GROUND ENGINEERING

Geotechnical Subsurface Exploration Program
Eagle County Schools
Eagle Valley Elementary School
Eagle, Colorado

Final Report

Prepared for:
Eagle County Schools
c/o RLH Engineering, Inc.
948 Chambers Avenue
Eagle, Colorado 81631

Attention: Mr. Fred Voseipka

Job Number: 16-3797 **June 1, 2017**

TABLE OF CONTENTS

	Page
Purpose and Scope of Study	1
Planned Development	1
Site Conditions	2
Geologic Setting	3
Geologic Hazards	3
Subsurface Exploration	7
Laboratory Testing	8
Subsurface Conditions	8
Engineering Seismicity	10
Foundation/Floor System Overview	11
Foundation System	12
Floor System	15
Mechanical Rooms/Mechanical Pads	17
Exterior Flatwork	17
Water Soluble Sulfates	21
Soil Corrosivity	22
Lateral Earth Pressures	25
Project Earthwork	26
Excavation Considerations	29
Utility Pipe Installation and Backfilling	31
Surface Drainage	33
Subsurface Drainage	37
Pavement Sections	39
Closure and Limitations	45
Locations of Test Holes	Figures 1A & 1B
Logs of Test Holes	Figures 2-5
Legend and Notes	Figure 6
Typical Underdrain Detail	Figure 7
Summary of Laboratory Test Results	Table 1
Summary of Soil Corrosion Test Results	Table 2

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 school to be constructed at the Eagle County Schools 3rd Street Campus, located at the address of 747 E. 3rd Street in Eagle, Colorado. Our study was conducted in general accordance with the Consultant Agreement between Eagle County Schools and GROUND, dated November 15, 2016 and GROUND's Proposal No. 1610-2185, dated November 2, 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 improvements are presented below.

Additionally, information obtained during our preliminary geotechnical evaluation at the site[1] was utilized during the preparation of this report.

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 improvements are included herein.

PLANNED DEVELOPMENT

We understand that proposed construction will consist of a new school at a location within the existing Eagle Valley Elementary and Eagle Valley Middle School campus off 3rd Street in Eagle, Colorado, totaling an approximately 49,000 square foot building footprint to address current and future enrollment growth. We understand that at least a partial below grade level/walk-out is planned for construction. In addition, we assume

---

[1]GROUND Engineering Consultants, Inc., 2017, *Preliminary Geotechnical Subsurface Exploration Program, Eagle County Schools, Eagle Valley Elementary School Replacement, Eagle, Colorado,* Job No. 16-3718, prepared for Eagle County Schools in care of RLH Engineering, Inc, dated January 12, 2017

that development will also include installation of underground utilities. Site grading information was provided by the project team and indicated that cuts up to approximately 20 feet and fills up to approximately 9 feet will be necessary to facilitate site grading. Building load information provided by the project team indicated a maximum load of 220 kips. The project site is shown in Figures 1A & 1B. Once final grading, building layout, and building load information is available, we should also be notified to review and re-evaluate the parameters provided herein, as necessary.

If the proposed development differs significantly from that described above, GROUND should be notified to re-evaluate the conclusions and parameters contained herein.

SITE CONDITIONS

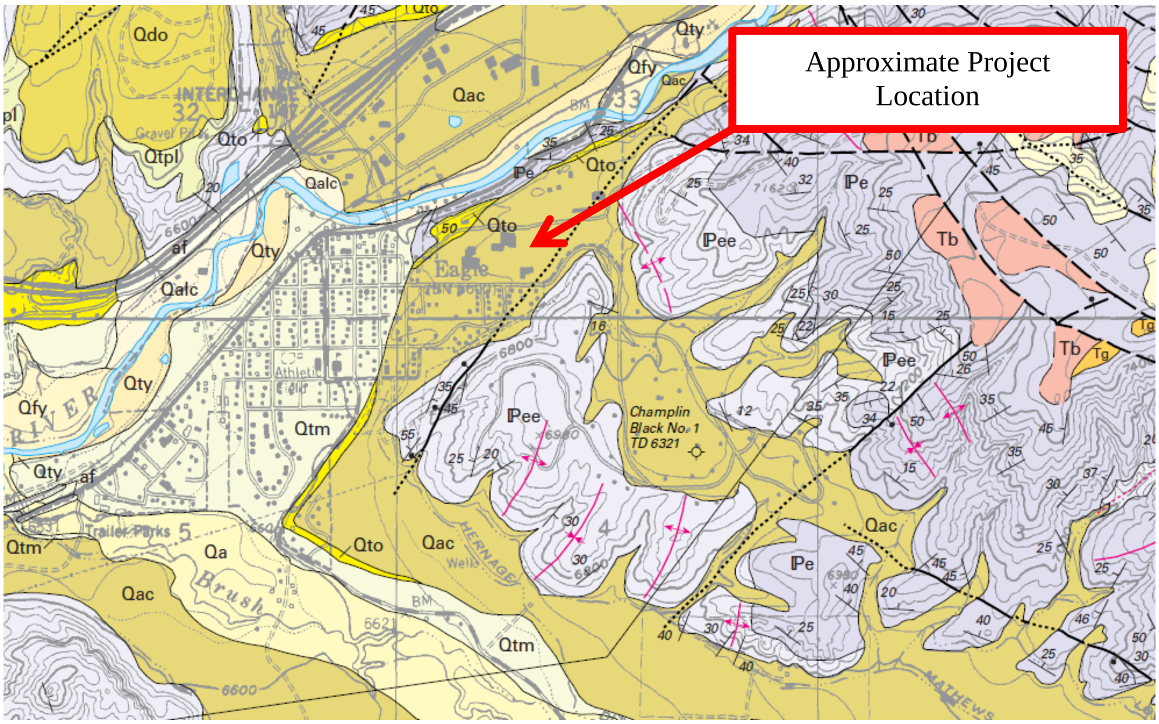
At the time of our exploration, the project site consisted of an existing school facility with associated fields, playgrounds, and other improvements. The general topography

across the project site was relatively gently to moderately sloping with slopes generally ranging from approximately 1 to 4 percent generally descending toward the north and west. Steeper slopes are associated with the transition between the upper and lower athletic fields on the eastern side of the project site. The project site is bordered by 2nd Street to the north, 3rd Street to the south, Eagle Valley Elementary Playground on the west, and athletic fields to the east.

Man-made fill was apparently encountered 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

Published geologic maps, e.g., Lidke (2002)[2] depict the site as underlain by undistinguished Holocene to middle Pleistocene alluvium and colluvium (**Qac**). These materials are described as gravels, silts and sands. These surficial deposits are mapped as underlain by the Middle Pennsylvanian Eagle Valley Evaporite (**Pee**). This formation is generally composed of gypsum, anhydrite, and interbedded siltstone with minor amounts of dolomite. It includes thick halite (salt) deposits locally.



Geologic Hazards

<u>*Soluble Mineral Dissolution and Sinkholes*</u> Gypsum, halite and other minerals in the Eagle Valley Evaporite (and the Eagle Valley Formation) are vulnerable to dissolution in groundwater. The resultant voids have propagated to the ground surface resulting in 'sink holes.' Sink holes of various ages have been mapped along the Eagle, Colorado

[2] Lidke, David J., 2002, *Geologic Map of the Eagle Quadrangle, Eagle County, Colorado*: United States Geological Survey, Map MF-2361.

and Roaring Fork River valleys, as well as along the tributaries to those rivers (such as Gypsum Creek or Brush Creek). Sink holes appear to have developed with the greatest frequency where the Eagle Valley Evaporite is overlain in the shallow subsurface by stream-gravels.

Therefore, the risk of sink hole development at or near the sites of the subject school building must be considered at least moderate. The likelihood of development of a sink hole at a given location, however, is difficult to forecast. Additional geotechnical drilling and geophysical studies attempting to locate nascent sink holes in the near surface have been unreliable, in our experience.

