GROUND
ENGINEERING

September 19, 2018

Subject: Updated Site Plan, **Retail Building, Constitution Avenue and Marksheffel Road**, Colorado Springs, Colorado.

Job Number 15-3527

Mr. Zach Lauterbach
Evergreen Devco, Inc. and Evergreen – Constitution and Marksheffel
2390 East Camelback Road, #410
Phoenix, Arizona 85016

Dear Mr. Lauterbach:

GROUND Engineering Consultants, Inc. (GROUND) previously performed a subsurface exploration program for design and construction of the proposed Retail Buildings at Constitution Avenue and Marksheffel Road in Colorado Springs, Colorado. Based on this report, four (4) retail pads were to be overlot graded in order to deliver "pad ready" sites to future tenants. GROUND drilled one exploratory test hole within each lot and provided preliminary geotechnical parameters in our report (not to be utilized for design). According to information provided by the Client on September 14, 2018, we understand that an updated site plan indicating that the western portion of the site contains 5 lots instead of 4 lots is now planned. GROUND has updated the site plan presenting the test hole locations drilled in 2015.

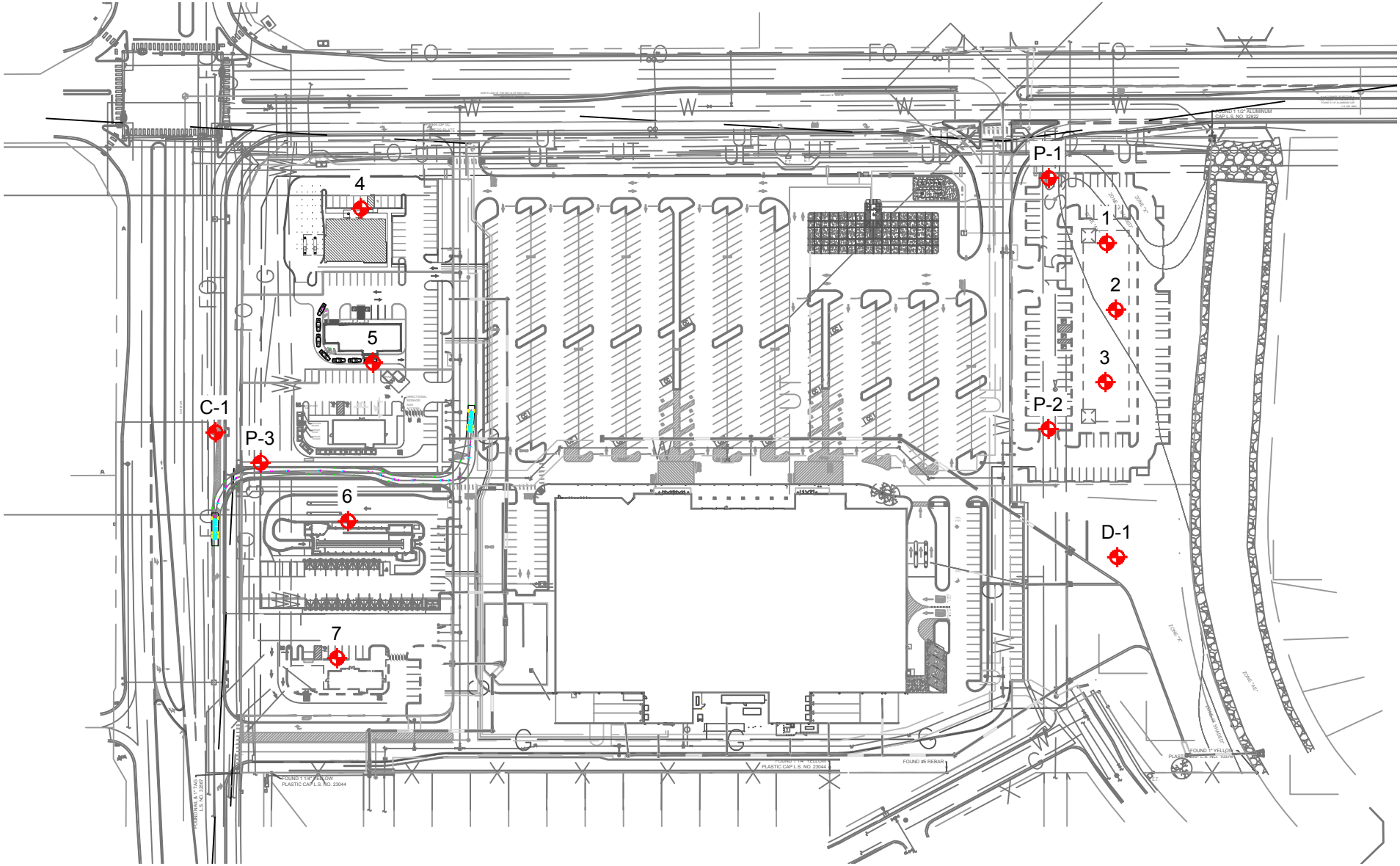
Reference is made to our June 12, 2015 report for a description of the subgrade preparation and subsurface conditions, our general geotechnical findings and parameters, and the limitations on our work, that also apply to the information provided by GROUND herein. We consider all parameters in that report not specifically superseded herein to remain valid.

