Geotechnical Subsurface Exploration Program
PAC Dental at Highway 7 & Sheridan Parkway
Erie, Colorado

Prepared for:
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Job Number: 16-3717

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PURPOSE AND SCOPE OF STUDY

This report presents the results of a subsurface exploration program performed by GROUND Engineering Consultants, Inc. (GROUND) for the proposed PAC Dental building that will be constructed northwest of the intersection of Highway 7 and Sheridan Boulevard in Erie, Colorado. Our study was conducted in general accordance with GROUND’s Proposal No. 1611-2223, dated November 22, 2016.

Field and office studies provided information regarding surface and subsurface conditions, including existing site vicinity improvements and groundwater. Material samples retrieved during the subsurface exploration were tested in our laboratory to assess the engineering characteristics of the site earth materials. Results of the field, office, and laboratory studies for the proposed facility are presented below.

This report has been prepared to summarize the data obtained and to present our conclusions and opinions based on the proposed construction and the subsurface conditions encountered. Design parameters and a discussion of engineering considerations related to construction of the proposed facility are included herein.

PROPOSED CONSTRUCTION

We understand that proposed construction will consist of a new, single-story dental office structure, approximately 5,000 square feet in building footprint. We assume that no below grade levels are planned for construction. Additionally, paved parking areas and drive lanes are planned for construction. Development will also include installation of underground utilities to service the proposed structure. Building loads were unavailable at the time of this report preparation, but are anticipated to be relatively light. Information provided by the project team indicated that minimal cuts/fills of up to approximately 1 foot are anticipated to facilitate construction. The project site is shown in Figure 1. If proposed construction, including the anticipated site grading and structural loading, differs from those described above, or changes subsequently, GROUND should be notified to re-evaluate the information in this report. Once final grading and loading information is available, we should also be notified to review and re-evaluate the parameters provided herein, as necessary.
SITE CONDITIONS

At the time of our exploration, the project site consisted of an undeveloped lot with roughly graded soils at the surface. Based on our experience with the project site, overlot grading operations occurred that generally resulted in material cuts throughout the site. A landscaped area including rock was associated with the site on the western perimeter. The general topography across the project site was relatively flat with slopes ranging from approximately 1 to 2 percent generally descending toward the north. The southern perimeter of the site slopes away from Highway 7 to the north a distance of approximately 6 feet. The project site is bordered by commercial development to the north and west (King Soopers and associated fuel station), undeveloped lots to the east, and Highway 7 to the south.

Based on our review of Google Historical Imagery of the proposed project site, man-made fill materials were observed in our test holes likely due to potential past overlot grading operations associated with the King Soopers facility and associated fuel station. Man-made fill was observed in the test holes at the time of drilling. The exact extents, limits, and composition of any man-made fill were not determined as part of the scope of work addressed by this study and should be expected to potentially exist at varying depths and locations across the site.

GEOLOGIC SETTING

The subject parcel lies within the Denver Basin geologic province that consists largely of a sequence of sedimentary rock formations deposited and preserved in a structural depression in north-central Colorado. In the general project area, these sedimentary rocks dip eastward at low angles (less than 10 degrees, typically) and are overlain by a variety of surficial deposits including alluvial (stream-laid) sediments, eolian (wind-blown) materials and colluvial (slope-wash) deposits.
According to published geologic maps, e.g., Colton (1978[1]) the overburden deposits underlying the project area are Upper Holocene to Bull Lake Glaciation Windblown Clay, Silt, and Sand materials (Qe). These surficial soils are shown as underlain by the Laramie Formation (KI) consisting of claystone, shale, sandy shale, and sandstone. Windblown deposits are known to be susceptible to local hydro-consolidation or ‘collapse.’

Other published geologic maps e.g. Trimble and Matchette (1979[2]) depict exposure of the Dawson and Arapahoe Formation (TKDa) in the region of the site. We interpret that the absence of windblown, silt, and sand materials present on published geologic maps is due to the earthwork associated with the King Soopers project and fuel station. We interpret the overlying soils to be residual (developed in place) soils derived from the underlying claystone and sandstone. We interpret the claystones encountered in the test holes to be Laramie Formation (KI) deposits.

SUBSURFACE EXPLORATION

The subsurface exploration for the project was conducted on December 2nd, 2016. A total of five (5) test holes were drilled with a truck-mounted rig, advancing continuous flight auger to evaluate the subsurface conditions as well as to retrieve soil and bedrock samples for laboratory testing and analysis. Of these, three (3) test holes were drilled within the proposed building footprint and two (2) test holes were drilled within the proposed pavement areas. The foundation test holes were drilled to depths ranging from approximately 22 to 30 feet below existing grades and the pavement test holes were drilled to depths of approximately 2 to 9½ feet below existing grades. A representative of GROUND directed the subsurface exploration, logged the test holes in the field, and prepared the soil and bedrock samples for transport to our laboratory. Practical rig refusal was encountered in some of the test holes at depths ranging from approximately 2 to 31 feet below existing grade.

Samples of the subsurface materials were retrieved with a 2-inch I.D. California liner sampler. The sampler was 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 exploratory test holes are presented in Figure 2. Explanatory notes and a legend are provided in Figure 3. GROUND utilized the Client-provided site plan indicating existing features, etc., and Google Map imagery to approximately locate the test holes.

LABORATORY TESTING

Samples retrieved from our test holes were examined and visually classified in the laboratory by the project engineer. Laboratory testing of soil and bedrock samples obtained from the subject site included standard property tests, such as natural moisture contents, dry unit weights, grain size analyses, swell-consolidation testing, unconfined compressive strength, and liquid and plastic limits. Water-soluble sulfate and corrosivity tests were completed on selected samples of the soils as well. Laboratory tests were
performed in general accordance with applicable ASTM and AASHTO protocols. Results of the laboratory testing program are summarized in Tables 1 and 2.

**SUBSURFACE CONDITIONS**

The subsurface conditions encountered in the test holes generally consisted of man-made fill material, approximately 6 to 10 inches thick, underlain by sand and clay. These materials were underlain by claystone and sandstone bedrock encountered at depths near the existing surface to approximately 6 feet below existing grades that extended to foundation test hole termination depths of approximately 22 to 30 feet below existing grades.

It also should be noted that coarse gravel, cobbles and boulders are not well represented in samples obtained from small diameter test holes. At this site, therefore, it should be anticipated that gravel and cobbles, and possibly boulders, may be present in the fill and native soils, as well as comparably sized fragments of construction debris, even where not included in the general descriptions of the site soil types below.

*Man-Made Fill* was generally composed of clay and sand, was moderately to highly plastic, fine to coarse grained with gravel and occasional cobbles, dry to moist, and medium brown in color.

*Sands and Clays* were interbedded, fine to medium grained, low- to moderately plastic, medium dense, dry to slightly moist, light to medium brown in color, calcareous, and iron stained.

*Claystone and Sandstone Bedrock (Comparatively Un-Weathered Bedrock)* were interbedded with lenses of sandstone, non to moderately plastic, fine to medium grained, very hard and relatively resistant, dry to slightly moist, iron stained, and light brown to gray in color. Cemented sandstone bedrock was encountered during our exploration program and will present excavation difficulties during construction.

*Groundwater* was encountered in Test Holes 2 and 3 at approximate depths of 25 to 27 feet below existing grade at the time of drilling. 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.

**Swell-Consolidation Testing** of samples of the on-site materials encountered in the project test holes indicated a potential for heave (See Table 1). Swells ranging from approximately 0.4 to 5.7 percent and a consolidation of 0.6 percent were measured upon wetting under various surcharge pressures (see Table 1).

**ENGINEERING SEISMICITY**

According to the 2015 International Building Code® (Section 1613 Earthquake Loads), “Every structure, and portion thereof, including nonstructural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resist the effects of earthquake motions in accordance with ASCE 7, excluding Chapter 14 and Appendix 11A. The seismic design category for a structure is permitted to be determined in accordance with Section 1613 (2015 IBC) or ASCE 7.” Exceptions to this are further noted in Section 1613.

Utilizing the USGS’s Seismic Design Maps Tool (http://earthquake.usgs.gov/designmaps/us/application.php) and site latitude/longitude coordinates of 40.000751 and -105.010396 (obtained from Google Earth), respectively, the project area is indicated to possess an $S_{DS}$ value of 0.192 and an $S_{D1}$ value of 0.092.

Per 2015 IBC, Section 1613.3.2 Site class definitions, “Based on the site soil properties, the site shall be classified as Site Class A, B, C, D, E or F in accordance with Chapter 20 of ASCE 7. Where the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the building official or geotechnical data determines that Site Class E or F soil is likely to be present at the site”.

Based on the soil conditions encountered in the test holes drilled on the site, our review of applicable geologic maps, as well as our experience within the Project site vicinity, GROUND estimates that a Site Class D according to ASCE 7 (Table 20.3-1) could be anticipated for seismic foundation design. This parameter was estimated utilizing the above-referenced table as well as extrapolation of data beyond the deepest depth explored. Actual shear wave velocity testing/analysis and/or exploration to 100 feet was not performed. In the event the Client desires to potentially utilize Site Class C for
design, according to ASCE 7, actual downhole seismic shear wave velocity testing and/or exploration to subsurface depths of at least 100 feet, should be performed. In the absence of additional subsurface exploration/analysis, a Site Class D should be utilized for design.

