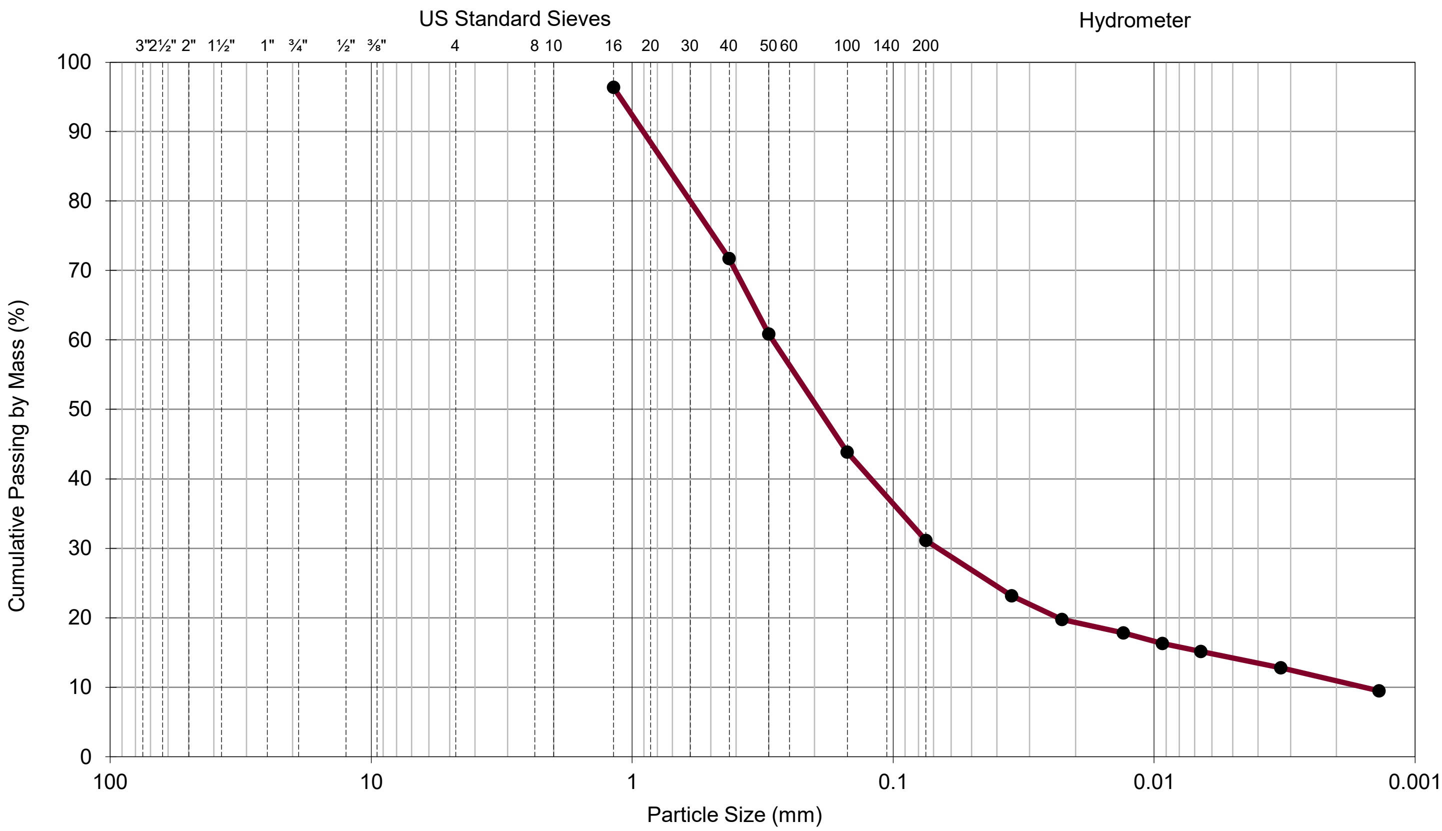
JOB NO.: 22-3043

FIGURE: 4

MONITORING AND OBSERVATION HOLE ASSEMBLY

Centura Parker NHC

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



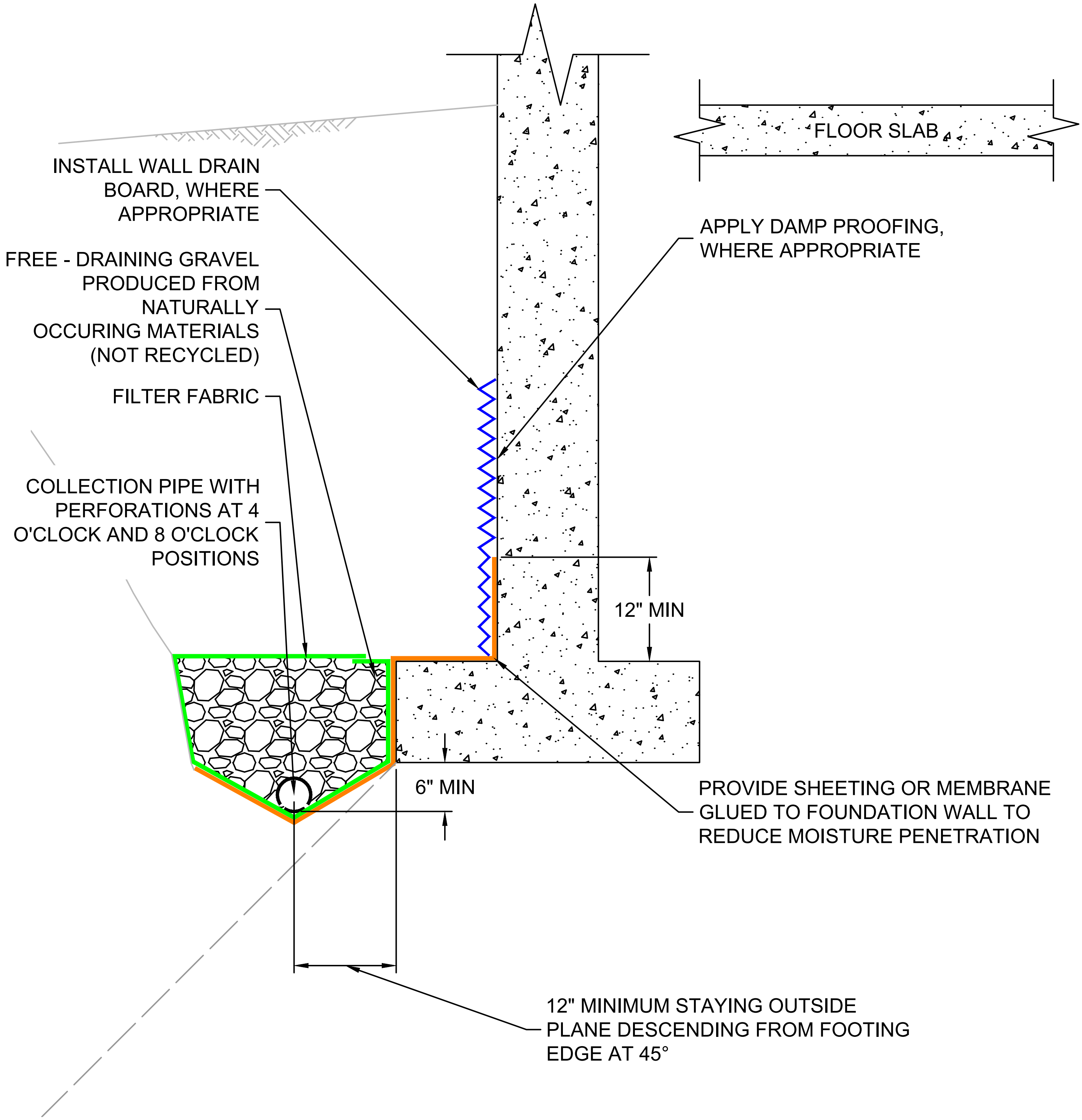
Coarse Gradation			Fine Gradation			Hydrometer		Grading	
US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	Particle Size (mm)	Passing by Mass (%)	Coefficient	Value
6 in	150	-	No. 4	4.75	-	0.035	23	D90	0.907
5 in	125	-	No. 8	2.36	-	0.023	20	D85	0.737
4 in	100	-	No. 10	2.00	-	0.013	18	D80	0.599
3 in	75	-	No. 16	1.18	96	0.009	16	D60	0.290
2.5 in	63	-	No. 20	0.85	-	0.007	15	D50	0.193
2 in	50	-	No. 30	0.60	-	0.003	13	D40	0.122
1.5 in	37.5	-	No. 40	0.425	72	0.001	9	D30	-
1 in	25.0	-	No. 50	0.300	61	-	-	D15	-
3/4 in	19.0	-	No. 60	0.250	-	-	-	D10	-
1/2 in	12.5	-	No. 100	0.150	44	-	-	D05	-
3/8 in	9.5	-	No. 140	0.106	-	-	-	Cu	-
No. 4	4.75	-	No. 200	0.075	31.1	-	-	Cc	-