Also, it is our opinion that the risk at the location of the proposed school site is no greater than at most nearby sites in Eagle. The existing elementary school facility and the numerous residences and other buildings near the elementary school all have been constructed and utilized despite the similar risk. Geotechnical measures to mitigate the risk of structural damage from sink hole development – such as a deep, geo-textile-reinforced, remedial fill section – are relatively expensive and are un-proven in their effectiveness. Therefore, if Eagle County Schools can accept the risk of sink hole development, the building may be constructed without measures to mitigate that specific risk. GROUND will be available to discuss this risk in more detail.

Expansive Soils Swelling clayey soils and bedrock change volume in response to changes in moisture content that can occur seasonally, or in response to changes in land use, including development. Expansion potentials vary with moisture contents, density, and details of the clay chemistry and mineralogy. The swell potential in any particular area can vary markedly both laterally and vertically due to the complex interbedding of the site soil and bedrock materials. Moisture changes also occur erratically, resulting in conditions that cannot always be predicted.

The shallow earth materials underlying the site included clays with some silts, sands, and gravels with some cobbles and boulders likely. The plasticity of the site soils ranged from low to moderately plastic. Swells ranging from approximately 0.4 to 0.9 percent were measured on selected samples at various surcharge pressures (see Table 1). However, greater potential swells may be associated with the site soils.

Collapsible Soils Certain surficial deposits in the Eagle Valley area, typically evaporate materials are known to be susceptible to local hydro-consolidation or "collapse." Hydro-consolidation consists of a significant volume loss due to re-structuring of the constituent grains of the soil to a more compact arrangement upon wetting.

Consolidation testing performed on site materials indicated consolidations ranging from approximately 0.2 to 4.9 percent during testing of selected samples at various surcharge pressures (See Table 1.) Greater consolidations may be possible in site soils.

Radon Testing for the possible presence of radon gas prior to project development does not yield useful results regarding the potential accumulation of radon in completed structures. Radon accumulations typically are found in basements or other enclosed portions of buildings built in areas underlain at relatively shallow depths by granitic crystalline rock. The likelihood of encountering radon in concentrations exceeding applicable health standards on the subject site, underlain by relatively deep soils and sedimentary bedrock, is significantly lower.

Radon testing should be performed in the building(s) on-site, after construction is completed. Proper ventilation usually is sufficient to mitigate potential radon accumulations. Building designs should accommodate such ventilation for all building areas.

Seismic Activity / Faulting Neither site reconnaissance nor review of available geologic maps indicated the trace of an active or potentially active fault traversing or immediately adjacent to the site. Therefore, the likelihood of surface fault rupture at the site is considered to be relatively low.

Lidke (2002) depicts a fault close to the southeast margin of the site (approximately 1/8 mile to the southeast). This fault is depicted as not offsetting the middle Pleistocene to Holocene alluvium, only the older, underlying units including the Eagle Valley Evaporite and the overlying Eagle Valley Formation (**Pe**). Therefore, this fault is not considered to be active.

We consider the site to fall within the parameters of a Seismic Site Class D site, in accordance with 2015 IBC based on extrapolation of available data to depth. If a quantitative assessment of the classification is needed, shear wave velocity testing to

100+ feet or other surface testing methods will be required. A proposal for this work can be provided upon request. Compared with other regions of Colorado, recorded earthquake frequency in the project area is moderate.

Slope Stability and Erosion Lidke (2002), as well as larger scale geologic maps providing coverage of the site that were reviewed for this study, did not depict landslide deposits on or adjacent to the subject site.

The site is generally gently sloping to the north and west. During our preliminary reconnaissance of site area, no evidence was obviously noted of mass-wasting processes associated with steep slopes, such as landslides, slumps, or unusual soil creep. Therefore, the likelihood of project developments being affected by existing large scale, unanticipated slope instabilities is considered low.

Flooding The subject property lies less than approximately 700 feet south of the Eagle River, however, the project site is depicted by FEMA (2010)[3] as Zone X indicating a minimal risk of flooding. Therefore, the site does not appear to be vulnerable to flooding with the exception of heavy rainfall and associated temporary ponding of run-off in areas of relatively slow surface drainage. The site should be evaluated by a civil engineer in that regard.

Wetland Potential No obvious indications of conditions similar to jurisdictional wetlands were apparent during GROUND's site reconnaissance. Additionally, according to the U.S. Fish and Wildlife Service[4], the project site is not designated as a wetland area. However, during site development all regulations concerning wetland protection, as well as any other areas designated as wetlands by the Federal Wetlands Protection Act should be adhered to. Explicit designation of wetlands was not included as part of the scope of this study.

Mining Activity and Subsidence Review of U.S. Geological Survey maps covering the site such as Lidke (2002), and other available, published maps depicting areas of mining activities, did not indicate past mining activities on or immediately adjacent to the subject

[3] Federal Emergency Management Administration, 2010, http://map1.msc.fema.gov/idms/IntraView.cgi?KEY=21462945&IFIT=1 accessed on 12/30/2016
[4] U.S. Fish and Wildlife Service, National Wetlands Inventory, May 20, 2010, www.fws.gov/wetlands

parcel. Additionally, no surface indications of mining activities (i.e. subsidence) were apparent on the site during the site reconnaissance. Therefore, there appears to be little potential for surface subsidence associated with consolidation of former mine workings at depth.

Published geologic maps do indicate potential formations underlying the site at relatively deep depths that include evaporite (salt, gypsum, etc.) deposits, limestones or other materials vulnerable to subsurface dissolution. This potential is discussed above under the *Soluble Mineral Dissolution and Sinkholes* section of this report.

SUBSURFACE EXPLORATION

Subsurface exploration for the project was conducted on December 9th and 12th, 2016, and May 17th and 18th, 2017. A total of twenty (20) test holes were drilled using a truck-mounted drill rig advancing continuous flight auger. Eleven (11) of the test holes were initially drilled within the previously indicated preliminary locations based on information provided by the client. Following receipt of the final building layout for the proposed school, an additional nine (9) holes were drilled. Seven (7) of these test holes were drilled within the proposed building footprint and the remaining two (2) test holes were drilled within the proposed private paved areas. The test holes were advanced to depths of about 5 to 51 feet below existing grade within the approximate areas planned for development. Practical drill rig refusal was encountered in some of the test holes during drilling operations. Test holes were drilled to evaluate the subsurface conditions as well as to retrieve samples for laboratory testing and analysis. A representative of GROUND directed the subsurface exploration, logged the test holes in the field, and prepared the samples for transport to our laboratory. The test holes were backfilled immediately following drilling operations due to safety concerns.

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, a procedure 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. Depth and elevations 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 on Figure 1. Logs of the test holes are presented on Figures 2 through 5. Explanatory notes and a legend are provided on Figure 6. GROUND utilized the Client-provided site plan indicating existing features, etc. and Google Map imagery to approximately locate the test holes. Provided test hole elevations (from this study only) were estimated based on Client-provided grading plans.

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 included standard property tests, such as natural moisture contents, dry unit weights, grain size analyses, and Atterberg limits. Swell-consolidation testing, unconfined compressive strength, water soluble sulfates, and corrosivity testing were performed on select samples as well. Laboratory tests were performed in general accordance with applicable ASTM protocols. Results of the laboratory testing program are summarized in Table 1 and Table 2.

SUBSURFACE CONDITIONS

In general, the test holes penetrated a thin layer of topsoil[5], approximately 2 to 6 inches thick (thicker or thinner thickness likely exist locally), underlain by clay. These materials were underlain by claystone/sandstone bedrock at depths ranging from approximately 39 to 40 feet below existing grades in test holes TH-12 and TH-13. The test holes extended to depths of approximately 5 to 51 feet below existing grades.

Fill and/or debris materials were apparently recognized in some of the test holes, and may/likely exist elsewhere on site. Delineation of the complete lateral and vertical extents of the fills at the site, and their composition, was beyond our present scope of services. If detailed fill soil compositions at the site are of significance, they should be evaluated using test pits.