The largest recorded earthquake (estimated magnitude 6.2 to 6.6) in Colorado occurred in November 1882. While the specific location of this earthquake is very uncertain, it is postulated to have occurred in the Front Range near Rocky Mountain National Park. The most recent significant seismic movements associated with the Rock Mountain Arsenal Fault (Commerce City, Colorado) occurred in the 1960s, generating earthquakes up to magnitude 5.5. Since the early 1960s, numerous earthquakes with magnitudes up to approximately 5, with the majority possessing magnitudes of 2 to 4, have been experienced within the State. Recently, earthquakes ranging in magnitude from 3.7 (Craig, Colorado) to 3.9 (Eads, Colorado and Trinidad, Colorado) occurred during the time period of July, 2009 through August, 2009. On August 23, 2011, a 5.3 magnitude earthquake occurred 9 miles west-southwest of Trinidad, Colorado. Earthquakes with similar magnitudes, and potentially greater, are anticipated to continue by the USGS, throughout the State. Furthermore, based on the subsurface conditions at the site and the risks associated with this nearest fault, the risk of liquefaction of the site soils is considered low.

FOUNDATION/FLOOR SYSTEMS OVERVIEW

Anticipated building loads were unavailable at the time of this report preparation. We assume building loads will be relatively light and provided information indicated material cuts/fills up to approximately 1 foot may be necessary to facilitate construction. Based on our laboratory testing program, the overburden materials possess a moderate potential for heave. Based on the data obtained for this study and our experience on similar projects, our estimates indicate likely vertical, post-construction movements on the order of 3 to 4 inches where structural elements are supported directly on the existing earth materials. Please refer to Appendix A: Geotechnical Considerations for Design for additional information regarding our analysis program.

For the least potential of post-construction movement, it is GROUND’s opinion that a deep foundation system consisting of straight-shaft drilled piers advanced into the
underlying bedrock with a structural floor system be utilized. Additionally, building entryways and other attached building appurtenances should ideally be founded on piers the same as the main building structure, to reduce the potential of differential movement. Utilizing this option as well as other applicable recommendations provided in this report, GROUND anticipates potential post-construction foundation movements of approximately ½-inch.

As an alternate foundation/floor system (but not equal in foundation/floor slab performance), spread footings and a slab-on-grade floor could be utilized provided the Owner is aware of the potential for post-construction movement, as stated herein, and accepts the risks of post-construction movement. Based on our exploration and analysis programs, over-excavation and replacement of the site earth materials resulting in a fill prism thickness of at least 9 feet should be performed in order to reduce, but not eliminate, the potential for heave/consolidation, differential and total movement, the associated risks for distress/damage, and to create a relatively homogeneous soil condition beneath the slab and footings. This fill prism thickness should be referenced as beneath the slab + gravel layer (assuming an approximate 5 to 6 inch slab and 4 inches of gravel). Depending upon the actual footing bearing elevation, we anticipate approximately 6 feet of fill prism soil will result beneath foundations. The fill prism layer should extend laterally at least 10 feet beyond the building beneath any building appurtenances including entryways, patios, courtyards, etc. Utilizing this option as well as other applicable suggestions provided in this report, GROUND estimates potential total movements of approximately 1 inch and differential movements of approximately ½ inch over a distance of 40 feet. These materials should be placed in accordance with the information provided in the Project Earthwork section of our report. Realized movement should be expected to exceed these estimates in localized areas.

Fill prism materials could consist of the on-site generated in-situ fill soils, bedrock material, and/or approved import material. Please note that the proper processing of the on-site bedrock materials can be very difficult and will likely require substantial effort on behalf of the earthwork contractor. Excavated claystone will require a well-coordinated effort to moisture treat, process, place and compact properly. In-place bedrock deposits were dense and relatively dry, and require a significant volume of water to be mixed into the excavated material to bring it to a uniform moisture contents. Adequate watering, and compaction equipment that aids in breaking down the material (e.g., a Caterpillar
825 compactor-roller, tractor-towed disk, etc.), will be necessary. These materials should be placed in accordance with the information provided in the *Project Earthwork* section of our report.

Inadequate site drainage and/or ineffective fill processing will result in an increase in the movement estimate provided. In addition, actual movements may be more or less depending on the subsurface materials present and the overall site drainage after construction is completed and landscape irrigation commences.

**FOUNDATION SYSTEMS**

*Deep Foundations*

**Geotechnical Parameters for Drilled Pier Design** If proper design and construction as described below is implemented effectively, then post-construction, vertical foundation movements may be taken as ½ inch with similar differential movements over spans of about 40 feet.

Based on the results of the field exploration, laboratory testing, and experience, the design criteria presented below should be observed for a straight-shaft, drilled pier foundation system.

1) Drilled piers should bear in ‘comparatively unweathered’ bedrock underlying the site. For design purposes, ‘comparatively unweathered’ bedrock may be taken to be at and below depths ranging from near the existing surface to approximately 6 feet below existing grades. For bidding purposes, this elevation may vary. Lignite and lignite-containing bedrock is not considered ‘comparatively unweathered’ bedrock.

2) Drilled piers should be at least **18 inches** in diameter and should be designed with a maximum length to diameter ratio of 30 to 1. The actual length to diameter ratios should be determined by the structural engineer.

3) Drilled piers should have a minimum length of **32 feet**. The actual drilled pier lengths should be determined by the structural engineer based on loading, etc., with further increases in length possibly required by the conditions encountered during installation at each drilled pier location.
4) Drilled piers also should penetrate at least 12 feet into comparatively unweathered bedrock or 3 drilled pier diameters, whichever is greater.

5) Based on the bedrock penetration, minimum pier length and estimating the top of comparatively unweathered bedrock to range from near the surface to approximately 6 feet below existing grade, drilled pier lengths of approximately 32 feet are anticipated to meet the geotechnical criteria. Actual drilled pier lengths commonly will be greater due to structural considerations, conditions in the drilled pier holes, actual comparatively unweathered bedrock depths, etc.

6) Drilled piers bearing in comparatively unweathered bedrock may be designed for an allowable end bearing pressure of 30,000 psf.

The portion of the drilled pier penetrating comparatively unweathered bedrock may be designed for a skin friction value of 2,250 psf for the portion of the pier penetrating comparably unweathered bedrock. 100 percent of the skin friction may be used to resist both compressional loads and uplift.

7) Estimated settlement of properly constructed drilled piers will be low, on the order of ½-inch, to mobilize skin friction.

8) Drilled piers should be designed for a minimum dead load pressure of 8,000 psf based on drilled pier cross-section area.

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

9) Drilled piers should be reinforced as determined by the structural engineer. At a minimum, drilled piers should be reinforced for their full length to resist the tensile loading created by the swelling soils and bedrock. Tension may be estimated as an uplift skin friction of 1,600 psf applied to the upper 20 feet of the drilled pier.

Reinforcement design also should include any deficit between the dead load applied in design and the minimum dead load provided above.
10) A 8-inch or thicker continuous void should be provided beneath grade beams, drilled pier caps, and foundation walls to concentrate drilled pier loadings. The void space should be protected from backfill intrusion.

11) Penetration of comparatively unweathered bedrock in drilled pier shafts should be roughened artificially to assist the development of peripheral shear between the drilled pier and bedrock. Artificially roughening of drilled pier holes should consist of installing shear rings 3 inches high and 2 inches deep in the portion of each drilled pier penetrating comparatively unweathered bedrock and below a depth of 20 feet, from top of pier. The shear rings should be installed 18 inches on centers.

The specifications should allow a geotechnical engineer to waive the requirement for shear rings depending on the conditions actually encountered in individual drilled pier holes, however.

12) Groups of closely spaced drilled piers placed to support concentrated loads will require an appropriate reduction of the estimated capacities. Reduction of axial capacity generally can be avoided by spacing drilled piers at least 3 diameters center to center. At this spacing or greater, no reduction in axial capacities or horizontal soil modulus values is required. Drilled pier groups spaced less than 3 diameters center to center should be studied on an individual basis to determine the appropriate axial capacity reduction(s). The settlement of closely spaced groups of drilled piers should be studied on an individual basis.

Linear arrays of drilled piers, however, must be spaced at least 8 diameters center to center to avoid reductions in lateral capacity when loaded in line with the array (parallel to the line connecting the drilled pier centers). Linear arrays of drilled piers spaced more closely than 8 diameters center to center should be studied to determine the appropriate lateral capacity reductions in that direction.

Refer to Figures 4 and 5 for additional information regarding reductions in lateral and axial capacity for closely spaced piers.

**Drilled Pier Construction** The following should be considered during the construction of drilled pier foundations.
13) The depth of comparatively unweathered bedrock should be determined in the field at each drilled pier location and may differ from other information provided herein.

14) Lenses or beds of relatively soft bedrock not suitable for foundation support may be encountered within the comparatively unweathered bedrock section, which may result in lengthening the drilled piers.

15) Some bedrock beneath the site was very hard to resistant. The pier-drilling contractor should mobilize equipment of sufficient size and operating capability to achieve the design lengths and bedrock penetration.

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

16) Groundwater was encountered in the test holes at depths of approximately 25 to 27 feet below existing grade at the time of drilling. Groundwater, combined with granular soils, often results in caving during pier installation. Seating of the casing in the upper layers of the bedrock may not create positive cutoff of water infiltration. The contractor should be prepared to address this condition.

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

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

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

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

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

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

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

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

25) Based on the data obtained for this study and our experience with similar sites and conditions, lateral load analysis using the Terzaghi method may take the values tabulated below for the modulus of horizontal subgrade reaction ($K_h$) to be characteristic of the soils and bedrock underlying the site, based on a simplified soil / bedrock profile. Resistance to lateral loads by deep foundations should be neglected in the upper 3 feet of soils, whether fill or native and within the fill materials.
Horizontal Modulus Subgrade Reaction ($K_h$) – Terzaghi Method

<table>
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<tr>
<th>Material</th>
<th>$K_h$ based on Foundation Element Width / Diameter 1.5-Foot Diameter</th>
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<tbody>
<tr>
<td>Sands and Clays</td>
<td>60 tcf</td>
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<tr>
<td>Claystone and Sandstone Bedrock</td>
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Note that the $K_h$ values tabulated above are dependent on deep foundation element width or diameter. For deep foundation elements greater than 1.5-feet in diameter, divide the $K_h$ values by the corresponding pier diameters (measured in feet).