Location: RW-1 at 8 feet
Description: Clayey, Silty SAND

Classification: SC-SM / A-2-4 (0)
Liquid Limit: 21
Plasticity Index: 7
Activity: 0.6

Gravel (%): #N/A
Sand (%): #N/A
Silt/Clay (%): 31
< .002 mm (%): 11

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.



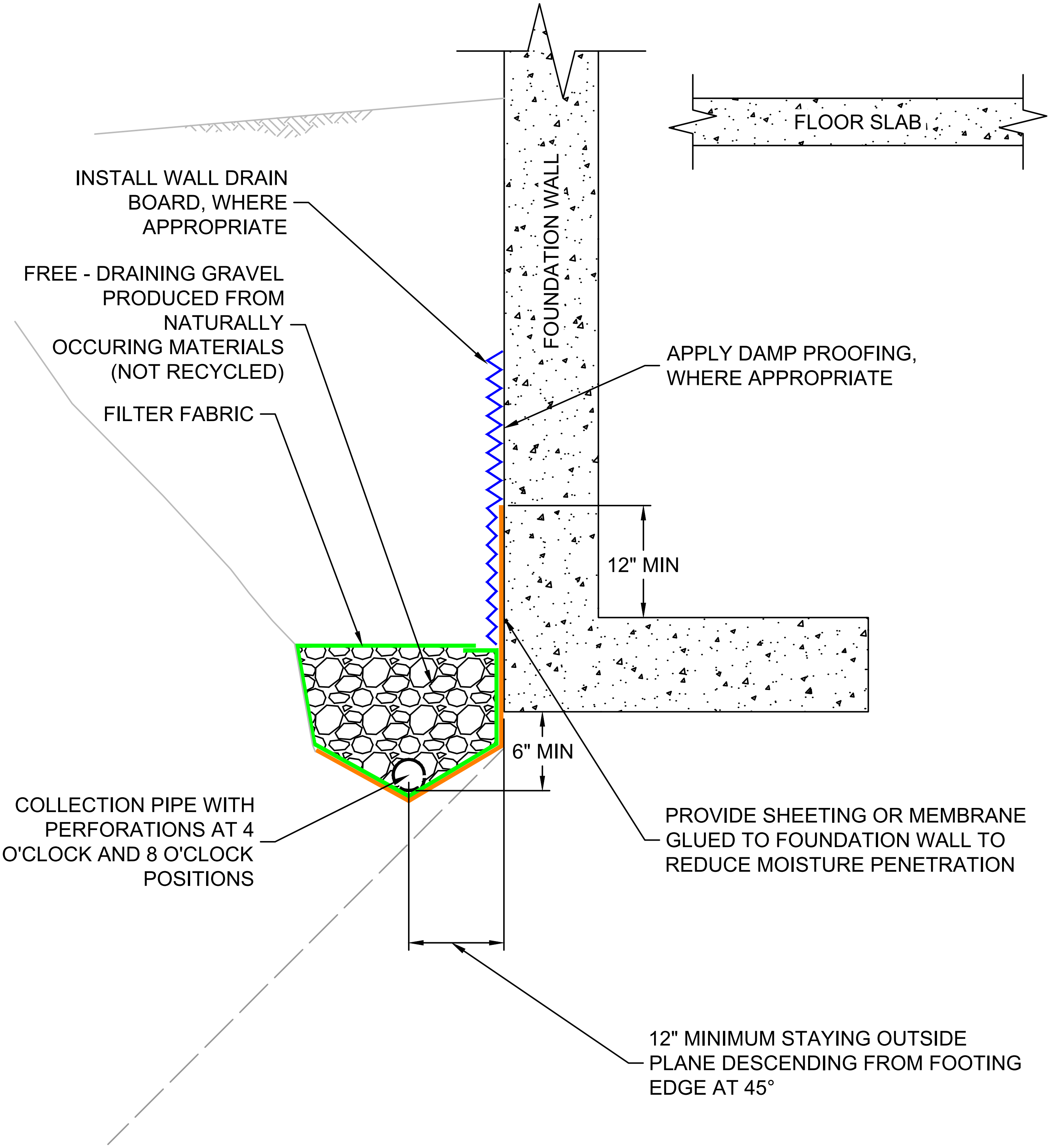
NOTES:

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 SCLE
SEE TEXT FOR ADDITIONAL INFORMATION

GROUND ENGINEERING	JOB NO.: 22-3043
	FIGURE: 6

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.
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4. Reproduction of this document should be in color.

NOT TO SCLE
SEE TEXT FOR ADDITIONAL INFORMATION

GROUND ENGINEERING	JOB NO.: 22-3043
	FIGURE: 7

TYPICAL UNDERDRAIN DETAIL

GROUND ENGINEERING

Centura Parker NHC

TABLE 1: SUMMARY OF LABORATORY TEST RESULTS

Sample Location		Natural Moisture Content	Natural Dry Density	Gradation			Atterberg Limits		Swell/Consolidation		Unconfined Compressive Strength		USCS Equivalent Classification	AASHTO Equivalent Classification (Group Index)	Sample Description
Test Hole No.	Depth *(feet)*	*(%)*	*(pcf)*	Gravel *(%)*	Sand *(%)*	Fines *(%)*	Liquid Limit	Plasticity Index	Volume Change *(%)*	Surcharge Pressure *(psf)*	*(psi)*	*(ksf)*			
1	7	3.5	SD	7	77	16.1	NV	NP	-	-	-	-	SM	A-2-4 (0)	Silty SAND
1	12	17.4	98.4	0	47	53.0	26	6	-0.6	1500	-	-	s(CL-ML)	A-4 (1)	Sandy, Silty CLAY
1	42	33.3	82.1	0	42	57.6	57	17	-	-	30.1	4.33	s(MH)	A-7-5 (9)	SILTSTONE Bedrock
2	4	3.2	108.2	1	77	21.8	21	6	-	-	-	-	SC-SM	A-2-4 (0)	Clayey, Silty SAND
2	14	30.6	88.6	0	29	71.3	39	13	-0.5	1750	-	-	(ML)s	A-6 (9)	Sandy SILT
2	19	37.7	81.6	0	12	88.0	43	19	-	-	-	-	CL	A-7-6 (18)	CLAY
RW-1	3	6.3	98.1	0	66	33.8	26	7	0.4	350	-	-	SC-SM	A-2-4 (0)	Clayey, Silty SAND
RW-1	8	7.2	106.6	0	69	31.1	21	7	-	-	-	-	SC-SM	A-2-4 (0)	Clayey, Silty SAND
RW-1	13A	30.5	93.8	0	43	57.1	35	13	-	-	4.2	0.6	s(CL)	A-6 (5)	Sandy CLAY
P-1	1	2.2	101.7	1	86	12.9	NV	NP	-	-	-	-	SM	A-2-4 (0)	Silty SAND
P-2	2	5.9	SD	2	72	25.8	33	13	-	-	-	-	SC	A-2-6 (0)	Clayey SAND

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

Job No. 22-3043

TABLE 2: SUMMARY OF SOIL CORROSION TEST RESULTS

Sample Location		Water Soluble Sulfates	pH	Redox Potential	Sulfide Reactivity	Resistivity	Chlorides	USCS Equivalent Classification	AASHTO Equivalent Classification (Group Index)	Sample Description
Test Hole No.	Depth *(feet)*	*(%)*		*(mv)*		*(ohm-cm)*	*(ppm)*			
2	4	0.03	7.9	- 37	Negative	3,187	< 10	SC-SM	A-2-4 (0)	Clayey, Silty SAND