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

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 comprised of silty to sandy clay materials with sands and gravels. These materials were observed to contain some organic debris (wood). These materials were fine to gravel grained (cobbles and boulders possible), slightly moist to moist, low to moderately plastic, and light brown to dark brown in color.

Clay was silty and somewhat interbedded with sands and gravels. These materials were fine to gravel grained (cobbles and boulders possible), slightly moist to moist, non-plastic to moderately plastic, medium to hard, somewhat calcareous, occasionally iron stained, and light brown to dark brown to olive-gray in color.

Claystone/Sandstone Bedrock was low to moderately plastic, fine to coarse grained with some gravels, hard to very hard and slightly resistant, slightly moist to moist, occasionally caliche, and light brown to red-brown in color.

Groundwater was not obviously encountered in the test holes at the time of drilling. The test holes were backfilled immediately following drilling operations due to safety.

Groundwater levels should also be expected to fluctuate, and likely rise, in response to annual and longer-term cycles of precipitation, irrigation, snow melt, surface drainage, land use, and the development of transient, perched water conditions. It has been our experience that surface and groundwater levels fluctuate greatly in mountainous areas, primarily due to seasonal conditions such as spring runoff. These conditions are often highly variable and difficult to predict. Although these conditions generally exist for 1 to 3 months annually, their impact on design can be significant. In Eagle County, Colorado, it is common during construction to encounter dry conditions in the Fall and wet conditions in the Spring with relative groundwater fluctuations of 10 feet or more. This is particularly critical for foundation and deep utility excavations, cut slopes, culvert sizing, and for development adjacent to intermittently dry streams or rivers. Furthermore, if development has not established positive surface drainage particularly, prior to temporary winter shutdown procedures, other components of partial and complete development are compromised. The Contractor and the Project Team should consider these complex conditions prior to commencing construction.

Swell-Consolidation Testing suggested a potential for both consolidation and swell in the tested on-site materials. Consolidations ranging from approximately 0.2 to 4.9 percent and swells ranging from approximately 0.4 to 0.9 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 (2012 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 39.655465°N and -106.818091°W (obtained from Google Earth), respectively, the project area is indicated to possess an S_{DS} value of 0.304 and an S_{D1} value of 0.124.

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 (estimate this using the 2015 IBC/ASCE 7 guidelines) 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.

FOUNDATION/FLOOR SYSTEMS OVERVIEW

As stated, material cuts up to approximately 20 feet and material fills up to approximately 9 feet are planned to facilitate anticipated grading operations based on the provided finish floor elevations and the approximate existing ground surface. Additionally, based on our laboratory testing program, site earth materials possess a potential for both heave and consolidation. 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 2 or more inches where structural elements are supported directly on the existing earth materials. For the least potential for total and differential movement, it is GROUND's opinion that the proposed school structure be supported on a deep foundation system consisting of drilled piers/driven piles, and provided with a structural floor system. Additionally, building entryways and other attached building appurtenances should ideally be founded on piers/piles the same as the main building structure, to reduce the potential of differential movement. Utilizing this option as well as other applicable parameters provided in this report, GROUND anticipates potential post-construction foundation movements of approximately ½-inch. Please note that practical drill rig refusal was encountered in some of the test holes during drilling operations. These conditions may result in difficulty during drilled pier/driven pile installation and should be anticipated by the contractor. Specialized equipment/tooling may be necessary to penetrate the underlying bedrock. Drilled pier/driven pile parameters can be provided upon request.

As an alternate foundation/floor system (but not equal in performance), a shallow foundation/floor system consisting of spread footings and a slab-on-grade system may be utilized for the proposed structure provided it is placed on a uniform fill thickness (fill prism) constructed beneath and beyond the building footprint, and consisting of properly moisture-density treated structural fill (CDOT Class I or approved) materials, in order to reduce (but not eliminate) the potential for movement. Based on the apparent soil/grading conditions encountered in the test holes, the fill prism should extend to a depth of at least 9 feet beneath the underslab gravel layer of the slab. The prism layer should extend laterally approximately 10 feet beyond the building footprint perimeter and beneath any building appurtenances including entryways, patios, courtyards etc. Utilizing this option as well as other applicable suggestions provided in the report, GROUND anticipates potential movements on the order of 1 inch and differential movements on the order of ½ inch over a distance of 40 feet. Realized movements should be expected to exceed these estimates in localized areas and may result in structural/aesthetic damage requiring repairs. The use of native, on-site materials was also considered for use in the construction of the uniform fill prism, however. Based on our analysis, potential movements in excess of 1½ inches, were estimated to be likely.

Inadequate site drainage and/or ineffective fill processing will also result in an increase in the movement estimates provided. In addition, realized 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. In the event the earth materials supporting the proposed building's foundation and floor systems experiences moisture infiltration, post-construction movements in excess of these provided herein should be anticipated.

FOUNDATION SYSTEMS

Shallow Foundations

Geotechnical Parameters for Shallow Foundation Design

1) Footings should bear on properly compacted structural fill 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 building footprint.

   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 moisture-conditioned materials, as previously discussed, may be designed for an allowable soil bearing pressure of 1,500 psf for footings up to 12 feet in width (based on a provided maximum load of 220 kips). Utilization of a reduced soil bearing pressure will reduce the potential for post-construction foundation movements. In the event the footing width is greater than 12 feet, GROUND should be notified to reevaluate these parameters.

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

   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.

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 4 or more feet below the lowest adjacent exterior finish grades to have adequate soil cover for frost protection

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

6) Geotechnical parameters for lateral resistance to foundation loads are provided in the *Lateral 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.

9) In order to reduce differential settlements between footings or 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 properly 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 SYSTEMS

Slab-on-Grade Floors

Geotechnical Parameters for Slab-on-Grade Floors

1) Lightly loaded slabs should be placed on a minimum of **9 feet** of properly moisture conditioned and compacted structural fill materials, see *Foundation/Floor System Overview* section. The remedial fill section should extend at full depth at least 10 feet laterally beyond the slab perimeter.

2) An allowable subgrade vertical modulus (K) of 100 pci may be utilized for lightly loaded slabs supported by structural fill 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 materials should be excavated and replaced with properly compacted fill, placed in accordance with the *Project Earthwork* section of this report. All slab subgrade should be properly moisture-density treated prior to placement of concrete.

4) Slabs should be separated from all bearing walls and columns with slip joints, which allow unrestrained vertical movement.

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.

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, 6 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; movements are anticipated. 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. Where post-construction movements greater than ½ inch are not tolerable, deep foundations should be used. 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 Overview* section, any fill material should ideally be placed to a uniform depth in a properly, moisture-density treated manner prior to placement of any new structural element. Provided the owner understands that potential post-construction movements of 2 inches or more may be realized, we believe that subgrade under exterior flatwork or other (non-building) site improvements could be scarified to a depth of 12 or more inches. It has been our experience that greater overexcavation and replacement depths (i.e. 2 to 3 feet) often provides enhanced performance but at an increased initial cost. The excavated soil should be replaced as properly moisture-conditioned and compacted fill as outlined in the *Project Earthwork* section of this report. Movements will occur and distress resulting in damage and removal and replacement, should be anticipated.

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 concentration of water-soluble sulfates measured in selected samples retrieved from the test holes ranged from approximately 0.02 to 1.10 percent. Such concentrations of water-soluble sulfates represent a severe 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

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

Based on our test results and PCA and CDOT guidelines, GROUND recommends use of sulfate-resistant cement in all concrete exposed to site soil, conforming to one of the following Class 2 requirements:

(1) ASTM C 150 Type V with a minimum of a 20 percent substitution of Class F fly ash by weight

(2) ASTM C 150 Type II or III with a minimum of a 20 percent substitution of Class F fly ash by weight. The Type II or III cement shall have no more than 0.040 percent expansion at 14 days when tested according ASTM C 452

(3) ASTM C 1157 Type HS; Class C fly ash shall not be substituted for cement.