If “L-Pile” or a similar computer program is used for lateral analysis of the piles, recommended geotechnical parameters for input into that program are tabulated below for the same simplified soil / bedrock profile. These include, unit wet weights ($\gamma'$), angles of internal friction ($\phi$), cohesion ($c$), for the earth materials, as well as values for strain at 50 percent of failure stress ($\varepsilon_{50}$) and horizontal soil modulus ($K$). Again, resistance to lateral loads should be neglected in the upper 3 feet of soils, whether fill or native.

Note that below the water table (taken to be as shallow as 25 feet below existing grade) the unit weights must be adjusted for buoyancy by reducing the values by 62.4 pcf.
## Estimated Geotechnical Parameters for Lateral Load Analysis

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<tr>
<th>Soil / Bedrock Material</th>
<th>Parameter</th>
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<tr>
<td>Sands and Clays</td>
<td>$\gamma'$</td>
<td>120 pcf</td>
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<tr>
<td></td>
<td>$c$</td>
<td>1000 psf</td>
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<td>$\varepsilon_{50}$</td>
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<td>Claystone and Sandstone Bedrock</td>
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<td>$\varepsilon_{50}$</td>
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<tr>
<td></td>
<td>$k$</td>
<td>2,500 pci</td>
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### Shallow Foundations

#### Geotechnical Parameters for Shallow Foundation Design

1) Footings should bear on at least 6 feet of properly compacted on-site generated (and/or imported materials), as discussed in the Foundation/Floor System Overview section. The fill prism should extend laterally at least 10 feet beyond the perimeter of the buildings.

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

The fill section should be laterally consistent and of uniform thickness to reduce differential, post-construction foundation movements. A differential fill section will tend to increase differential movements.

2) Footings bearing on properly compacted on-site generated (and/or imported materials), as previously described in the Foundation/Floor Systems Overview section, may be designed for an allowable soil bearing pressure of 2,000 psf for footings up to 5 feet in width (assuming a maximum load of 50 kips). In the event the footing width is greater than 5 feet, we should be notified to reevaluate these parameters.
These values may be increased by \( \frac{1}{3} \) for transient loads such as wind or seismic loading. For larger footings, a lower allowable bearing pressure may be appropriate.

Compression of the bearing soils for the provided allowable bearing pressure is estimated to be 1 inch, based on an assumption of drained foundation conditions. If foundation soils are subjected to an increase/fluctuation in moisture content, the effective bearing capacity will be reduced and greater post-construction movements than those estimated above may result.

This estimate of foundation movement is from direct compression of the foundation soils.

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 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 \textit{Lateral Earth Pressure} section of this report.

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

8) The lateral resistance of spread footings will be developed as sliding resistance of the footing bottoms on the foundation materials and by passive soil pressure against the sides of the footings. Sliding friction at the bottom of footings may be taken as 0.34 times the vertical dead load.
9) In order to reduce differential settlements between footings along continuous footings, footing loads should be as uniform as possible. Differentially loaded footings will settle differentially. Similarly, differential fill thicknesses beneath footings will result in increased differential settlements.

**Shallow Foundation Construction**

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

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

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

13) All foundation subgrade should be compacted with a vibratory plate compactor prior to placement of concrete.

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

**FLOOR SYSTEM**

**Structural Floor**

Floor slabs present a very difficult problem where swelling materials are present near floor slab elevation because sufficient dead load cannot be imposed on them to resist the uplift pressure generated when the materials are wetted and expand. As noted above, expansive soils are present on the site. Therefore, a structural floor should be utilized for the proposed additions as the floor system resulting in the least potential for
post-construction floor movement, including differential movement. Entryway floor slabs/systems should ideally be constructed as structural floors.

Structural floors should be supported on grade beams and straight-shaft drilled piers in the same manner as the building structure. Requirements for the number and position of additional piers to support the floors will depend upon the span, design load, and structural design, and should be developed by the Structural Engineer. Geotechnical information for design and installation of drilled piers are provided in the Foundation System section of this report.

Structural floors should be constructed to span above a well-ventilated crawl space permitting utility lines to be installed above the swelling materials. The crawl space should be adequately sized to allow access to and maintenance of utility piping. Piping connections through floors, grade beams, or foundation walls should allow for differential movement between the piping and the structural element through which the piping is penetrating.

A vapor barrier meeting ASTM E-1745 (Class “A”) should be considered for installation below all structurally supported floors and if utilized, should be properly attached/sealed to foundation walls/drilled piers above the void material. The sheet material should not be attached to horizontal surfaces such that condensate might drain to wood or corrodirable metal surfaces.

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

New buildings generally lack ventilation due primarily to systematic efforts to construct air-tight, energy-efficient structures. Therefore, areas such as crawl spaces beneath structural floors are typically areas of elevated humidity which never completely dry. This condition can be aggravated in some locations by shallow groundwater or a perched groundwater condition, which can result in saturated soils within several feet of the finished building pad grade. Persistently warm, humid conditions in the presence of
cellulose, which is found in many typical construction materials, creates an ideal environment for the growth of molds, fungi and mildew. Published data suggest links between molds and illnesses. Therefore, crawl spaces beneath structural floors should be provided with adequate, active ventilation systems or other active mechanisms such as specially designed HVAC systems to reduce the potential for mold, fungus and mildew growth. Crawl spaces should be inspected periodically so that remedial measures can be taken in a timely manner, should mold, fungus or mildew be present and require removal.

The Owner must be willing to accept the risks of potential mold, fungus and mildew growth when electing to utilize a structural floor system. Additionally, the contractor is solely responsible for the means and methods during construction including adequate ventilation, and any observation or testing performed during construction does not relieve the contractor of that responsibility.

All plumbing lines should be carefully tested before operation. Where utility lines enter through the floor, positive bond breaks should be provided. Utility lines can be displaced by soils and bedrock movements, which are not reflected in the building. Design and installation of associated fixtures should accommodate this potential differential movement, which could be on the order of 2 inches.

**Slab-on-Grade Floors**

**Geotechnical Parameters for Slab-on-Grade Floors**

1) Lightly loaded slabs should be placed on at least 9 feet of properly moisture conditioned and compacted on-site generated materials as discussed in the *Foundation/Floor Systems Overview* section of this report. The remedial fill section should extend at full depth at least 10 feet beyond the building perimeter.

2) An allowable subgrade vertical modulus (K) of 75 pci may be utilized for lightly loaded slabs supported by the on-site materials. This value is for a 1-foot x 1-foot plate; they should be adjusted for slab dimension.

3) The prepared surface on which the slabs will be cast should be observed by the Geotechnical Engineer prior to placement of reinforcement. Exposed loose, soft,
or otherwise unsuitable bedrock materials should be removed and properly replaced.

4) Slabs should be separated from all bearing walls, columns, and footings with slip joints, which allow unrestrained vertical movement. Slabs should not bear on footings or other foundation elements.

5) 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.

6) Interior partitions (if applicable) resting on floor/concrete 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 door frames. A slip joint, which will allow at least 2 or more inches of vertical movement, is recommended. If slip joints are placed at the tops of walls, in the event that the slabs move, it is likely that the wall will show signs of distress, especially where the slabs meet the exterior wall.

7) Concrete slabs-on-grade should be placed on properly prepared subgrade. They should also be constructed and cured according to applicable standards and be provided with properly designed and constructed control joints. The design and construction of such joints should account for cracking as a result of shrinkage, tension, and loading; curling; as well as proposed slab use. Joint layout based on the slab design may require more frequent, additional, or deeper joints, and should also be based on the ultimate use and configuration of the slabs. Areas where slabs consist of interior corners or curves (at column blockouts or around corners) or where slabs have high length to width ratios, high degree of slopes, thickness transitions, high traffic loads, or other unique features should be carefully considered. The improper placement or construction of control joints will increase the potential for slab cracking. ACI, AASHTO, and other industry groups provide many guidelines for proper design and construction of concrete slabs-on-grade and the associated jointing.
8) Slabs should be adequately reinforced. Structural considerations for slab thickness, jointing, and steel reinforcement in floor slabs should be developed by the Structural Engineer. Placement of slab reinforcement continuously through the control joint alignments will tend to increase the effective size of concrete panels and reduce the effectiveness of control joints.

9) All plumbing lines should be carefully tested before operation. Where plumbing lines enter through the floor, a positive bond break should be provided. Flexible connections allowing 2 or more inches of vertical movement should be provided for slab-bearing mechanical equipment. Greater movements may occur depending upon the fill prism section selected by the owner.

10) Moisture can be introduced into a slab subgrade during construction and additional moisture will be released from the slab concrete as it cures. Placement of a properly compacted layer of free-draining gravel, 4 or more inches in thickness, beneath the slabs should be performed. This layer will help distribute floor slab loadings, ease construction, reduce capillary moisture rise, and aid in drainage. 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.

11) The Client/Project Team should review the American Concrete Institute's (ACI) Sections 301/302/360 for additional guidance regarding slab on grade design and construction. Vapor Barriers should meet applicable performance standards as stated in ASTM E 1745.

Slab movements are directly related to the increases in moisture contents to the underlying soils after construction is completed. The precautions and parameters itemized above will not prevent the movement of floor slabs if the underlying materials are subjected to moisture fluctuations. However, these steps will reduce the damage if such movement occurs.