Appendix A

Detailed Logs of the Test Holes

GROUND ENGINEERING

TEST HOLE 1

CLIENT: Portercare Adventist Health Systems

PROJECT NAME: Centura Parker NHC

JOB NO: 22-3043

PROJECT LOCATION: Parker, 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
100	0	**TOPSOIL**										
		Clays, Silts, and Sands: Clays, silts, and fine to medium sands. They were non- to highly plastic, soft to stiff and very loose to medium dense, dry to wet, and pale brown to dark brown to gray in color. Caliche and iron staining were noted locally.		14/12								
95	5											
				11/12	3.5		16	NV	NP			SM
90	10											
				7/12	17.4	98.4	53	26	6	-0.6 (1500)		s(CL-ML)
85	15											
				6/12								
80	20 ▽	Groundwater encountered at 20 feet at time of drilling.										
				0/12								
75	25											
				5/12								
70	30											
65	35											
60	40											
		Claystone, Siltstone, and Sandstone Bedrock: Interbedded claystones, siltstones, and sandstones. They were slightly to highly plastic, medium hard to hard, moist to very moist, and pale brown to dark gray in color. Iron staining and lignite were noted locally.		49/12	33.3	82.1	58	57	17		4.33	s(MH)
55	45											
				50/8								

Bottom of borehole at Approx. 47.67 feet.

GROUND ENGINEERING

TEST HOLE 2

CLIENT: Portercare Adventist Health Systems

PROJECT NAME: Centura Parker NHC

JOB NO: 22-3043

PROJECT LOCATION: Parker, 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
100	0		**TOPSOIL**										
			Clays, Silts, and Sands: Clays, silts, and fine to medium sands. They were non- to highly plastic, soft to stiff and very loose to medium dense, dry to wet, and pale brown to dark brown to gray in color. Caliche and iron staining were noted local										
95	5				20/12	3.2	108.2	22	21	6			SC-SM
90	10				15/12								
85	15				3/12	30.6	88.6	71	39	13	-0.5 (1750)		(ML)s
80	20		Groundwater encountered at 19 feet at time of drilling.		3/12	37.7	81.6	88	43	19			CL
75	25				4/12								
70	30				6/12								
65	35				2/12								
60	40				21/12								

Bottom of borehole at Approx. 40 feet.

TEST HOLE P-1

GROUND ENGINEERING

CLIENT: Portercare Adventist Health Systems

PROJECT NAME: Centura Parker NHC

JOB NO: 22-3043

PROJECT LOCATION: Parker, 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
100	0	**TOPSOIL**										
		Clays, Silts, and Sands: Clays, silts, and fine to medium sands. They were non- to highly plastic, soft to stiff and very loose to medium dense, dry to wet, and pale brown to dark brown to gray in color. Caliche and iron staining were noted local		16/12	2.2	101.7	13	NV	NP			SM
95	5			5-4-4								

Bottom of borehole at Approx. 5.5 feet.

ENGINEERING

TEST HOLE P-2

CLIENT: Portercare Adventist Health Systems

PROJECT NAME: Centura Parker NHC

JOB NO: 22-3043

PROJECT LOCATION: Parker, 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
100	0												
			TOPSOIL										
			Clays, Silts, and Sands: Clays, silts, and fine to medium sands. They were non- to highly plastic, soft to stiff and very loose to medium dense, dry to wet, and pale brown to dark brown to gray in color. Caliche and iron staining were noted local		6-8-3	5.9		26	33	13			SC
95	5				11/12								
					4-4-5								

Bottom of borehole at Approx. 8.5 feet.

TEST HOLE RW-1

GROUND ENGINEERING

CLIENT: Portercare Adventist Health Systems

PROJECT NAME: Centura Parker NHC

JOB NO: 22-3043

PROJECT LOCATION: Parker, 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
100	0		**TOPSOIL**										
			Clays, Silts, and Sands: Clays, silts, and fine to medium sands. They were non- to highly plastic, soft to stiff and very loose to medium dense, dry to wet, and pale brown to dark brown to gray in color. Caliche and iron staining were noted local										
					12/12	6.3	98.1	34	26	7	0.4 (350)		SC-SM
95	5												
					18/12	7.2	106.6	31	21	7			SC-SM
90	10												
					5/12								

Bottom of borehole at Approx. 14 feet.