(4) ASTM C 1157 Type MS plus Class F fly ash where the blend has less than 0.05 percent expansion at 6 months or 0.10 percent expansion at 12 months when tested according to ASTM C 1012.

(5) A blend of Portland cement meeting ASTM C 150 Type II or III with a minimum of 20 percent Class F fly ash by weight, where the blend has less than 0.05 percent expansion at 6 months or 0.10 percent expansion at 12 months when tested according to ASTM C 1012.

(6) ASTM C 595 Type IP(HS); Class C fly ash shall not be substituted for cement.

When fly ash is used to enhance sulfate resistance, it shall be used in a proportion greater than or equal to the proportion tested in accordance to ASTM C 1012, shall be the same source, and it shall have a calcium oxide content no more than 2.0 percent greater than the fly ash tested according to ASTM C 1012.

All concrete exposed to site soil and bedrock should have a minimum compressive strength of 4,500 psi.

The contractor should be aware that certain concrete mix components affecting sulfate resistance including, but not limited to, the cement, entrained air, and fly ash, can affect workability, set time, and other characteristics during placement, finishing and curing. The contractor should develop mix(es) for use in project concrete which are suitable with regard to these construction factors, as well as sulfate resistance. A reduced, but still significant, sulfate resistance may be acceptable to the owner, in exchange for desired construction characteristics.

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 in 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 ranging from approximately 7.0 to 7.4.

Reduction-Oxidation testing indicated negative potentials: approximately -20 to -54 millivolts. Such low potentials typically create a more corrosive environment.

Sulfide Reactivity testing for the presence of sulfides indicated 'positive' 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 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 ranging from approximately 1,966 to 3,120 ohm-centimeters in samples of the site earth materials.

Corrosivity Assessment The American Water Works Association (AWWA, 2010[6]) 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.

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

TABLE A.1 SOIL-TEST EVALUATION

Soil Characteristic / Value	Points
Resistivity	
<1,500 ohm-cm	10
1,500 to 1,800 ohm-cm	8
1,800 to 2,100 ohm-cm	5
2,100 to 2,500 ohm-cm	2
2,500 to 3,000 ohm-cm	1
>3,000 ohm-cm	0
pH	
0 to 2.0	5
2.0 to 4.0	3
4.0 to 6.5	0
6.5 to 7.5	0 *
7.5 to 8.5	0
>8.5	3
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 ranging from approximately 11½ to 21½ in the on-site materials, the soil appears to generally comprise a corrosive environment for buried metals.

If additional information are 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 122 pcf and a phi angle of 25 degrees.

Lateral Earth Pressures (Equivalent Fluid Unit Weights)

Material Type	Water Condition	At-Rest *(pcf)*	Active *(pcf)*	Passive *(pcf)*	Friction Coefficient
On-Site Backfill	Drained	70	50	300	0.31
Structure Backfill (CDOT Class 1)	Drained	55	35	400	0.45

The upper 1 foot of embedment should be neglected for passive resistance, however. Where this passive soil pressure is used to resist lateral loads, it should be understood that significant lateral strains will be required to mobilize the full value indicated above, likely 1 inch or more. A reduced passive pressure can be used for reduced anticipated strains, however.

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.

Existing Fill Soils: Fill materials were apparently encountered in some of the test holes at the time of drilling. If encountered, these materials may not be suitable for replacement as backfill. The Geotechnical Engineer should be retained during site excavations to observe the excavated fill materials and provide parameters for its suitability for reuse.

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.

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 75 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 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 4 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. Backfilled areas adjacent to the building and other improvements should be anticipated to settle requiring re-establishment of surface grades.

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 generally encountered silty clay below exising grades. Caving could be encountered locally. Coarse, nested cobbles and boulders, as well as running sands, may be present in the subsurface materials and should be anticipated by the contractor at any depth. Variable penetration resistance values at various depths were encountered in these materials, as well.

Therefore, we anticipate some excavation difficulties in the majority of the site, even with conventional heavy-duty excavation equipment in good working condition.

As stated, the contractor should take precaution to not undermine adjacent structural elements. In the event that adjacent foundations are in close proximity or become evident during excavation, temporary shoring may be required.

Temporary, un-shored excavation slopes up to 10 feet in height be cut no steeper than 2:1 (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 not obviously encountered at the time of drilling. Should seepage or flowing groundwater be encountered in project excavations, the slopes should be flattened as necessary to maintain stability or a geotechnical engineer should be retained to evaluate the conditions and provide additional discussion or parameters, as appropriate. The risk of slope instability will be significantly increased in areas of seepage along excavation slopes.

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

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 300 psf per foot of embedment, to a maximum of 3,000 psf. Sliding friction at the bottom of thrust blocks may be taken as 0.31 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 75 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, GROUND recommends that with regard to potential migration of fines into the pipe bedding, design and installation follow ASTM D2321, Appendix X1.8. If the granular bedding does not meet filter criteria for the enclosing soils, then non-woven filter fabric (e.g., Tencate 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 can be beneficial to interrupt the granular bedding section to reduce the rates and volumes of water transmitted along utility alignments 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. (The site soils generally classified as Types IV and V bedding, per ASTM D2321, Section 5.) Materials proposed for use as pipe bedding should be approved prior to use. Imported materials should be approved 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 the 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. 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. Where 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) Other facility areas where drainage seeps into subsurface soils may be susceptible to frost heave, which can damage site improvements.

12) Maintenance as described herein may include complete removal and replacement of site improvements as well as located earthwork operations in order to maintain/re-establish effective surface drainage.

13) Detention ponds commonly are incorporated into drainage design. When a detention pond 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, GROUND suggests that 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.

Because a partial below-grade level is planned for the school building, GROUND recommends that a perimeter underdrain system be included in project drainage design (at least for portions below grade). In addition, if elements are included that extend to greater depths (e.g., an elevator pit) those features should be provided with local underdrain systems.

Because of the lateral extent of the building floor, the underdrain system also should include laterals under the building floor. The laterals should be located so that no portion of the floor is more than 75 feet from an underdrain.

A drain system should be installed at the interface of the structural and native materials as part of the fill prism construction.

Geotechnical Parameters for Underdrain Design The underdrain system(s) for the project should be designed in accordance with the recommendations below. The actual underdrain layout, outlets, and locations should be developed by a civil engineer.

The underdrain system(s) 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) The underdrain system should consist of perforated PVC collection pipe at least 4 inches in diameter, non-perforated 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 wall.

5) Damp-proofing should be applied to the exterior of the foundation wall.

6) The foundation wall also should be provided with a Tencate MiraFi® G-Series backing (or comparable wall drain provisions) on the exterior side. The 'drain board' should be installed so that it is in hydraulic continuity with the underdrain system.

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

8) The high point(s) for the collection pipe flow lines should be below the grade beam or shallow foundation bearing elevation. 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.

9) 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.

10) 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 A-4 to A-6 soils in accordance with the American Association of State Highway and Transportation Officials (AASHTO) classification system.

Based on our laboratory testing results, a resilient modulus value of 3,562 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

Specific traffic loadings were not available at the time of this report preparation. Based on our experience with similar facilities, an equivalent 18-kip daily load application (EDLA) value of 5 was assumed for the automobile-only parking lot 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. In areas of heavy truck traffic such as a drop-off lane, an equivalent 18-kip daily load application (EDLA) value of 10 was assumed. The EDLA value of 10 was converted to an equivalent 18-kip single axle load (ESAL) value of 73,000 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.