**MECHANICAL ROOMS/MECHANICAL PADS**

Often, slab-bearing mechanical rooms/mechanical equipment are incorporated into projects. Our experience indicates these are located as partially below-grade or
adjacent to the exterior of a structure. These elements should be founded on the same type of foundation systems as the main structure. Furthermore, mechanical connections must allow for potential differential movements.

**EXTERIOR FLATWORK**

Care should be taken with regard to proper design and subgrade preparation under and around site improvements. Similar to slab-on-grade floors, exterior flatwork and other hardscaping placed on the soils encountered on-site will experience post-construction movements due to volume change of the subsurface soils and the relatively light loads that they impose. Both vertical and lateral soil movements can be anticipated. Distress to hardscaping will result. The measures outlined below will help to reduce, but not eliminate, damages to these improvements.

As stated in the *Foundation/Floor System* section, foundations and floor systems placed directly on the on-site materials could experience 3 to 4 inches or more of movement (including differential and total movements). Similar movement potentials should be anticipated for exterior flatwork and other hardscaping. As stated, in order to reduce the potential for post-construction movement and based on our heave/consolidation analysis, over-excavation and replacement of the site earth materials to a depth of approximately 5 feet should be performed. We understand that these depths may not be cost effective for some projects. Provided the owner understands the risks identified above and accepts the potential for post-construction movement as discussed in this report, the subgrade under exterior flatwork or other (non-building) site improvements can be processed to lesser depths. Over-excavation and replacement to a depth of at least **24 inches** may be more feasible, but will result in movements and subsequent distress to site improvements. These movements will likely be more severe if surface drainage is not effective and maintained. The excavated soil should be replaced as properly moisture-conditioned and compacted fill as outlined in the *Project Earthwork* section of this report. Areas of flatwork will require removal and replacement as an element of future maintenance.

The processing depth should occur *prior* to placing any additional fill required to achieve finished design grades. This processing depth will not eliminate potential movements.
The excavated soil should be replaced as properly moisture-conditioned and compacted fill as outlined in the *Project Earthwork* section of this report.

Prior to placement of flatwork, a proof roll should be performed to identify areas that exhibit instability and deflection. The soils in these areas should be removed and replaced with properly compacted fill or stabilized.

Flatwork should be provided with effective control joints. Increasing the frequency of joints may improve performance. Industry guidelines developed by ACI, PCA, and others should be consulted regarding construction and control joints.

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

As discussed in the *Surface Drainage* section of this report, proper drainage also should be maintained after completion of the project and re-established as necessary. In no case should water be allowed to pond on or near any of the site improvements or a reduction in performance should be anticipated.

**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. 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 the 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 commonly it may not be practical to implement some of these measures for reducing scaling due to safety considerations, project scheduling, etc. In
such cases, 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 (“frost heave”) in sustained cold weather up to 2 inches or more. This heaving can develop relatively rapidly. A portion of this movement typically is recovered when the soils thaw, but due to loss of soil density, some degree of displacement will remain. This can result even where the subgrade soils were prepared properly.

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

**WATER-SOLUBLE SULFATES**

The concentrations of water-soluble sulfates measured in selected samples retrieved from the test holes ranged from approximately 0.01 to 0.02 percent during testing. Such concentrations of water-soluble sulfates represent a negligible environment for sulfate attack on concrete exposed to these materials. Degrees of attack are based on the scale of ‘negligible,’ ‘moderate,’ ‘severe’ and ‘very severe’ as described in the “Design and Control of Concrete Mixtures,” published by the Portland Cement Association (PCA). The Colorado Department of Transportation (CDOT) utilizes a corresponding scale with 4 classes of severity of sulfate exposure (Class 0 to Class 3) as described in the published table below.
REQUIREMENTS TO PROTECT AGAINST DAMAGE TO CONCRETE BY SULFATE ATTACK FROM EXTERNAL SOURCES OF SULFATE

<table>
<thead>
<tr>
<th>Severity of Sulfate Exposure</th>
<th>Water-Soluble Sulfate (SO₄) In Dry Soil (%)</th>
<th>Sulfate (SO₄) In Water (ppm)</th>
<th>Water Cementitious Ratio (maximum)</th>
<th>Cementitious Material Requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class 0</td>
<td>0.00 to 0.10</td>
<td>0 to 150</td>
<td>0.45</td>
<td>Class 0</td>
</tr>
<tr>
<td>Class 1</td>
<td>0.11 to 0.20</td>
<td>151 to 1500</td>
<td>0.45</td>
<td>Class 1</td>
</tr>
<tr>
<td>Class 2</td>
<td>0.21 to 2.00</td>
<td>1501 to 10,000</td>
<td>0.45</td>
<td>Class 2</td>
</tr>
<tr>
<td>Class 3</td>
<td>2.01 or greater</td>
<td>10,001 or greater</td>
<td>0.40</td>
<td>Class 3</td>
</tr>
</tbody>
</table>

Based on these data GROUND, makes no suggestions for use of special, sulfate-resistant cement in project concrete.

SOIL CORROSIVITY

The degree of risk for corrosion of metals in soils commonly is considered to be in two categories: corrosion in undisturbed soils and corrosion in disturbed soils. The potential for corrosion in undisturbed soil is generally low, regardless of soil types and conditions, because it is limited by the amount of oxygen that is available to create an electrolytic cell. In disturbed soils, the potential for corrosion typically is higher, but is strongly affected by soil chemistry and other factors.

A preliminary corrosivity analysis was performed to provide a general assessment of the potential for corrosion of ferrous metals installed in contact with earth materials at the site, based on the conditions existing at the time of GROUND’s evaluation. Soil chemistry and physical property data including pH, reduction-oxidation (redox) potential, and sulfides content were obtained. Test results are summarized on Table 2.

**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 (AWWA, 2010). Testing indicated pH values of approximately 6.6 to 7.3.

**Reduction-Oxidation** testing indicated negative potentials: -41 to -7 millivolts. Such low potentials typically create a more corrosive environment.
Sulfide Reactivity testing for the presence of sulfides indicated ‘trace’ results. The presence of sulfides in the site soils also suggests a more corrosive environment.

Soil Resistivity In order to assess the “worst case” for mitigation planning, samples of materials retrieved from the test holes were tested for resistivity in the laboratory, after being saturated with water, rather than in the field. Resistivity also varies inversely with temperature. Therefore, the laboratory measurements were made at a controlled temperature.

Measurements of electrical resistivity indicated values of approximately 6,808 to 13,348 ohm-centimeters in samples of the site earth materials. The following table presents the relationship between soil resistivity and a qualitative corrosivity rating (ASM, 2003).

<table>
<thead>
<tr>
<th>Soil Resistivity (ohm-cm)</th>
<th>Corrosivity Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt;20,000</td>
<td>Essentially non-corrosive</td>
</tr>
<tr>
<td>10,000 – 20,000</td>
<td>Mildly corrosive</td>
</tr>
<tr>
<td>5,000 – 10,000</td>
<td>Moderately corrosive</td>
</tr>
<tr>
<td>3,000 – 5,000</td>
<td>Corrosive</td>
</tr>
<tr>
<td>1,000 – 3,000</td>
<td>Highly corrosive</td>
</tr>
<tr>
<td>&lt;1,000</td>
<td>Extremely corrosive</td>
</tr>
</tbody>
</table>

Corrosivity Assessment The American Water Works Association (AWWA, 2010) has developed a point system scale used to predict corrosivity. The scale is intended for protection of ductile iron pipe but is valuable for project steel selection. When the scale equals 10 points or higher, protective measures for ductile iron pipe are suggested. The AWWA scale (Table A.1 Soil-test Evaluation) is presented below. The soil characteristics refer to the conditions at and above pipe installation depth.

---

2 American Water Works Association ANSI/AWWA C105/A21.5-05 Standard.
Table A.1 Soil-test Evaluation

<table>
<thead>
<tr>
<th>Soil Characteristic / Value</th>
<th>Points</th>
</tr>
</thead>
<tbody>
<tr>
<td>Resistivity</td>
<td></td>
</tr>
<tr>
<td>&lt;1,500 ohm-cm</td>
<td>10</td>
</tr>
<tr>
<td>1,500 to 1,800 ohm-cm</td>
<td>8</td>
</tr>
<tr>
<td>1,800 to 2,100 ohm-cm</td>
<td>5</td>
</tr>
<tr>
<td>2,100 to 2,500 ohm-cm</td>
<td>2</td>
</tr>
<tr>
<td>2,500 to 3,000 ohm-cm</td>
<td>1</td>
</tr>
<tr>
<td>&gt;3,000 ohm-cm</td>
<td>0</td>
</tr>
<tr>
<td>pH</td>
<td></td>
</tr>
<tr>
<td>0 to 2.0</td>
<td>5</td>
</tr>
<tr>
<td>2.0 to 4.0</td>
<td>3</td>
</tr>
<tr>
<td>4.0 to 6.5</td>
<td>0</td>
</tr>
<tr>
<td>6.5 to 7.5</td>
<td>0*</td>
</tr>
<tr>
<td>7.5 to 8.5</td>
<td>0</td>
</tr>
<tr>
<td>&gt;8.5</td>
<td>3</td>
</tr>
</tbody>
</table>

Redox Potential

| < 0 (negative values) | 5      |
| 0 to +50 mV           | 4      |
| +50 to +100 mV        | 3½     |
| > +100 mV             | 0      |

Sulfide Content

| Positive              | 3½     |
| Trace                 | 2      |
| Negative              | 0      |

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 points for this range.