Should you have any questions regarding the above-presented recommendations, please contact this office.

Sincerely,
GROUND ENGINEERING CONSULTANTS, INC.

Amy Crandall

Amy Crandall, P.E.



SITE PLAN PROVIDED BY CLIENT

1
Indicates test hole number and approximate location.

(Not to Scale)

GROUND
ENGINEERING CONSULTANTS

LOCATION OF TEST HOLES

JOB NO.: 15-3527	FIGURE: 1
CADFILE NAME: 3527SITE-R1 180919.DWG	

GROUND ENGINEERING

Geotechnical Subsurface Exploration Program
Retail Buildings
Constitution Avenue and Marksheffel Road
Colorado Springs, Colorado
Final Submittal

Prepared for:
Evergreen Devco, Inc. and Evergreen-Constitution & Marksheffel, L.L.C.
2390 East Camelback Road, #410
Phoenix, Arizona 85016

Attention: Mr. Russell Perkins

Job Number: 15-3527 **June 12, 2015**

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TABLE OF CONTENTS

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 Retail Buildings that will be constructed near the intersection of Constitution Avenue and Marksheffel Road in Colorado Springs, Colorado. Our study was conducted in general accordance with Agreement between Evergreen Devco, Inc. and GROUND dated March 24, 2015 and GROUND's Proposal Number 1501-0094 Revised, dated February 25, 2015.

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

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

PROPOSED CONSTRUCTION

We understand that proposed construction will consist of a single-story in-line retail facility, approximately 15,000 square feet in building footprint. Additionally, four (4) retail pads will be overlot graded in order to deliver "pad ready" site to future tenants. A detention pond and improvements to Marksheffel Road (i.e curb cut,/right-in/right-out entrance/exit) will also be performed. Public pavement design in accordance with the City of Colorado Springs was not included in this scope of services. Based on provided grading information, a finish floor elevation of approximately 6,444 feet is planned for the proposed inline retail structure. Therefore, material cuts up to approximately 2 feet and fills up to approximately 4 feet appear to be necessary to facilitate grading operations for the proposed structure. Building loads were unavailable at the time of this report preparation, but are anticipated to be light. The project site is shown in Figure 1. Development will also include installation of underground utilities to service the proposed structures. If proposed construction, including the anticipated site grading, differ from

those described above, or changes subsequently, GROUND should be notified to re-evaluate the information in this report.

SITE CONDITIONS

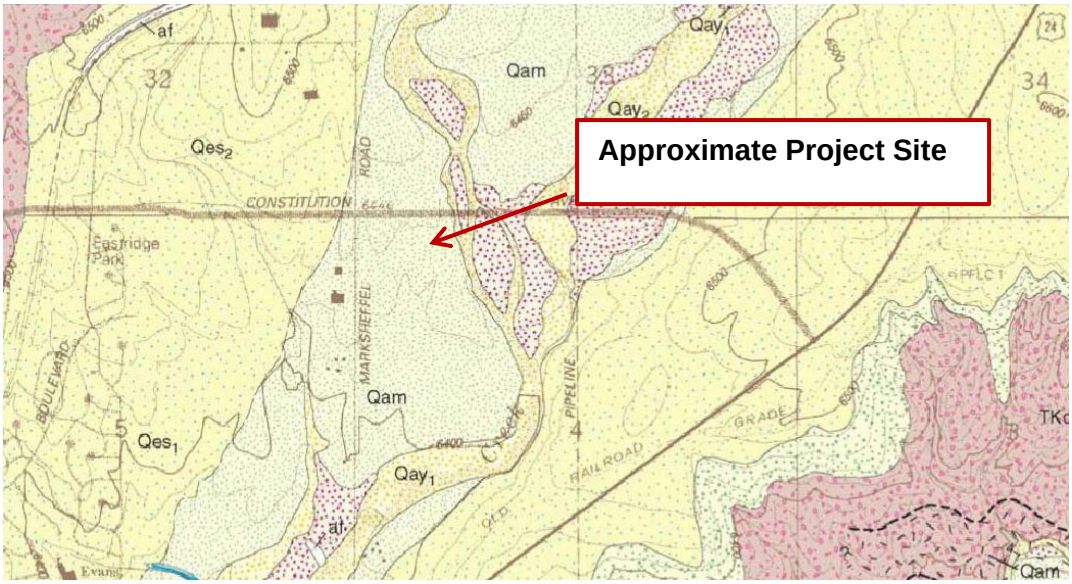
At the time of our exploration, the project site existed primarily as vacant, undeveloped land with short to medium grasses and weeds, with sparse patches of trees and bushes. Dirt paths traversed the site running north-south, east-west, and winding diagonally. The topography across the site vicinity was gently sloping to hilly consisting of several feet of elevation difference across the site and slopes up to approximately 6 percent descending toward the south. The project site is generally bound to the north by Constitution Avenue, to the west by Marksheffel Road, and to the east and south by residential neighborhoods.