Minimum Pavement Sections

Location	***Flexible Section*** *(inches Asphalt)*	***Composite Section*** *(inches Asphalt / inches Aggregate Base)*	***Rigid Section*** *(inches Concrete)*
General Parking Areas	6	4 / 8	6
High Traffic Areas	7	4½ / 8	7

Additionally, trash collection area, 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 the *Foundation/Floor System* section, the conditions within the project site include existing man-made fill, a potential for swell and consolidation, and foundation/floor systems placed directly on the on-site materials could experience movements on the order of 2 inches (including differential and total movements). Similar movement potentials should be anticipated for pavement areas. In order to reduce the potential for post-construction movement, over-excavation and replacement of the site earth materials should be performed. Greater over-excavation depths (i.e. 2 to 3 or more feet) will result in greater long term performance but at greater initial cost. Provided the owner understands the risks identified above and accepts the potential for post-construction movement as discussed in this report, the subgrade under pavement areas could be scarified to a depth of 12 or more inches. This depth will result in movements and subsequent distress to pavement. Movements in excess of 1 inch should be anticipated. These movements will likely be even more severe if surface drainage in 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 pavement will require removal and replacement as an element of future maintenance.

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 Eagle County Schools as it pertains to the proposed Eagle Valley Elementary School 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 Fall/Winter 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 1A and 1B, 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, maintenance and repairs, as well as the limitations inherent within such estimations. <u>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.</u>

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.

Eric C. Mocko, P.E.

Reviewed by Jason A. Smith, REM, P.E.

GOOGLE EARTH AERIAL IMAGE (04/03/2015)

1A
⊕ Indicates test hole number and approximate location drilled under job number 16-3718.

1
⊕ Indicates test hole number and approximate location drilled under job number 17-3597.

(Not to Scale)

GROUND
ENGINEERING CONSULTANTS

LOCATION OF TEST HOLES

JOB NO.: 17-3597	FIGURE: 1A
CADFILE NAME: 3597SITE1.DWG	

SITE PLAN PROVIDED BY CLIENT

1A
⊕ Indicates test hole number and approximate location drilled under job number 16-3718.

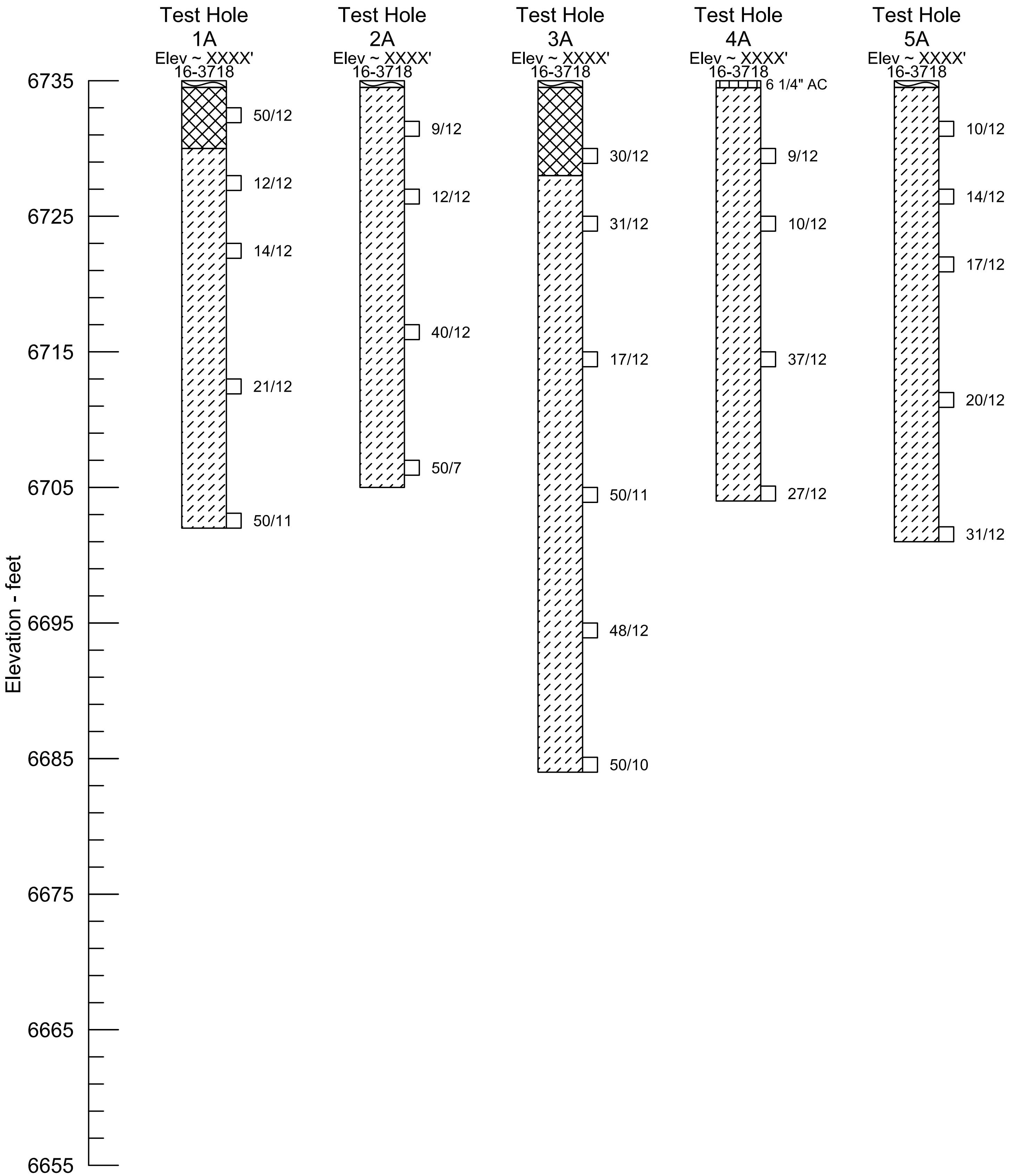
1
⊕ Indicates test hole number and approximate location drilled under job number 17-3597.

(Not to Scale)

GROUND
ENGINEERING CONSULTANTS

LOCATION OF TEST HOLES

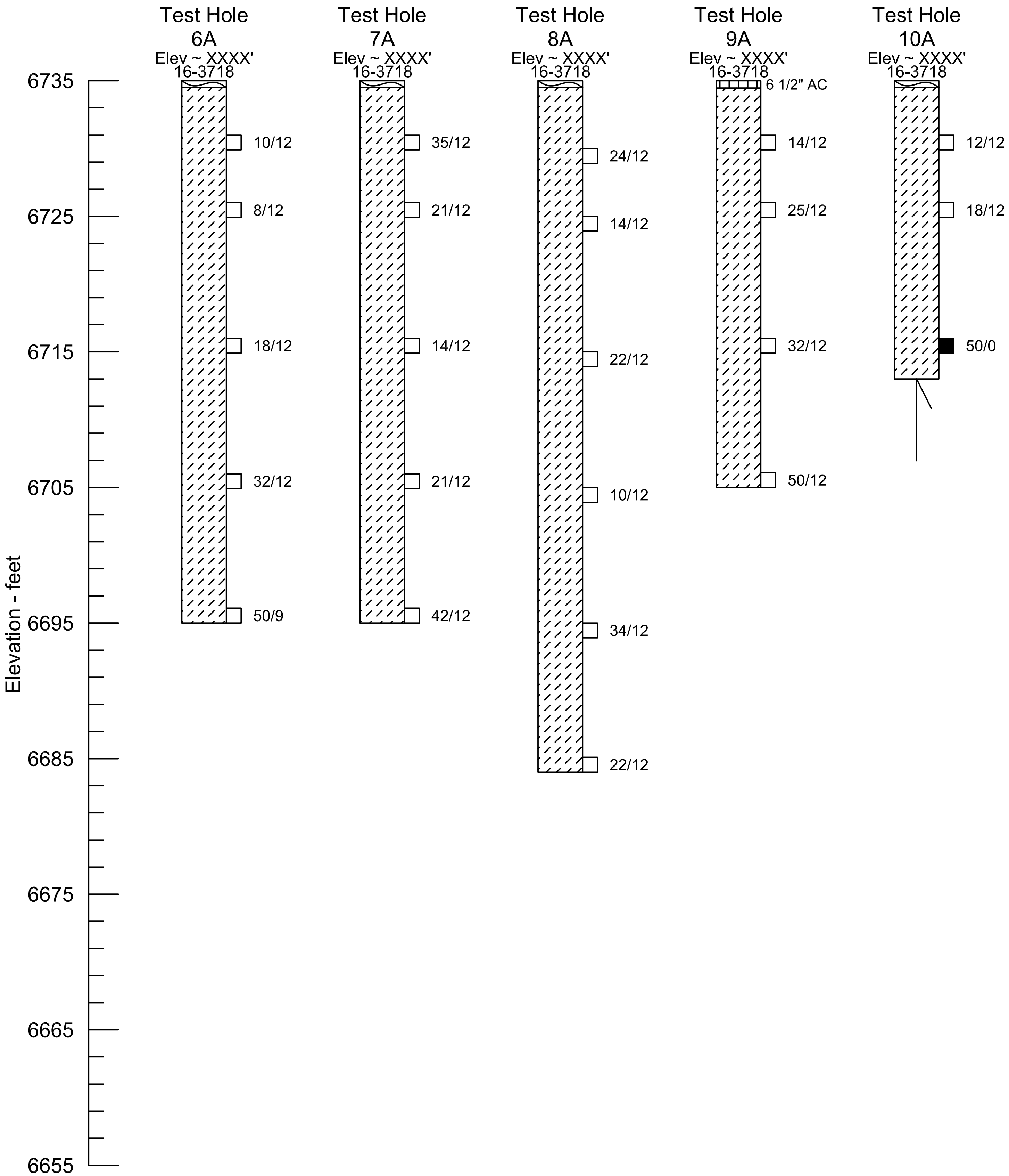
JOB NO.: 17-3597	FIGURE: 1B
CADFILE NAME: 3597SITE2.DWG	