The redox potential of a soil is significant, because the most common sulfate-reducing bacteria can only live in anaerobic conditions. A negative redox potential indicates anaerobic conditions in which sulfate reducers thrive. A positive sulfide reaction reveals a potential problem caused by sulfate-reducing bacteria. Anaerobic conditions are regarded as potentially corrosive.
Based on a maximum possible score of 25.5 using the AWWA method, the value of 10 for the use of corrosion protection, and scores of approximately 10 in the on-site soil, the soil appears to comprise a potentially moderately corrosive environment for buried metals.

If additional information is needed regarding soil corrosivity, 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, may significantly alter corrosion potential.

LATERAL EARTH PRESSURES

Structures which are laterally supported and can be expected to undergo only a limited amount of deflection should be designed for “at-rest” lateral earth pressures. The cantilevered retaining structures will be designed to deflect sufficiently to mobilize the full active earth pressure condition, and may be designed for “active” lateral earth pressures. “Passive” earth pressures may be applied in front of the structural embedment to resist driving forces.

The at-rest, active, and passive earth pressures in terms of equivalent fluid unit weight for the on-site backfill and CDOT Class 1 structure backfill are summarized on the table below. Base friction may be combined with passive earth pressure if the foundation is in a drained condition. The values for the on-site material in the upper 10 feet provided in the table below were approximated utilizing a unit weight of 118 pcf and a phi angle of 22 degrees.
Lateral Earth Pressures (Equivalent Fluid Unit Weights)

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Water Condition</th>
<th>At-Rest (pcf)</th>
<th>Active (pcf)</th>
<th>Passive (pcf)</th>
<th>Friction Coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>On-Site Backfill</td>
<td>Drained</td>
<td>73</td>
<td>53</td>
<td>220</td>
<td>0.27</td>
</tr>
<tr>
<td>Structure Backfill (CDOT Class 1)</td>
<td>Drained</td>
<td>55</td>
<td>35</td>
<td>400</td>
<td>0.45</td>
</tr>
</tbody>
</table>

If the selected on-site soil meets the criteria for CDOT Class 1 structure backfill as indicated in the *Project Earthwork* section of this report, the lateral earth pressures for CDOT Class 1 structure backfill as shown on the above table may be used. To realize the lower equivalent fluid unit weight, the selected structure backfill should be placed behind the wall to a minimum distance equal to the retained wall height.

The lateral earth pressures indicated above are for a horizontal upper backfill slope. The additional loading of an upward sloping backfill as well as loads from traffic, stockpiled materials, etc., should be included in the wall/shoring design. GROUND can provide the adjusted lateral earth pressures when the additional loading conditions and site grading are clearly defined.

**PROJECT EARTHWORK**

The following information is for private improvements; public roadways or utilities should be constructed in accordance with applicable municipal / agency standards.

*General Considerations:* Site 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, vegetation and other deleterious materials should be removed and disposed of off-site. Relic underground utilities should be abandoned in accordance with applicable regulations, removed as necessary, and properly capped.
Topsoil present on-site should not be incorporated into ordinary fills. Instead, topsoil should be stockpiled during initial grading operations for placement in areas to be landscaped or for other approved uses.

**Use of Existing Fill Soils:** Fill soils were encountered in the test holes at the time of drilling. The existing fill soils may not be suitable for re-use as fill for this project due to the presence of trash, organic material, construction debris, or other deleterious materials. Excavated fill materials should be evaluated and tested, as appropriate, with regard to re-use.

**Use of Existing Native Soils:** Overburden soils that are free of trash, organic material, construction debris, and other deleterious materials are suitable, in general, for placement as compacted fill. Organic materials should not be incorporated into project fills.

Fragments of rock, cobbles, and inert construction debris (e.g., concrete or asphalt) larger than 3 inches in maximum dimension will require special handling and/or placement to be incorporated into project fills. In general, such materials should be placed as deeply as possible in the project fills. A Geotechnical Engineer should be consulted regarding appropriate guidance for usage of such materials on a case-by-case basis when such materials have been identified during earthwork. Standard recommendations that likely will be generally applicable can be found in Section 203 of the current CDOT Standard Specifications for Road and Bridge Construction.

**Reconditioning Expansive Soils and Bedrock:** Expansive materials including overburden soils and claystone bedrock were observed in the test holes and include swell potentials up to 5.7 percent, measured against various surcharge pressures as tested in our laboratory. Earth materials with greater swell potentials could exist on-site. In our experience, these materials have been excavated and replaced with variable success due to the natural properties of highly expansive materials as well as poorly controlled materials processing and varying placement techniques. The Client, Owner, and Contractor must understand that expansive and/or resistant bedrock will require additional processing including but not limited to stockpile moisture conditioning and multiple sub-excavations including partial to complete removal of previously moisture conditioned and compacted fill materials. The following parameters will not eliminate
post-construction movement associated with structures/utility trenches/improvements constructed on expansive soils and bedrock, but may tend to make movements more uniform.

Excavated materials will require a well-coordinated effort to moisture treat, process, place, and compact properly. In-place bedrock deposits were hard to relatively resistant and relatively dry, and require a significant volume of water to be mixed into the excavated material to bring them to a uniform moisture content from optimum to 3 percent above the optimum prior to compaction. The daily moisture content in the tested fill should be on the order of between 2 and 3 percent above the optimum as an additional requirement. Bedrock fragments should be reduced so as to achieve a soil-like mass. Adequate watering, and compaction equipment that aids in breaking down the material (e.g., a Caterpillar 825 compactor-roller), likely will be needed. Excavated bedrock will require additional moisture conditioning and processing in an open area outside of utility trenches prior to placement as backfill. Crushing or other methods should be anticipated to sufficiently reduce sandstone bedrock.

Potential earthwork contractors should be made aware that as stated, significant processing and reprocessing of the on-site materials will likely be required. The placement of on-site fill materials should be monitored by a representative of the Geotechnical Engineer, under separate contract, requested by the Owner. Moisture content and relative compaction testing may need to be conducted at an increased frequency in order to provide a greater level of information to the contractor. In addition, immediately following placement and compaction, a geotechnical engineer should be retained to collect drive samples of the fill materials and they should be tested to determine the resultant swell-potential of the in-place materials. Scheduling of this testing is not the responsibility of the Geotechnical Engineer. Materials represented by samples exhibiting 0.5 percent swell or greater upon wetting against a 1,000 psf surcharge should be re-worked at increased moisture contents and re-compacted in accordance with the parameters above. Typical earthworking procedures may not be successful in reducing swell potentials to less than 0.5 percent and other methods/procedures may be necessary. When processed, movements as a result of heave/settlement may still occur in these fill soils, in some areas significant (greater than 1½ inches). Therefore, the surface and subsurface drainage measures, landscape
irrigation, underdrain, site grading, and maintenance described within this report should be closely considered during the design phase.

**Imported Fill Materials:** If it is necessary to import material to the site, the imported soils should be free of organic material, and other deleterious materials. Imported material should consist of relatively impervious soils that have less than 60 percent passing the No. 200 Sieve and should have a plasticity index less than 15. Representative samples of the materials proposed for import should be tested and approved prior to transport to the site.

**Fill Platform Preparation:** Prior to filling, the top 8 to 12 inches of in-place materials on which fill soils will be placed should be scarified, moisture conditioned and properly compacted in accordance with the parameters below to provide a uniform base for fill placement. *If over-excavation is to be performed, then these parameters for subgrade preparation are for the subgrade below the bottom of the specified over-excavation depth.*

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. The surfaces to receive fill must be effectively stable prior to placement of fill.

GROUND’s experience within the project area suggests the frost depth to be approximately 3 feet, below ground surface.

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

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 modified Proctor dry density at moisture contents within 2 percent of optimum moisture content as determined by ASTM D1557.
Soils that classify as ML, MH, CL or CH should be compacted to 98 percent of the maximum standard Proctor density at moisture contents from 1 percent below to 3 percent above the optimum moisture content as determined by ASTM D698.

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

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 suggested ranges are obtained.

**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, GROUND does not suggest this procedure 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, 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.

It is GROUND’s opinion that wherever possible, excavations be backfilled with approved, on-site soils placed as properly compacted fill. Where this is not feasible, use of “Controlled Low Strength Material” (CLSM), i.e., a lean, sand-cement slurry (“flowable fill”) or a similar material for backfilling should be considered.
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 filled ground, typically on the order of 1 to 2 percent of the fill depth. 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 will typically take place during earthwork construction, provided the contractor achieves the compaction levels 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 site slopes supported by on-site soils up to 10 feet in height may 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.

**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. Dry, resistant, sandstone and claystone bedrock was encountered in the test holes during exploration. Cemented sandstone was also be encountered during excavation. Adequate watering, and compaction equipment that aids in breaking down the material (e.g., a Caterpillar 825 compactor-roller) will be necessary. We anticipate that excavation into the bedrock will be slow even with conventional, heavy duty, excavating equipment, and will entail greater than typical wear on the equipment used.

Temporary, un-shored excavation slopes up to 10 feet in height be cut no steeper than 1½:1 (horizontal : vertical) in the site soils in the absence of seepage. Sloughing on the slope faces should be anticipated at this angle. Local conditions encountered during construction, such as groundwater seepage and loose sand, will require flatter slopes.
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.

Should site constraints prohibit the use of the slope angles, temporary shoring should be used. The shoring should be designed to resist the lateral earth pressure exerted by building, traffic, equipment, and stockpiles.

Groundwater was encountered in Test Holes 2 and 3 at depths of approximately 25 and 27 feet below existing grade at the time of drilling. Therefore, groundwater is not anticipated to be a significant factor in relatively shallow excavations. If seepage or groundwater is encountered during excavation, the Geotechnical Engineer should evaluate the conditions and provide additional parameters as appropriate.

Drilled pier excavations will likely encounter groundwater, as well as very hard and relatively resistant bedrock. The contracted should be prepared to penetrate resistant bedrock and to install the piers in the presence of groundwater.