Man-made fill materials were not obviously observed in the test holes at the time of drilling. The exact extents, limits, and composition of any man-made fill were not determined as part of the scope of work addressed by this study and should be expected to potentially exist at varying depths and locations across the site.

GEOLOGIC SETTING

The subject parcel lies within the Denver Basin geologic province that consists largely of a sequence of sedimentary rock formations deposited and preserved in a structural depression in north-central Colorado. In the general project area, these sedimentary rocks dip eastward at low angles (less than 10 degrees, typically) and are overlain by a variety of surficial deposits including alluvial (stream-laid) sediments, eolian (wind-blown) materials and colluvial (slope-wash) deposits.

Published maps, e.g. Thorson (2002[1]), depict the site as underlain by the Late Pleistocene Middle Alluvium (Qam), Late Holocene Young Alluvium One (Qay_1), and Late to Middle Holocene Young Alluvium Two (Qay_2). These deposits consist of light brownish-gray, pale-brown, light yellowish brown and grayish brown, poorly sorted sand, silty sand, and subordinate amounts of gravel. The alluvium materials are underlain by the Upper Cretaceous to Eocene Dawson Formation – Facies unit two ($Tkda_2$). In the project vicinity, available information indicates that this formation consists predominately of brownish-gray, yellowish-gray, and light yellow brown, pebbly sandstone interbedded with yellowish gray to grayish green fine to coarse grained micaceous sandstone and sandy claystone.



SUBSURFACE EXPLORATION

The subsurface exploration for the project was conducted on March 23 and 24, 2015. A total of eleven (11) test holes were drilled with a truck-mounted, continuous flight power auger rig to evaluate the subsurface conditions as well as to retrieve soil and bedrock samples for laboratory testing and analysis. Of these, three (3) test holes were drilled within the proposed retail building footprint limits, two (2) test holes were drilled within the proposed private pavement areas, one test hole was drilled within the proposed detention pond, and one (1) test hole was drilled within the proposed entrance/exit of

[1] Thorson, J.P., 2002, Geologic Map of the Pikeview 7.5 Minute Quadrangle, El Paso County, Colorado, Colorado Geological Survey Open-File Report OF01-03.

Marksheffel Road. The remaining four (4) test holes were drilled within the four pads (one per pad) in order to provide preliminary geotechnical findings and design parameters. The test holes were drilled to depths ranging from approximately 30 to 40 feet below existing grades. A representative of GROUND directed the subsurface exploration, logged the test holes in the field, and prepared the soil and bedrock samples for transport to our laboratory. An asphalt core within Marksheffel Road was also performed to determine actual in-place thickness.

Samples of the subsurface materials were retrieved with a 2-inch I.D. California liner sampler. The sampler was driven into the substrata with blows from a 140-pound hammer falling 30 inches. This procedure is similar to the Standard Penetration Test described by ASTM Method D1586. Penetration resistance values, when properly evaluated, indicate the relative density or consistency of soils. Depths at which the samples were obtained and associated penetration resistance values are shown on the test hole logs.

The approximate locations of the test holes are shown in Figure 1. Logs of the exploratory test holes are presented in Figures 2 to 4. Explanatory notes and a legend are provided in Figure 5. GROUND utilized the Client-provided site plan indicating existing features, Google Map imagery and a hand-held GPS to approximately locate the test holes.