GROUND ENGINEERING CONSULTANTS

LOGS OF TEST HOLES

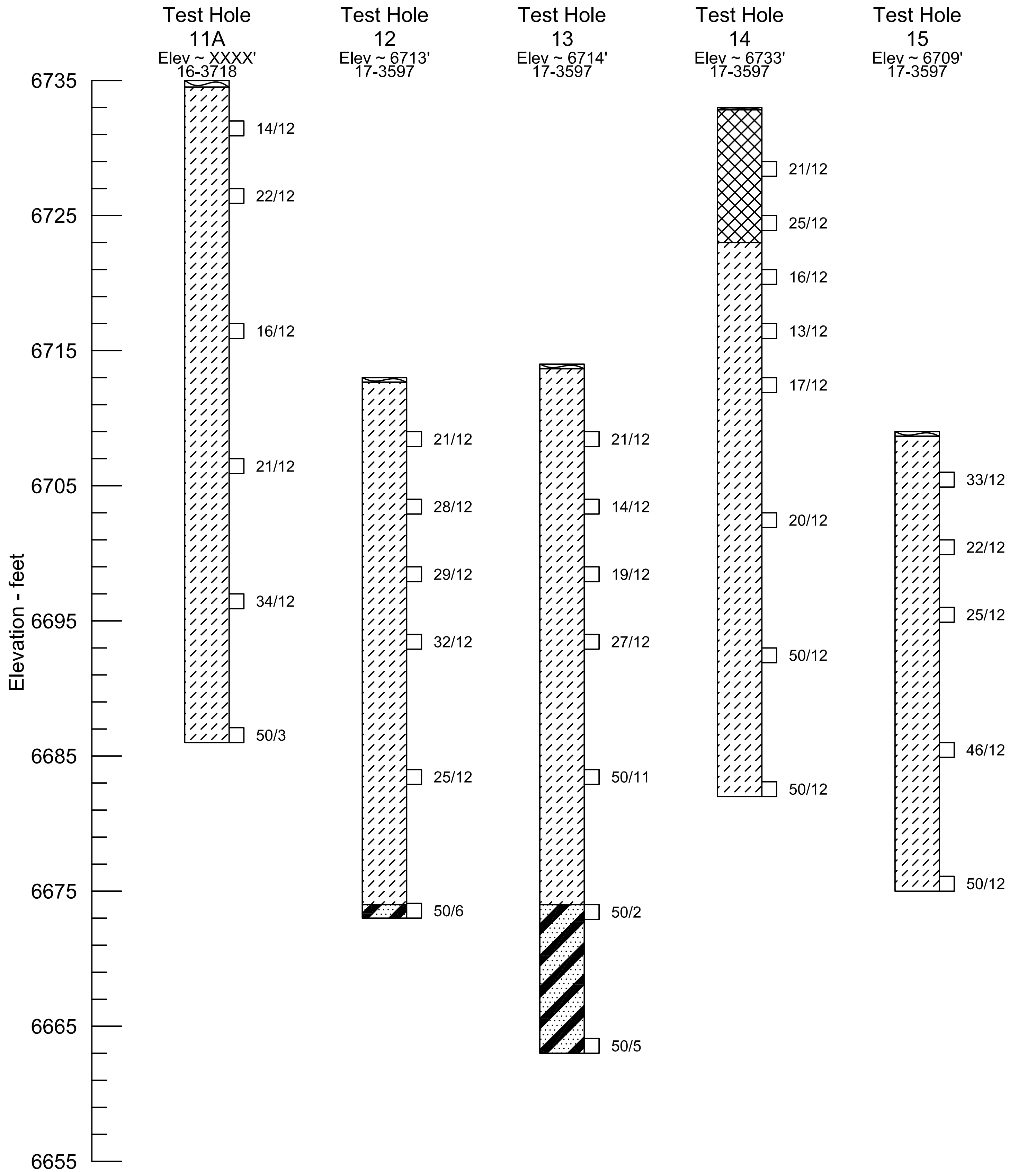
JOB NO.: 17-3597	FIGURE: 2
CADFILE NAME: 3597LOG01.DWG	