Good surface drainage should be provided around temporary excavation slopes to direct surface runoff away from the slope faces. A properly designed drainage swale should be provided at the top of the excavations. In no case should water be allowed to pond at the site. Slopes should also be protected against erosion. Erosion along the slopes will result in sloughing and could lead to a slope failure.

Excavations in which personnel will be working must comply with all OSHA Standards and Regulations. 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 above solely as a service to the Client, and is not assuming responsibility for construction site safety or the Contractor’s activities.

UTILITY PIPE INSTALLATION AND BACKFILLING

Pipe Support: The bearing capacity of the site soils appeared adequate, in general, for support of anticipated water lines. The pipe + water are less dense than the soils which will be displaced for installation. Therefore, GROUND anticipates no significant pipe settlements in these materials where properly bedded.
Excavation bottoms may expose soft, loose or otherwise deleterious materials, including debris. 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, they may be designed for an allowable passive soil pressure of 220 psf per foot of embedment, to a maximum of 2,200 psf. Sliding friction at the bottom of thrust blocks may be taken as 0.27 times the vertical dead load.

**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, we suggest backfilling the entire depth of the trench (both bedding and common backfill zones) with “controlled low strength material” (CLSM), i.e., a lean, sand-cement slurry, “flowable fill,” or similar material along all trench alignment reaches with low tolerances for surface settlements.

We suggest that CLSM used as pipe bedding and trench backfill exhibit a 28-day unconfined compressive strength between 50 to 200 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: 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 Client/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.

If it is necessary to import material for use as backfill, the imported soils should be free of vegetation, organic debris, and other deleterious materials. Imported material should consist of relatively impervious soils that have less than 60 percent passing the No. 200 Sieve and should have a plasticity index of less than 15. Representative samples of the materials proposed for import should be tested and approved prior to transport to the site.

Soils placed for compaction as trench backfill should be conditioned to a relatively uniform moisture content, placed and compacted in accordance with 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, we suggest the use of CLSM or similar material in lieu of granular bedding and compacted soil backfill 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, it is our opinion that with regard to potential migration of fines into the pipe bedding, design and installation follow ASTM D2321. If the granular bedding does not meet filter criteria for the enclosing soils, then non-woven filter fabric (e.g., Mirafi® 140N, or the equivalent) should be placed around the bedding to
reduce migration of fines into the bedding which can result in severe, local surface settlements. Where this protection is not provided, settlements can develop/continue several months or years after completion of the project. In addition, clay or concrete cut-off walls should be installed to interrupt the granular bedding section to reduce the rates and volumes of water transmitted along the sewer alignment which can contribute to migration of fines.

If granular bedding is specified, the contractor should not anticipate that significant volumes of on-site soils will be suitable for that use. Materials proposed for use as pipe bedding should be tested by a geotechnical engineer for suitability prior to use. Imported materials should be tested and approved by a geotechnical engineer prior to transport to the site.

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 movement will result primarily from the introduction of water into the soil 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 grade beam or floor elevations. Therefore, wetting of the site 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 incorporated as part of project design and during construction. 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 project. 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 foundation excavations and underslab areas should be avoided during and after construction as well as throughout the improvements’ design life. Permitting increases/variations in moisture to the adjacent or supporting soils may result in a decrease in bearing capacity and an increase in
volume change of the underlying soils, and increased total and/or differential movements.

2) Positive surface drainage measures should be provided and maintained to reduce water infiltration into foundation soils.

The ground surface surrounding the exterior of each building should be sloped to drain away from the foundation in all directions. A minimum slope of 12 inches in the first 10 feet should be incorporated in the areas not covered with pavement or concrete slabs, or a minimum 3 percent in the first 10 feet in the areas covered with pavement or concrete slabs. Reducing the slopes to comply with ADA requirements may be necessary by other design professionals but may entail an increased potential for moisture infiltration and subsequent volume change of the underlying soils and resultant distress.

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

3) Drainage should be established and maintained to direct water away from sidewalks and other hardscaping as well as utility trench alignments. 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 / increased heave. 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) On some sites, slopes may descend toward buildings locally. Such slopes can be created during grading even on comparatively flat sites. As previously stated, the current site topography slopes toward the center of the site. In such cases,
even where the slopes as described above are implemented effectively, water may flow toward and beneath a structure or other site improvements with resultant additional, post-construction movements. If the final site configuration includes graded or retained slopes descending toward the improvements, surface drainage swales and/or interceptor drains should be installed between the improvements and the slope.

Where irrigation is applied on or above slopes, drainage structures commonly are needed near the toe-of-slope to prevent on-going or recurrent wet conditions.

6) Roof downspouts and drains should discharge well beyond the perimeter of the structure foundations (minimum 10 feet) and backfill zones and be provided with positive conveyance off-site for collected waters.

7) Based on our experience with similar facilities, the project may include landscaping/watering near site improvements. Irrigation water – both that applied to landscaped areas and over-spray – is a significant cause of distress to improvements. To reduce the potential for such distress, vegetation requiring watering should be located 10 or more feet from building perimeters, 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.

8) Use of drip irrigation systems can be beneficial for reducing over-spray beyond planters. Drip irrigation can also be beneficial for reducing the amounts of water introduced to foundation/subgrade 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 a building, it is GROUND’s opinion that the plants 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 a limited increase in risk, the use of water-tight
planters may be replaced by local shallow underdrains beneath the planter beds. Colorado Geological Survey – Special Publication 43 provides additional guidelines for landscaping and reducing the amount of water that infiltrates into the ground.

GROUND understands many municipalities require landscaping within 10 feet of building perimeters. Provided that positive, effective surface drainage is initially implemented and maintained throughout the life of the facility and the Owner understands and accepts the risks associated with this requirement, vegetation that requires little to no watering may be located within 10 feet of the building perimeter.

9) Inspections must be made by facility representatives to make sure that the landscape irrigation is functioning properly throughout operation and that excess moisture is not applied.

10) Plastic membranes should not be used to cover the ground surface adjacent to the building as soil moisture tends to increase beneath these membranes. Perforated “weed barrier” membranes that allow ready evaporation from the underlying soils may be used.

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.

11) Maintenance as described herein may include complete removal and replacement of site improvements in order to maintain effective surface drainage.

12) Detention ponds commonly are incorporated into drainage design. When a detention ponds fills, the rate of release of the water is controlled and water is retained in the pond for a period of time. Where in-ground storm sewers direct surface water to the pond, the granular pipe bedding also can direct shallow groundwater or infiltrating surface water toward the pond. Thus, detention ponds can become locations of enhanced and concentrated infiltration into the subsurface, leading to wetting of foundation soils in the vicinity with consequent heave or settlement. Therefore, unless the pond is clearly down-gradient from the proposed buildings and other structures that would be adversely affected by
wetting of the subgrade soils, including off-site improvements, the detention pond should be provided with an effective, low permeability liner. In addition, cut-off walls and/or drainage provisions should be provided for the bedding materials surrounding storm sewer lines flowing to the pond.

SUBSURFACE DRAINAGE

As a component of project civil design, properly functioning, subsurface drain systems (underdrains) can be beneficial for collecting and discharging saturated subsurface waters. Underdrains will not collect water infiltrating under unsaturated (vadose) conditions, or moving via capillarity, however. In addition, if not properly constructed and maintained, underdrains can transfer water into foundation soils, rather than remove it. This will tend to induce 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.

Although inclusion of an underdrain system is common on commercial sites like the subject facility, particularly where shallow foundations are used, professional opinion varies regarding the potential benefits relative to the cost. Therefore, the owner and the design team and contractor should assess the net benefit of an underdrain system as a component of overall project drainage.

If, however, below-grade or partially below-grade level(s) are added to the building, then we recommend that an underdrain system be included. 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. If a (partially) below-grade level is limited in extent, the underdrain system, etc., may be local to that area.

Geotechnical Parameters for Underdrain Design Where an underdrain system is included in project drainage design, it should be designed in accordance with the recommendations below. The actual underdrain layout, outlets, and locations should be developed by a civil engineer.
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, as well as a waterproof membrane.

2) The free-draining gravel should contain less than 5 percent passing the No. 200 Sieve and more than 50 percent retained on the No. 4 Sieve, and have a maximum particle size of 2 inches. Each collection pipe should be surrounded on the sides and top (only) with 6 or more inches of free-draining gravel.

3) 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 waterproof membrane should underlie the gravel and pipe, and be attached to the foundation grade beam or stem wall as shown in the detail.

5) The underdrain system should be designed to discharge at least 5 gallons per minute of collected water.

6) 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 sufficient for effective drainage, but a minimum of 1 percent. (Flatter gradients may be used but will convey water less efficiently and entail an increased risk of local post-construction movements.)

Pipe gradients also should be designed to accommodate at least 1 inch of differential movement after installation along a 50-foot run.
7) Underdrain ‘clean-outs’ should be provided at intervals of no more than 100 feet to facilitate maintenance of the underdrains. Clean-outs also should be provided at collection and discharge pipe elbows of 60 degrees or more.

8) 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.

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. The standard care of practice in pavement design describes the flexible pavement section as a “20-year” design pavement: however, most flexible pavements will not remain in satisfactory condition without routine maintenance and rehabilitation procedures performed throughout the life of the pavement. Pavement designs for the private pavements were developed in general accordance with the design guidelines and procedures of the American Association of State Highway and Transportation Officials (AASHTO).

Subgrade Materials

Based on the results of our field exploration and laboratory testing, the potential pavement subgrade materials classify as ranging from A-4 to A-6 soils in accordance with the American Association of State Highway and Transportation Officials (AASHTO) classification system.