LABORATORY TESTING

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

SUBSURFACE CONDITIONS

The subsurface conditions encountered in the test holes generally consisted of a thin veneer of topsoil, approximately 2 to 4 inches thick, underlain by sand. The test holes extended to the foundation test hole termination depths of approximately 30 to 40 feet below the existing grades. In the geotechnical report for the King Soopers facility which GROUND performed in conjunction with this project, the overburden materials were underlain by sandstone bedrock encountered at a depth of approximately 35 feet (elevations of 6,397 feet) below existing grades in one of the test holes. Topsoil thickness was solely visually estimated; and was not laboratory tested for plant/vegetation growth variability. The asphalt thickness of Marksheffel Road was approximately 5½ inches thick underlain by aggregate base course. The thickness of the aggregate base course was not included in our scope of services.

Sand was silty, with interbedded layers of silt, was fine to coarse grained with occasional gravel, non-plastic to medium plastic, loose to dense, slightly moist to moist, brown to tan in color, and occasionally iron stained.

Groundwater was encountered in the test holes at depths of approximately 31 to 36 feet below existing grades at the time of drilling and in Test Hole 1 at a depth of approximately 34 feet below existing when measured 7 days later. Groundwater levels can be expected to fluctuate, however, in response to annual and longer-term cycles of precipitation, irrigation, surface drainage, nearby rivers and creeks, land use, and the development of transient, perched water conditions.

Swell-Consolidation Testing of samples of the on-site materials encountered in the project test holes indicated a potential for consolidation (See Table 1). Consolidations ranging from approximately 0.3 to 0.5 percent were measured at surcharge loads ranging from 400 to 500 psf.

Permeability Testing was conducted on a California liner sample obtained from Test Hole D1 at a depth of approximately 3 feet below existing grade. The permeability value determined was $2x10^{-6}$ cm/sec.

ENGINEERING SEISMICITY

According to the 2012 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://geohazards.usgs.gov/designmaps/us/application.php) and site latitude/longitude coordinates of 38.866942 and –104.680306 (obtained from Google Earth), respectively, the project area is indicated to possess an S_{DS} value of 0.184 and an S_{D1} value of 0.095.

Per 2012 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 2012 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.

The largest recorded earthquake (estimated magnitude 6.2 to 6.6) in Colorado occurred in November 1882. While the specific location of this earthquake is very uncertain, it is postulated to have occurred in the Front Range near Rocky Mountain National Park.

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

FOUNDATION/FLOOR SYSTEMS OVERVIEW

Retail Building – East Side

As stated, building loads were unavailable at the time of this report preparation. According to the provided finish floor elevation of 6,444 feet for the proposed retail structure along the east side of the site, it appears that material fills up to approximately 4 feet and cuts up to approximately 2 feet will be necessary resulting in an increased potential for differential movement as the building would not be placed on a uniform thickness of fill material. GROUND estimates that shallow foundations and floor systems placed directly on the on-site materials could experience 1 to 1½ or more of movement (including differential and total movements). For the least potential of post-construction movement, a deep foundation system consisting of straight-shaft drilled piers advanced into the underlying bedrock should be utilized, provided with a structural floor system. This will result in the least risk of movement (approximately ½-inch). Given that the depth to bedrock ranges from approximately 35 to greater than 40 feet below existing grade, a deep foundation/structural floor option may not be desired for this project. In the event a deep foundation system is desired, GROUND should be contacted to provide applicable parameters.

As an alternate foundation/floor system (but not equal in performance), spread footings with a slab-on-grade floor system could be utilized provided the Owner is aware of the

potential for post-construction movement, as stated herein, and accepts the risks associated with potential post-construction movements. In order to reduce (but not eliminate) the potential of movement, including differential movement, as a result of variable fill depths, a uniform fill thickness (fill prism) consisting of approximately 4 feet should be constructed beneath the underslab gravel layer of the slab. This will result in approximately 1 foot of processed fill beneath the shallow foundations. This fill prism layer should extend laterally approximately 5 feet beyond the building and beneath any building appurtenances including entryways, patios, courtyards, etc. Utilizing this option as well as other applicable suggestions provided in this 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 may be more or less.

Shallow foundations consisting of spread footings with a slab-on-grade floor system placed directly on the on-site materials following overlot grading operations (no overexcavation/fill prism) may be considered provided the Owner is aware of the potential for movement, as stated herein, and accepts the risks associated with the post-construction movement. In addition, the upper 12 inches (or more) below the slabs should be scarified, moisture-conditioned, and re-compacted in accordance with the *Project* Earthwork section of our report following the overlot grading operations. Utilizing this option as well as other applicable parameters provided in this report, GROUND anticipates potential movements ranging from approximately 1 to 1½ inches and differential movements on the order of 1 inch over a distance of 40 feet. Realized movements may be more or less.