GROUND
ENGINEERING CONSULTANTS

LOGS OF TEST HOLES

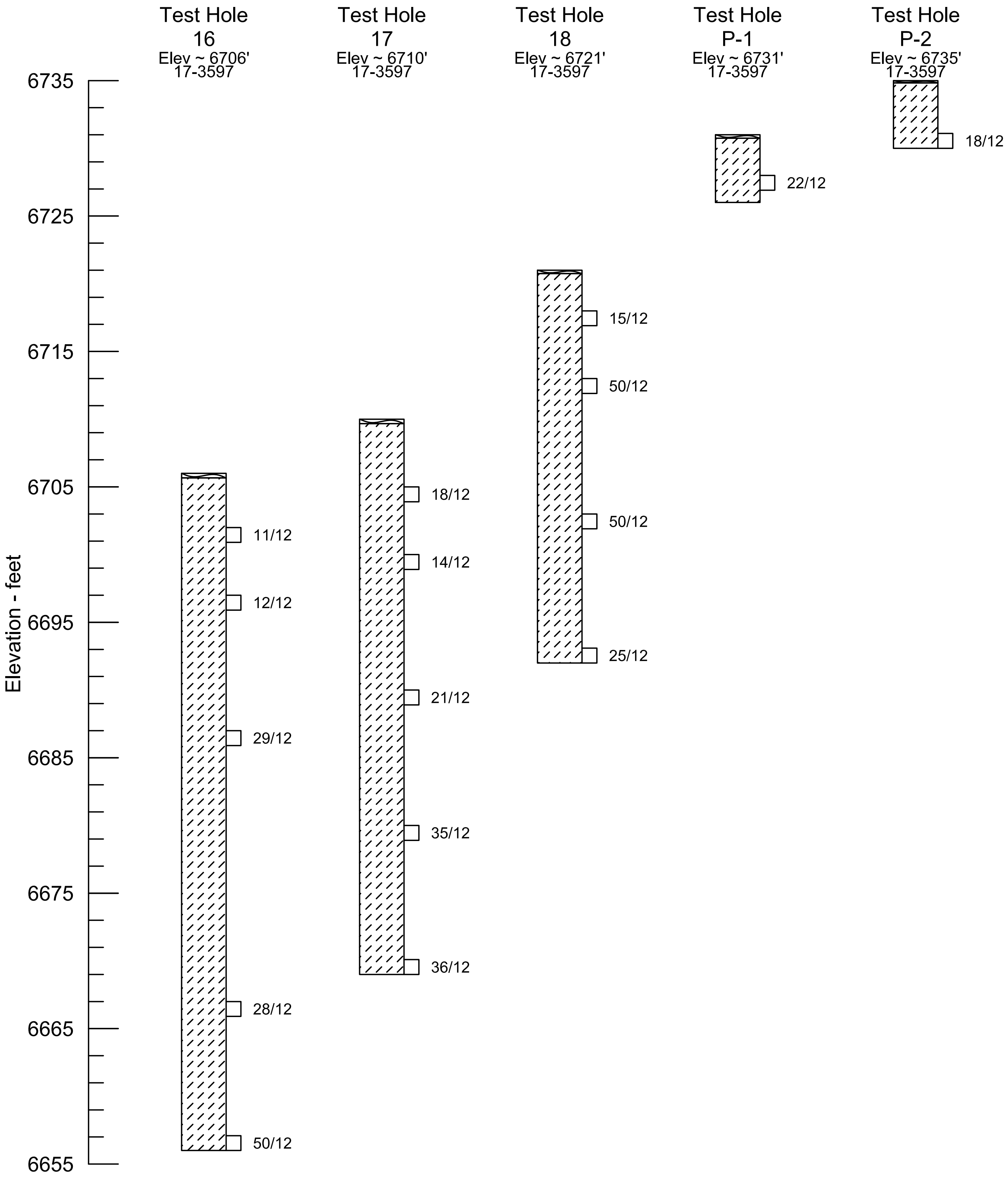
JOB NO.: 17-3597	FIGURE: 3
CADFILE NAME: 3597LOG02.DWG	



GROUND ENGINEERING CONSULTANTS

LOGS OF TEST HOLES

JOB NO.: 17-3597	FIGURE: 4
CADFILE NAME: 3597LOG03.DWG	



GROUND ENGINEERING CONSULTANTS

LOGS OF TEST HOLES

JOB NO.: 17-3597	FIGURE: 5
CADFILE NAME: 3597LOG04.DWG	

LEGEND:

Topsoil

Man-Made: Silty to sandy clay materials with sands and gravels. These materials were observed to contain some organic debris (wood). These materials were fine to gravel grained (cobbles and boulders possible), slightly moist to moist, low to moderately plastic, and light brown to dark brown in color.

Clay: Silty and somewhat interbedded with sands and gravels. These materials were fine to gravel grained (cobbles and boulders possible), slightly moist to moist, non-plastic to moderately plastic, medium to hard, somewhat calcareous, occasionally iron stained, and light brown to dark brown to olive-gray in color.

Claystone/Sandstone Bedrock: Low to moderately plastic, fine to coarse grained with some gravels, hard to very hard and slightly resistant, slightly moist to moist, occasionally caliche, and light brown to red-brown in color.

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

Drive sample, 1-3/8 inch I.D. standard 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.

20/25/30 Drive sample blow count, indicates 20, 25, and 30 blows of a 140-pound hammer falling 30 inches were required to drive the sampler 18 inches.

Practical Rig Refusal

NOTES:

1) Test holes were drilled on 12/09 and 12/12/2016, 05/17 and 05/18/2017 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 16-3718 test holes were not measured and the logs of the test holes are drawn to depth. Elevations of the 17-3597 test holes were extrapolated from client provided documents and the logs of the test holes are hung to elevation.

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

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

6) Groundwater was not encountered during drilling. Ground water levels can fluctuate seasonally and in response to landscape irrigation.

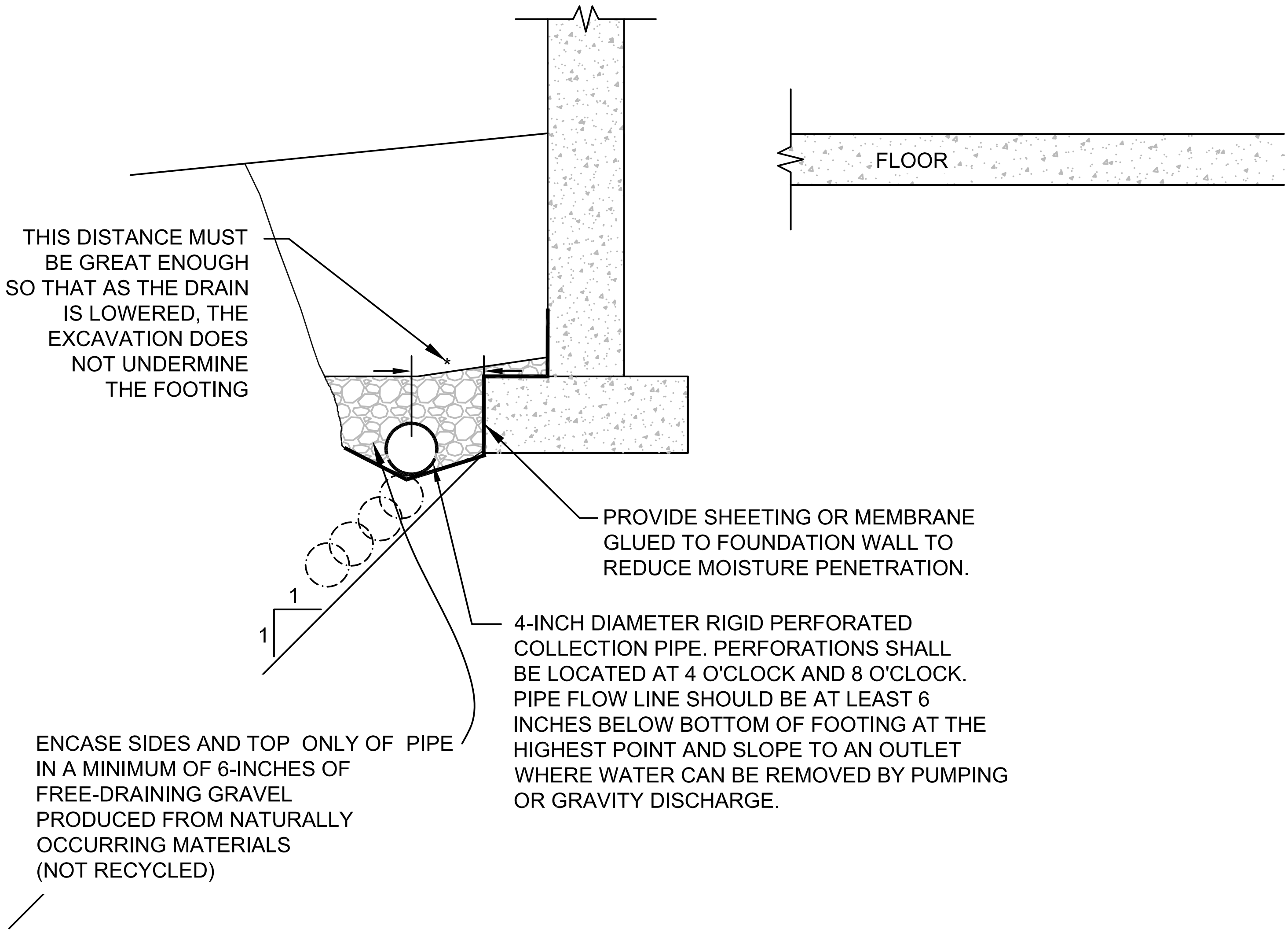
7) The material descriptions on this legend are for general classification purposes only. See the full text of this report for descriptions of the site materials and related information.