Based on our experience at similar sites, a resilient modulus value of 3,592 psi was estimated for the on-site materials. 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**

Traffic data for the proposed facility was unavailable at the time of our report preparation. Based on our experience with similar projects, an equivalent 18-kip daily load application (EDLA) value of 5 was assumed for the general parking areas. The EDLA value of 5 was converted to an equivalent 18-kip single axle load (ESAL) value of 36,500 for a 20-year design life. If anticipated traffic loadings differ significantly from these assumed values, GROUND should be notified to re-evaluate the pavement sections below.

**Pavement Design**

The soil resilient modulus and the ESAL values were used to determine the required design structural number for the project pavements. The required structural number was then used to develop the pavement sections. Pavement designs were based on the DARWin™ computer program that solves the 1993 AASHTO pavement design equations. A reliability level of 80 percent and a terminal serviceability of 2 were utilized for design of the pavement sections. A structural coefficient of 0.40 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.

<table>
<thead>
<tr>
<th>Location</th>
<th>Flexible Section (inches Asphalt)</th>
<th>Composite Section (inches Asphalt / inches Aggregate Base)</th>
<th>Rigid Section (inches Concrete)</th>
</tr>
</thead>
<tbody>
<tr>
<td>General Parking Area and Drive Lane</td>
<td>6</td>
<td>4 / 6</td>
<td>6</td>
</tr>
</tbody>
</table>

Additionally, trash collection areas, as well as other pavement areas subjected to high turning stresses or heavy truck traffic, be provided with rigid pavements consisting of Portland cement concrete (see table above). Additionally, the owner should consider reinforced concrete in these areas. Concrete sections should be underlain by 6 inches of properly compacted aggregate base.

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.
Concrete pavements 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. Concrete should have a minimum modulus of rupture of third point loading of 650 psi. Normally, concrete with a 28-day compressive strength of 4,000 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.

In areas of repeated turning stresses the concrete pavement joints should be fully tied or doweled. We suggest that civil design consider joint layout in accordance with CDOT’s M Standards. Standard plans for placement of ties and dowels, etc., (CDOT M Standards) for concrete pavements can be found at the CDOT website: http://www.dot.state.co.us/DesignSupport/

If composite flexible sections are placed, the aggregate base material should meet the criteria of CDOT Class 6 aggregate base course. Base course should be placed in uniform lifts not exceeding 8 inches in loose thickness and compacted to at least 95 percent of the maximum dry density a uniform moisture contents within 3 percent of the optimum as determined by ASTM D1557 / AASHTO T-180, the “modified Proctor.”

Subgrade Preparation: As stated, in order to reduce the potential for post-construction movement, over-excavation and replacement of the site earth materials to a depth of approximately 5 feet should be performed. However, we understand that these depths may not be cost effective for most projects. Provided the owner understands the risks identified above and accepts the potential for post-construction movement and distress/damage as discussed in this report, the subgrade under pavement areas can be processed to lesser depths. Over-excavation and replacement to a depth of at least 12 inches may be performed, but will result in movements and subsequent distress to site improvements. Movements similar to these discussed in the Exterior Flatwork section should be expected. These movements will likely be even more severe if surface drainage is not effective and maintained. GROUND anticipates post-construction movements within paved areas will occur resulting in damage that will require removal and replacement.
The Contractor should be prepared either to dry the subgrade materials or moisten them, as needed, prior to compaction. It may be difficult for the contractor to achieve and maintain compaction in some on-site soils encountered without careful control of water contents. Likewise, some site soils likely will “pump” or deflect during compaction if moisture levels are not carefully controlled. The Contractor should be prepared to process and compact such soils to establish a stable platform for paving, including use of chemical stabilization, if necessary.

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. Passing a proof roll is an additional requirement, beyond placement and compaction of the subgrade soils in accordance with this report. Some soils that are compacted in accordance with the parameters herein may not be stable under a proof roll, particularly at moisture contents in the upper portion of the acceptable range.

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.)

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

A crack sealing and fog seal/chip seal program should be performed on the pavements every 3 to 4 years. After approximately 8 to 10 years, patching, additional crack sealing, and asphalt overlay may be required. Prior to future overlays, it is important that all transverse and longitudinal cracks be sealed with a flexible, rubberized crack sealant in order to reduce the potential for propagation of the crack through the overlay. Traffic volumes that exceed the values utilized by this report will likely necessitate the need of pavement maintenance practices on a schedule of shorter timeframe than that stated above. The greatest benefit of preventive maintenance is achieved by placing the treatments on sound pavements that have little or no distress.

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 use of thick base course or reinforced concrete pavement can reduce this. Our office should be contacted if these alternates are desired.

The assumed 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. An effective program of regular maintenance should be developed and implemented to seal cracks, repair distressed areas, and perform thin overlays throughout the life of the pavements.

CLOSURE

Geotechnical Review

The author of this report should be retained to review project plans and specifications to evaluate whether they comply with the intent of the information in this report.

The geotechnical parameters and conclusions 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 information in this report, or by providing alternative parameters.

Materials Testing

The client should consider retaining a Geotechnical Engineer to perform materials testing during construction. The performance of such testing or lack thereof, 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 their work; furthermore, testing by the geotechnical engineer does not preclude the contractor from obtaining or providing whatever services they deem necessary to complete the project in accordance with applicable documents.
Limitations

This report has been prepared for Covenant Group as it pertains to the proposed PAC Dental facility as described herein. It may not contain sufficient information for other parties or other purposes. The owner or any prospective buyer relying upon this report must be made aware of and must agree to the terms, conditions, and liability limitations outlined in the proposal.

In addition, GROUND has assumed that project construction will commence by Spring/Summer 2017. Any changes in project plans or schedule should be brought to the attention of the Geotechnical Engineer, in order that the geotechnical parameters may be re-evaluated and, as necessary, modified.

The geotechnical conclusions and information 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, the Geotechnical Engineer should be advised at once, so that re-evaluation of the information may be made in a timely manner. In addition, a contractor who relies upon 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.

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

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 project improvements are understood by the Client, Project Owner (if different), or properly conveyed to any future owner(s). Utilizing these parameters for planning, design, and/or construction constitutes understanding and acceptance of conclusions or information provided herein, potential risks, associated improvement performance, as well as the limitations inherent within such estimations. If any information referred to herein is not well understood, it is imperative for the Client, Owner (if different), or anyone using this report to contact the author or a company principal immediately.

This report was prepared in accordance with generally accepted soil and foundation engineering practice in the project area at the date of preparation. Current applicable codes may contain criteria regarding performance of structures and/or site improvements which may differ from those provided herein. Our office should be contacted regarding any apparent disparity. GROUND makes no warranties, either expressed or implied, as to the professional data, opinions or information contained herein. Because of numerous considerations that are beyond GROUND’s control, the economic or technical performance of the project cannot be guaranteed in any respect.

GROUND appreciates the opportunity to complete this portion of the project and welcomes the opportunity to provide the Owner with a cost proposal for construction observation and materials testing prior to construction commencement.

Sincerely,
GROUND Engineering Consultants, Inc.

Brian Knecht G.I.T. Reviewed by Amy Crandall P.E.
SITE PLAN PROVIDED BY CLIENT

1

Indicates test hole number and approximate location.

(Not to Scale)
LEGEND:

- **Man-Made Fill:** Generally composed of clay and sand, was moderately to highly plastic, fine to coarse grained with gravel and occasional cobbles, dry to moist, and medium brown in color.

- **Sands and Clays:** Interbedded, fine to medium grained, low- to moderately plastic, medium dense, dry to slightly moist, light to medium brown in color, calcareous, and iron stained.

- **Claystone and Sandstone Bedrock (Comparatively Un-Weathered Bedrock):** Interbedded with lenses of sandstone, non to moderately plastic, fine to medium grained, very hard and relatively resistant, dry to slightly moist, iron stained, and light brown to gray in color.

Drive sample, 2-inch I.D. California liner sample

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.

Practical Rig Refusal

0 Depth to water level and number of days after drilling that measurement was taken.

NOTES:

1) Test holes were drilled on 12/02/2016 with 4-inch diameter continuous flight augers.

2) Locations of the test holes were measured 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.

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 this legend are for general classification purposes only. See the full text of this report for descriptions of the site materials and related information.

8) All test holes were immediately backfilled upon completion of drilling, unless otherwise specified in this report.
Axial Capacity Reductions as Functions of Closely Spaced Pier / Pile Elements.