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

<u>Preliminary Geotechnical Foundation/Floor Systems – Retail Pads (western portion)</u>

Regarding the retail pads on the western portion of the project site, preliminary grading suggest that the proposed structures may include finish floor elevations ranging from

approximately 6,439 to 6,445 feet resulting in variable thicknesses of material fills (up to approximately 8 feet) increasing the potential for differential movement as the proposed building(s) would not be placed on a uniform thickness of fill material. For the least potential for movement, a deep foundation system consisting of drilled piers with a structural floor system should be constructed. As stated above, given that the depth to bedrock ranges from approximately 35 to greater than 40 feet below existing grade, a deep foundation/structural floor option may not be desired for this project. If a shallow foundation system and slab-on-grade floor system is desired, ideally, a uniform fill prism would be placed beneath the entire structure to the depth of the greatest fill thickness as a result of grading operations. Lesser depths of overexcavation and replacement across the building footprint may be considered provided the Client/Owner and future tenant understands the potential for differential movement as discussed previously. ***Discussions/parameters provided below for the retail building on the east side of the site may be utilized for preliminary/informational purposes only, provided it is understood that a final geotechnical exploration program and subsequent reports for each retail structure must be performed prior to construction for building-specific foundation and floor system.***

FOUNDATION SYSTEMS

Shallow Foundations

The geotechnical parameters indicated below may be used for design of shallow foundations for the proposed retail structure.

Geotechnical Parameters for Shallow Foundation Design

1) Footings for the retail building on the east side of the site should bear on a uniform fill prism of at least 1 foot of properly compacted on-site generated materials or approved import materials, as discussed in the *Foundation/Floor System Overview* section. As stated, for an increased potential of movement, footings for these structures may bear directly on the existing on-site materials (no overexcavation/fill prism).

   The fill section should extend should extend at full thickness across the structure/slab footprint and at least 5 feet laterally beyond the perimeter(s).

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.

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

2) Footings may be designed for an allowable soil bearing pressure of 2,000 psf for footings up to 6 feet in width.

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 from direct compression of the foundation soils <u>is in addition to</u> movements from expansive soils heave, consolidation, and/or collapse of hydro-compressive soils.

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

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

4) Footings/structures (slabs) should bear at an elevation 3 or more feet below the lowest adjacent exterior finish grades to have adequate soil cover for frost protection

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

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

Shallow Foundation Construction

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

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

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

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

12) All foundation subgrade should be properly moisture-density treated prior to placement of concrete.

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

FLOOR SYSTEMS

Slab-on-Grade Floors

Geotechnical Parameters for Slab-on-Grade Floors

1) Prior to placement of lightly loaded slabs for the retail building, construction of a uniform fill prism consisting of at least 4 feet properly compacted on-site generated materials or approved import materials, as discussed in the *Foundation/Floor System Overview* section should be performed below the bottom of the underslab gravel layer. The fill section should extend at full depth at least 5 feet laterally beyond the slab perimeter.

   As stated, for an increased potential of movement, the upper 12 inches (or more) below the slabs can be scarified, moisture-conditioned, and re-compacted in accordance with the *Project* Earthwork section of our report following overlot grading operations.

2) An allowable subgrade vertical modulus (K) of 100 pci may be utilized for lightly loaded slabs supported by the site soils.

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 depending upon the fill prism section selected by the owner.

10) Moisture can be introduced into a slab subgrade during construction and additional moisture will be released from the slab concrete as it cures. Placement of a properly compacted layer of free-draining gravel, 4 or more inches in thickness, beneath the slabs should be performed. This layer will help distribute floor slab loadings, ease construction, reduce capillary moisture rise, and aid in drainage. The free-draining gravel should contain less than 5 percent material passing the No. 200 Sieve, more than 50 percent retained on the No. 4 Sieve, and a maximum particle size of 2 inches.

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

12) Floor slab cracking can result from a variety of factors including the slab system design, the concrete mix characteristics, the weather conditions at the time of placement, the curing procedures followed, the timing of joint cutting, differential settlement, etc., or a combination of these factors. Typically, in our experience, such cracks represent only a minor aesthetic concern or a minor nuisance for facility operations. If commonly developed slab cracking is a more significant concern for this project, however, then additional measures – geotechnical, design, and construction quality control – may be necessary. In such a case, the slab performance expectations should be conveyed clearly to the floor slab designer so that the concrete mix, slab reinforcement, joint layout, curing process, etc., can be adjusted accordingly. GROUND can provide additional geotechnical information regarding remedial earthwork in support of an enhanced slab design upon request.

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

MECHANICAL ROOMS/MECHANICAL PADS

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

EXTERIOR FLATWORK

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

Provided the owner understands the risks identified above, 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.