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

GROUND ENGINEERING CONSULTANTS

LEGEND AND NOTES

JOB NO.: 17-3597	FIGURE: 6
CADFILE NAME: 3597LEG.DWG	



NOTES:
1. THIS IS NOT A DESIGN-LEVEL DRAWING. IT SHOULD BE USED SOLELY FOR GENERAL INFORMATIONAL PURPOSES ONLY. ACTUAL UNDERDRAIN DESIGN SHOULD BE COMPLETED BY OTHERS.
2. THE UNDERDRAIN SYSTEM MUST BE TESTED BY THE CONTRACTOR AFTER INSTALLATION AND BACKFILLING TO VERIFY THAT IT FUNCTIONS PROPERLY.
3. INCLUSION OF THIS FIGURE IN CONSTRUCTION DOCUMENTS IS DONE SO AT THE DOCUMENT PREPARER'S RISK. REPRODUCTION OF THIS DOCUMENT MUST BE IN COLOR.
4. THIS DOCUMENT, TOGETHER WITH THE CONCEPTS AND DESIGNS PRESENTED HEREIN, AS AN INSTRUMENT OF SERVICE, IS INTENDED ONLY FOR THE SPECIFIC PURPOSE AND CLIENT WHICH IT WAS PREPARED. REUSE OF AND IMPROPER RELIANCE ON THIS DOCUMENT WITHOUT WRITTEN AUTHORIZATION AND ADAPTATION BY GROUND ENGINEERING CONSULTANTS, INC. SHALL BE WITHOUT LIABILITY TO GROUND ENGINEERING CONSULTANTS, INC.

GROUND
ENGINEERING CONSULTANTS

TYPICAL FOUNDATION
UNDERDRAIN DETAIL

JOB NO.: 17-3597	FIGURE: 7
CADFILE NAME: 3597DRAIN.DWG	

GROUND ENGINEERING

TABLE 1

SUMMARY OF LABORATORY TEST RESULTS

Sample Location		Natural	Natural	Gradation		Percent	Atterberg Limits		Percent	Unconfined	USCS	AASHTO	
Test Hole No.	Depth *(feet)*	Moisture Content *(%)*	Dry Density *(pcf)*	Gravel *(%)*	Sand *(%)*	Passing No. 200 Sieve	Liquid Limit	Plasticity Index	Swell (Surcharge Pressure)	Compressive Strength *(psf)*	Classification	Classification (GI)	Soil or Bedrock Type
1	2	15.0	115.9	-	-	84	32	13	0.4(250 psf)	-	CL	A-6(10)	Lean CLAY
1	12	22.3	98.8	-	-	96	33	11	-	-	CL	A-6(11)	Lean CLAY
2	8	16.6	107.2	11	25	64	25	4	-	-	(CL-ML)s	A-4(1)	Silty CLAY w/ Sand
2	18	16.1	102.7	34	34	32	80	53	-	-	g(SC)	A-2-7(6)	Gravelly, Clayey SAND
3	5	17.3	109.8	-	-	65	31	10	-	-	CL	A-4(5)	Lean CLAY
3	30	9.0	118.4	-	-	95	34	15	-0.2(2,000 psf)	-	CL	A-6(14)	Lean CLAY
4	10	23.4	94.3	-	-	96	34	12	-	-	CL	A-6(12)	Lean CLAY
4	20	4.2	122.3	54	28	18	27	8	-	-	(GC)s	A-2-4(0)	Clayey GRAVEL w/ Sand
5	8	19.2	106.2	-	-	89	28	10	-	-	CL	A-4(8)	Lean CLAY
5	33	14.0	116.0	5	37	58	25	8	-	5,220	s(CL)	A-4(2)	Sandy CLAY
6	4	11.0	97.2	-	-	68	23	1	-0.8(500 psf)	-	ML	A-4(0)	Sandy SILT
6	29	14.2	110.7	-	-	71	34	14	-	12,360	CL	A-6(8)	Lean CLAY
7	9	24.8	85.9	-	-	52	36	7	-	-	ML	A-4(2)	SILT
7	29	7.6	90.3	1	76	23	NV	NP	-	-	SM	A-2-4(0)	Silty SAND
8	5	15.0	108.4	-	-	80	18	1	0.0(625 psf)	-	ML	A-4(0)	SILT
8	40	19.1	108.4	-	-	74	27	9	-	-	CL	A-4(5)	Lean CLAY
9	9	16.0	114.1	-	-	77	27	10	-	-	CL	A-4(6)	Lean CLAY
9	19	11.1	114.0	-	-	88	24	8	-	-	CL	A-4(5)	Lean CLAY
10	4	15.1	107.9	-	-	69	30	8	-	-	CL	A-4(4)	Lean CLAY
10	9	15.4	105.6	-	-	82	27	9	-4.9(1,125 psf)	-	CL	A-4(6)	Lean CLAY
11	3	14.3	104.9	-	-	72	29	13	-1.2(325 psf)	-	CL	A-6(7)	Lean CLAY
11	8	10.3	115.1	-	-	47	27	7	-	-	SC-SM	A-4(1)	Silty, Clayey SAND
12	4	9.5	-	-	-	87	28	7	-1.4 (500 psf)	-	CL-ML	A-4(5)	Silty Clay
12	39	12.2	113.7	-	-	79	27	11	-	16,300	CL	A-6(1)	Lean CLAYSTONE
13	5	10.9	102.4	-	-	48	21	4	-2.6 (650 psf)	-	SC-SM	A-4(0)	Silty, Clayey SAND
13	15	20.9	102.2	-	-	95	31	8	-	-	CL	A-4(8)	Lean CLAY
14	12	25.7	93.6	-	-	77	32	9	-	1,120	CL	A-4(6)	Lean CLAY
14	16	17.1	108.2	-	-	85	27	3	-0.5 (750)	-	ML	A-4(2)	SILT
15	8	12.1	104.5	-	-	90	29	9	-1.2 (1,000)	-	CL	A-4(7)	Lean CLAY
15	23	6.3	117.2	-	-	39	25	7	-	-	SC-SM	A-4(0)	Silty, Clayey SAND
16	19	20.1	100.0	-	-	87	18	8	-	-	CL	A-4(3)	Lean CLAY
16	39	16.9	109.6	-	-	82	28	9	-	-	CL	A-4(6)	Lean CLAY
17	5	17.2	101.2	-	-	85	27	6	-	-	CL-ML	A-4(4)	Silty CLAY
17	10	17.7	102.2	-	-	91	28	7	-	-	CL-ML	A-4(6)	Silty CLAY
18	8	11.0	114.8	-	-	88	32	12	0.9 (1,000)	-	CL	A-6(10)	Lean CLAY
18	18	12.0	113.8	-	-	90	24	4	-4.4 (2,250)	-	CL-ML	A-4(2)	Silty CLAY
P1	3	17.5	108.7	-	-	56	16	5	-	-	CL-ML	A-4(0)	Silty CLAY
P2	4	22.7	101.2	-	-	67	15	NP	-	-	ML	A-4(0)	SILT
Bulk	0-10	17.5*	108.0*	-	-	79	31	11	-	-	CL	A-6(8)	Lean CLAY

NV = Non-Viscous, NP = Non-Plastic

*Indicates Optimum Moisture Content and Maximum Dry Density Per ASTM D-698

Job No. 17-3597

GROUND ENGINEERING

TABLE 2

SUMMARY OF SOIL CORROSION TEST RESULTS

Sample Location		Water Soluble Sulfates	pH	Redox Potential	Sulfides Content	Resistivity	USCS Classification	Soil or Bedrock Type
Test Hole No.	Depth *(feet)*	*(%)*		*(mV)*		*(ohm-cm)*		
1	12	1.1	7.3	-36	Positive	1,966	CL	CLAY
10	4	0.06	7.0	-20	Positive	2,969	CL	Sandy CLAY
17	10	0.02	7.4	-54	Positive	3,120	CL-ML	Silty CLAY

Job No. 17-3597