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

Pier / Pile reductions should be interpolated from the graph above.
Lateral Capacity Reductions (p multipliers) as Functions of Closely Spaced Pier / Pile Elements

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

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

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

Figure to be reproduced in color for clarity.
## TABLE 1
SUMMARY OF LABORATORY TEST RESULTS

<table>
<thead>
<tr>
<th>Sample Location</th>
<th>Test Hole No.</th>
<th>Depth (feet)</th>
<th>Natural Moisture Content (%)</th>
<th>Natural Dry Density (pcf)</th>
<th>Percent Passing No. 200 Sieve</th>
<th>Atterberg Limits Liquid Limit</th>
<th>Plasticity Index</th>
<th>Percent Swell (Surcharge Pressure)</th>
<th>Unconfined Compressive Strength (psf)</th>
<th>USCS Classification</th>
<th>AASHTO Classification (GI)</th>
<th>Soil or Bedrock Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>6</td>
<td>9.7</td>
<td>105.3</td>
<td>66</td>
<td>25</td>
<td>4</td>
<td>-</td>
<td>-</td>
<td>A-4(1)</td>
<td>CL-ML</td>
<td>CLAYSTONE Bedrock</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>11</td>
<td>13.9</td>
<td>112.6</td>
<td>75</td>
<td>33</td>
<td>14</td>
<td>-0.6(1,375 psf)</td>
<td>-</td>
<td>A-6(9)</td>
<td>CL</td>
<td>CLAYSTONE Bedrock</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>3</td>
<td>8.7</td>
<td>105.5</td>
<td>87</td>
<td>29</td>
<td>10</td>
<td>1.2(375 psf)</td>
<td>-</td>
<td>A-4(8)</td>
<td>CL</td>
<td>CLAYSTONE Bedrock</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>13</td>
<td>13.6</td>
<td>99.0</td>
<td>56</td>
<td>30</td>
<td>8</td>
<td>-</td>
<td>840</td>
<td>A-4(2)</td>
<td>CL</td>
<td>CLAYSTONE Bedrock</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>5</td>
<td>10.3</td>
<td>112.4</td>
<td>88</td>
<td>32</td>
<td>12</td>
<td>5.7(625 psf)</td>
<td>-</td>
<td>A-6(10)</td>
<td>CL</td>
<td>CLAYSTONE Bedrock</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>10</td>
<td>14.2</td>
<td>102.1</td>
<td>86</td>
<td>28</td>
<td>6</td>
<td>0.4(1,250 psf)</td>
<td>-</td>
<td>A-4(4)</td>
<td>CL-ML</td>
<td>CLAYSTONE Bedrock</td>
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<tr>
<td></td>
<td>P-1</td>
<td>4</td>
<td>8.6</td>
<td>111.2</td>
<td>62</td>
<td>33</td>
<td>15</td>
<td>-</td>
<td>-</td>
<td>A-6(7)</td>
<td>CL</td>
<td>Sandy CLAY</td>
</tr>
</tbody>
</table>

Job No. 16-3717
## TABLE 2
### SUMMARY OF SOIL CORROSION TEST RESULTS

<table>
<thead>
<tr>
<th>Test Hole No.</th>
<th>Depth (feet)</th>
<th>Water Soluble Sulfates (%)</th>
<th>pH</th>
<th>Redox Potential (mV)</th>
<th>Sulfides Content</th>
<th>Resistivity (ohm-cm)</th>
<th>USCS Classification</th>
<th>Soil or Bedrock Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6</td>
<td>0.01</td>
<td>7.3</td>
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<td>Trace</td>
<td>13,348</td>
<td>CL-ML</td>
<td>CLAYSTONE Bedrock</td>
</tr>
<tr>
<td>P-1</td>
<td>4</td>
<td>0.02</td>
<td>6.6</td>
<td>-7</td>
<td>Trace</td>
<td>6,808</td>
<td>CL</td>
<td>Sandy CLAY</td>
</tr>
</tbody>
</table>

Job No. 16-3717
APPENDIX A

GEOTECHNICAL CONSIDERATIONS FOR DESIGN
GEOTECHNICAL CONSIDERATIONS FOR DESIGN

As noted above, data obtained for this study suggested potentials for post-construction heave of structures constructed on the soils and bedrock encountered at this site. Native, expansive earth materials and properly moisture-conditioned and compacted fill soils are generally stable with regard to their moisture-volume relationships so long as their moisture contents are kept unchanged. Post-construction wetting of expansive soils and bedrock gives rise to heave and resultant damages. Pavements, utilities, and sidewalks, etc., all can be affected, in addition to the building. The actual extent and distribution of post-construction uplift realized at a given location (and the nature and degree of resultant damages) will depend on a variety of factors. These factors include the spatial distribution of the expansion potentials in the underlying soils and bedrock, the type and design of the structure supported on them, structural loads, and the depth and lateral extent to which the soils become wetted after construction, etc. Because of the number and variability of these factors, the necessarily limited data set by which the geotechnical factors can be estimated, the variety of means by which estimates of post-construction heave can be made, and the inherent assumptions involved, estimates of future heave will tend to range widely in magnitude.

Various quantitative and semi-quantitative methods are used by geotechnical engineers in the Colorado Front Range area to estimate post-construction heave/settlement of structures, pavements, etc., as a step toward development of information for foundations, remedial earthworks, etc. Those typically used are based on practical engineering experience and judgment using combinations of measured values of soil moisture content, density and plasticity, one-dimensional swell/consolidation, and/or soil suction. The criteria provided in this report were based on the data presented herein, and 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 recommendations were based on our judgment of "likely movement potentials," (i.e., the amount of movement likely to be realized if site drainage is 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 the Owner or future prospective owners, upon receipt of this report, however, to discuss these risks and the geotechnical alternatives with us.

At any point in the design life of a structure, the vertical and lateral extents of future wetting of the underlying expansive soils will exert a dominant influence on the extent and distribution of future uplift. Professional experience and opinions differ with regard to depths to which significant, post-construction, soil moisture changes take place. Differing assumptions for this future “depth of wetting” at a site will give rise to differing estimates of potential, post-construction heave. Movement estimates based on much deeper than typically estimated depths of wetting will be larger than those based on more typically observed depths.

Engineering consulting and design practice always involves weighing the risks inherent in a given design approach against the construction costs associated with reducing those risks. The Owner (and subsequent prospective future owners) must, therefore, understand the risks and remedial approaches presented in this report (and the risk-cost trade-offs addressed by the Civil Engineer and Structural Engineer) in order to direct his design team to the portion of the Higher Cost / Lower Risk – Lower Cost / Higher Risk spectrum in which this project should be (or was) designed. If the Owner or a prospective future owner does not understand these risks, it is critical that he requests additional information or clarification so that his expectations reasonably can be met.

As noted above, the “depth of wetting” forecast for a given site strongly affects the anticipated performance of structure placed there. A 20-foot depth of wetting is greater than about 72 percent of the sites evaluated by Walsh and others (2009)\(^1\). This “depth of wetting” is appropriately conservative in our opinion for most projects of this type. Clearly, however, there is a potential for this “depth of wetting” to be exceeded, which if considered – other parameters being equal – would lead to greater estimates of post-construction movements. “Depths of wetting” of 30, 40 or 70 feet or more have been considered (e.g., Chao and others, 2006)\(^2\) and have been encountered locally in the

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field. These cases, however, generally are in unusual geologic conditions such as the "Designated Dipping Bedrock Area" as recognized by Jefferson County, Colorado, or identified forensically in unusual circumstances such as a pipe leak that has remained unrepaired for an extended period. Such "depths of wetting" are considered only rarely in engineering consulting practice, however, in more typical geologic settings in this area. If the Owner prefers a more conservative (or less conservative) "depth of wetting" be used to develop geotechnical recommendations, GROUND should be contacted to revise the geotechnical criteria herein.

Utilizing the above assumptions, data obtained for this study, and our experience on other projects in the vicinity, our estimates indicate potential likely post-construction movements of approximately 3 to 4 inches or more where building elements are supported directly on the existing earth materials that become wetted following construction to the depth described. Movements of this magnitude can cause severe cosmetic and/or structural distress to the proposed building. (These same general potentials for post-construction movement and damage also apply to project pavements, hardscaping, piping, and all other improvements in contact with the site earth materials where subject to post-construction wetting.)

At the subject site, several types of foundation systems, in conjunction with differing extents of remedial earthworks, etc., can be employed to support the proposed building on the soils and bedrock encountered in the test holes. Each combination entails a different degree of risk of post-construction building movements. These range from utilizing shallow spread footings and slab-on-grade concrete floors bearing directly on the native soils (entailing the greatest risk) to supporting the structures, including floors, on deep, drilled pier foundations bearing in the underlying bedrock at depths of more than 70 feet (entailing the least risk). Some research, in fact, has concluded that drilled piers as long as 100 feet or more may be needed to resist uplift in expansive materials such as those encountered on-site.

Ideally, the proposed building should be supported on drilled pier foundation systems, and provided with structural floors supported similarly. We assume that 70+ -foot piers are not practical economically for the subject project because such depths are not required to support the structural loads. It should be noted in this regard that many lightly to moderately loaded commercial and residential structures in the Colorado Front Range area have been supported successfully in similar materials on 20- to 40-foot
piers. Therefore, in the absence of direction otherwise from Covenant Group, we have assumed that the Owner prefers to use drilled piers of more conventional lengths for this project. The geotechnical criteria provided in the “Foundation System” section of this report for design of a drilled pier foundation system were developed accordingly. Although a drilled pier foundation system incorporating these criteria will not eliminate the risk of post-construction building movement, if the measures outlined in this report are implemented effectively, the likelihood of acceptable building performance to a reasonable engineering probability will be within local industry standards for construction of a drilled pier foundation system on soils and bedrock of this nature. Based on the conditions encountered in GROUND’s test holes, the assumptions outlined herein, including effective maintenance of site drainage, we estimate post-construction movements from heave and/or settlement of drilled pier foundations to be on the order of ½-inch. GROUND is available to meet, however, to discuss the risks and remedial approaches presented in this report, as well as other possible design approaches, upon request.

Irrespective of the foundation system and floor system selected by the Owner, some risk of post-construction building and floor movements will remain, even after effective implementation of the information presented in this report. The Owner and prospective future owners should understand these risks, as well as the site maintenance measures that are necessary to manage them. Other elements of the proposed improvements on this site (hardscaping, pavements, etc.) will be underlain by expansive earth materials, and likely will be damaged. Owner tolerances for movement and distress to these appurtenant improvements typically is greater than for buildings. Nevertheless, periodic maintenance will be required. To achieve performance similar to that of the building floors, similar foundation measures will be required. The Owner and prospective future owners should familiarize themselves with the measures presented in Noe and others (1997)\(^3\) and implement them at this site. This booklet is available from the Colorado Geological Survey in Denver, Colorado, and can be purchased from their website. (www.geosurvey.state.co.us). Although written for residential construction, the concepts and information outlined therein are generally applicable to building sites on expansive soils.

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