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

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

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

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

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

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

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

In GROUND's experience the measures below can be beneficial for reducing the likelihood of concrete scaling. It must be understood, however, that because of the other factors involved, including weather conditions and workmanship, surface damage to concrete can develop, even where all of these measures were followed. Also, the mix design criteria should be coordinated with other project requirements including the

criteria for sulfate resistance presented in the *Water-Soluble Sulfates* section of this report.

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

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

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

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

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

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

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

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

Frost and Ice Considerations Nearly all soils other than relatively coarse, clean, granular materials are susceptible to loss of density if allowed to become saturated and exposed to freezing temperatures and repeated freeze – thaw cycling. The formation of ice in the underlying soils can result in heaving of pavements, flatwork and other hardscaping ("frost heave") in sustained cold weather up to 2 inches or more. This heaving can develop relatively rapidly. A portion of this movement typically is recovered

when the soils thaw, but due to loss of soil density, some degree of displacement will remain. This can result even where the subgrade soils were prepared properly.

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

WATER-SOLUBLE SULFATES

The concentrations of water-soluble sulfates measured in selected samples retrieved from the test holes was less than the detectable limit of 0.01 percent by weight (See Table 2). Such concentration of water-soluble sulfates represents a negligible environment for sulfate attack on concrete exposed to these materials. Degrees of attack are based on the scale of ‘negligible,’ ‘moderate,’ ‘severe’ and ‘very severe’ as described in the “Design and Control of Concrete Mixtures,” published by the Portland Cement Association (PCA). The Colorado Department of Transportation (CDOT) utilizes a corresponding scale with 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 these data GROUND, makes no suggestions for use of a special, sulfate-resistant cement in project concrete.

SOIL CORROSIVITY

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

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

pH Where pH is less than 4.0, soil serves as an electrolyte; the pH range of about 6.5 to 7.5 indicates soil conditions that are optimum for sulfate reduction. In the pH range above 8.5, soils are generally high in dissolved salts, yielding a low soil resistivity (AWWA, 2010). Testing indicated pH values ranging from approximately 8.2 and 8.6.

Reduction-Oxidation testing indicated negative potentials: approximately -69 and -94 millivolts. Such low potentials typically create a more corrosive environment.

Sulfide Reactivity testing for the presence of sulfides indicated 'trace' results. The presence of sulfides in the site soils also suggests a more corrosive environment.

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

Measurements of electrical resistivity indicated values of approximately 1,430 and 6,690 ohm-centimeters in samples of the site earth materials. The following table presents the relationship between soil resistivity and a qualitative corrosivity rating (ASM, 2003) [2].

Corrosivity Ratings Based on Soil Resistivity

Soil Resistivity (ohm-cm)	***Corrosivity Rating***
>20,000	Essentially non-corrosive
10,000 – 20,000	Mildly corrosive
5,000 – 10,000	Moderately corrosive
3,000 – 5,000	Corrosive
1,000 – 3,000	Highly corrosive
<1,000	Extremely corrosive

Corrosivity Assessment The American Water Works Association (AWWA, 2010[3]) has developed a point system scale used to predict corrosivity. The scale is intended for protection of ductile iron pipe but is valuable for project steel selection. When the scale equals 10 points or higher, protective measures for ductile iron pipe are suggested. The AWWA scale (Table A.1 Soil-test Evaluation) is presented below. The soil characteristics refer to the conditions at and above pipe installation depth.

[2] ASM International, 2003, *Corrosion: Fundamentals, Testing and Protection,* ASM Handbook, Volume 13A.
[3] 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 <u>and</u> low or negative redox-potential results (< 50 mV) are obtained, add three points for this range.

The redox potential of a soil is significant, because the most common sulfate-reducing bacteria can only live in anaerobic conditions. A negative redox potential indicates anaerobic conditions in which sulfate reducers thrive. A positive sulfide reaction reveals a potential problem caused by sulfate-reducing bacteria. Anaerobic conditions are regarded as potentially corrosive.

Based on a maximum possible score of 25.5 using the AWWA method, the value of 10 for the use of corrosion protection, and scores of approximately 10 to 17 in the on-site

soil, the soil appears to comprise a potentially low to moderately 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 use of passive pressure under a saturated condition is not suggested. The values for the on-site material in the upper 10 feet provided in the table below were approximated utilizing a unit weight of 119 pcf and a phi angle of 32 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	56	37	389	0.42
Structure Backfill (CDOT Class 1)	Drained	55	35	400	0.45

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

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

PROJECT EARTHWORK

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

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

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

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

Existing Fill Soils: Man-made fill materials were not obviously encountered in the test holes at the time of drilling. Actual contents and composition of the man-made fill materials are not known; therefore, some of the excavated man-made fill 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 30 percent passing the No. 200 Sieve and should have a plasticity index less than 10.** Representative samples of the materials proposed for import should be tested and approved prior to transport to the site.

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

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

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

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

Soils that classify as GP, GW, GM, GC, SP, SW, SM, or SC in accordance with the USCS classification system (granular materials) should be compacted to 95 or more percent of the maximum modified Proctor dry density at moisture contents within 2 percent of optimum moisture content as determined by ASTM D1557.

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

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

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

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

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

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

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

It is GROUND's opinion that wherever possible, excavations be backfilled with approved, on-site soils placed as properly compacted fill. Where this is not feasible, use of "Controlled Low Strength Material" (CLSM), i.e., a lean, sand-cement slurry ("flowable fill") or a similar material for backfilling should be considered.

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

Settlements: Settlements will occur in filled ground, typically on the order of 1 to 2 percent of the fill depth. If fill placement is performed properly and is tightly controlled, in GROUND's experience the majority (on the order of 60 to 80 percent) of that settlement will typically take place during earthwork construction, provided the contractor achieves the compaction levels herein. The remaining potential settlements likely will take several months or longer to be realized, and may be exacerbated if these fills are subjected to changes in moisture content.

Cut and Filled Slopes: Permanent site slopes supported by on-site soils up to 10 feet in height may be constructed no steeper than 3:1 (horizontal : vertical). Minor raveling or surficial sloughing should be anticipated on slopes cut at this angle until vegetation is well re-established. Surface drainage should be designed to direct water away from slope faces.

EXCAVATION CONSIDERATIONS

The test holes for the subsurface exploration were excavated to the depths indicated by means of truck-mounted, flight auger drilling equipment. We anticipate no significant excavation difficulties in the majority of the site with conventional heavy-duty excavation equipment in good working condition.

Temporary, un-shored excavation slopes up to 10 feet in height be cut no steeper than 1½:1 (horizontal : vertical) in the site soils in the absence of seepage. Sloughing on the slope faces should be anticipated at this angle. Local conditions encountered during construction, such as groundwater seepage and loose sand, will require flatter slopes. Stockpiling of materials should not be permitted closer to the tops of temporary slopes than 5 feet or a distance equal to the depth of the excavation, whichever is greater.

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

Groundwater was encountered in the test holes at depths of approximately 31 to 36 feet below existing grades at the time of drilling and in Test Hole 1 at a depth of approximately 34 feet below existing when measured 7 days later. Based on the likely depths of excavation, groundwater is not anticipated to be a significant factor for shallow earthworks during construction of this project. If seepage or groundwater is encountered in shallow project excavations, the Geotechnical Engineer should be retained to evaluate the conditions and provided additional parameters, as appropriate.

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. Project excavations and shoring should be observed regularly by the Geotechnical Engineer throughout construction operations. 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 380 psf per foot of embedment, to a maximum of 3,800 psf. Sliding friction at the bottom of thrust blocks may be taken as 0.42 times the vertical dead load.

***Trench Backfillin*g**: 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 30 percent passing the No. 200 Sieve and should have a plasticity index of less than 10. Representative samples of the materials proposed for import should be tested and approved prior to transport to the site.

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

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

As discussed above, we suggest the use of CLSM or similar material in lieu of granular bedding and compacted soil backfill where the tolerance for surface settlement is low.

(Placement of CLSM as bedding to at least 12 inches above the pipe can protect the pipe and assist construction of a well-compacted conventional backfill although possibly at an increased cost relative to the use of conventional bedding.)

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

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

SURFACE DRAINAGE

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

The following drainage measures should be incorporated as part of project design and during construction. The facility should be observed periodically to evaluate the surface drainage and identify areas where drainage is ineffective. Routine maintenance of site

drainage should be undertaken throughout the design life of the project. If these measures are not implemented and maintained effectively, the movement estimates provided in this report could be exceeded.

1) Wetting or drying of the foundation excavations and underslab areas should be avoided during and after construction as well as throughout the improvements' design life. Permitting increases/variations in moisture to the adjacent or supporting soils may result in a decrease in bearing capacity and an increase in volume change of the underlying soils, and increased total and/or differential movements.

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

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

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

3) Drainage should be established and maintained to direct water away from sidewalks and other hardscaping as well as utility trench alignments. Where the ground surface does not convey water away readily, additional post-construction movements and distress should be anticipated.

4) In GROUND's experience, it is common during construction that in areas of partially completed paving or hardscaping, bare soil behind curbs and gutters, and utility trenches, water is allowed to pond after rain or snow-melt events. Wetting of the subgrade can result in loss of subgrade support and increased settlements / increased heave. By the time final grading has been completed,

significant volumes of water can already have entered the subgrade, leading to subsequent distress and failures. The contractor should maintain effective site drainage throughout construction so that water is directed into appropriate drainage structures.

5) On some sites, slopes may descend toward buildings locally. Such slopes can be created during grading even on comparatively flat sites. 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) Maintenance as described herein may include complete removal and replacement of site improvements in order to maintain effective surface drainage.

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

SUBSURFACE DRAINAGE

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

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

If, however, below-grade or partially below-grade level(s) are added, a perimeter underdrain system should be included. Damp-proofing should be applied to the exteriors of below-grade elements. The provision of Tencate MiraFi® G-Series backing

(or comparable wall drain provisions) on the exteriors of (some) below-grade elements may be appropriate, depending on the intended use. If a (partially) below-grade level is limited in extent, the underdrain system, etc., may be local to that area.

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

1) 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.

2) 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.

3) 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. Gravel should not consist of recycled concrete materials.

4) 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.

5) The waterproof membrane should underlie the gravel and pipe, and be attached to the foundation wall.

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

7) 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.

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

9) 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.

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

The collection and discharge pipe for the underdrain system should be laid on a slope sufficient for effective drainage, but a minimum of 1 percent. (Flatter gradients may be used but will convey water less efficiently and entail an increased risk of local post-construction movements.)

Pipe gradients also should be designed to accommodate at least 1 inch of differential movement after installation along a 50-foot run.

11) 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.

12) 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.

13) Underdrain systems should be periodically inspected and flushed/cleaned as necessary. Maintenance/repairs should be performed to ensure proper performance.

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

CBR testing (ASTM D 1883) was performed on a remolded sample after being submerged in water for a period of 96 hours for the King Soopers report conducted in conjunction with this project. CBR testing indicated a value of 12.1 percent at approximately 95 percent of the maximum modified dry density. This CBR value correlates to a resilient modulus value of 10,367 psi based on AASHTO guidelines.

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 10 was assumed for standard-duty traffic areas. 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. In areas of heavy-duty traffic areas, such as the entrance/exit of Marksheffel Road, an equivalent 18-kip daily load application (EDLA) value of 30 was assumed. The EDLA value of 30 was converted to an equivalent 18-kip single axle load (ESAL) value of 219,000 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 2.0 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)*
Standard-Duty Pavement	5	4 / 6	6
Heavy-Duty Pavement	6	4½ / 6	8

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 naturally occurring native 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

Shortly before placement of pavement in all pavement areas, including aggregate base, the exposed subgrade soils should be excavated and/or processed to a depth of at least 12 inches, mixed to achieve a uniform moisture content and then re-compacted in accordance with the recommendations provided in the *Project Earthwork* section of this report. Subgrade preparation should extend the full width of the pavement from back-of-curb to back-of-curb. Greater depths of subgrade processing will further reduce potential pavement movements.

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 Evergreen Devco, Inc. and Evergreen Constitution and Marksheffel, LLC as it pertains to the proposed retail facilities 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 2015. Any changes in project plans or schedule should be brought to the attention of the Geotechnical Engineer, in order that the geotechnical parameters may be re-evaluated and, as necessary, modified.

The geotechnical conclusions and information in this report relied upon subsurface exploration at a limited number of exploration points, as shown in Figure 1, as well as the means and methods described herein. Subsurface conditions were interpolated between and extrapolated beyond these locations. It is not possible to guarantee the subsurface conditions are as indicated in this report. Actual conditions exposed during construction may differ from those encountered during site exploration.

If during construction, surface, soil, bedrock, or groundwater conditions appear to be at variance with those described herein, the Geotechnical Engineer should be advised at once, so that re-evaluation of the information may be made in a timely manner. In addition, a contractor who relies upon this report for development of his scope of work or cost estimates may find the geotechnical information in this report to be inadequate for his purposes or find the geotechnical conditions described herein to be at variance with his experience in the greater project area. The contractor is responsible for obtaining the additional geotechnical information that is necessary to develop his workscope and cost estimates with sufficient precision. This includes current depths to groundwater, etc.

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

ALL DEVELOPMENT CONTAINS INHERENT RISKS. It is important that ALL aspects of this report, as well as the estimated performance (and limitations with any such estimations) of proposed project improvements are understood by the Client, Project Owner (if different), or properly conveyed to any future owner(s). Utilizing these parameters for planning, design, and/or construction constitutes understanding and acceptance of conclusions or information provided herein, potential risks, associated improvement performance, as well as the limitations inherent within such estimations. If any information referred to herein is not well understood, it is imperative for the Client,

Owner (if different), or anyone using this report to contact the author or a company principal immediately.

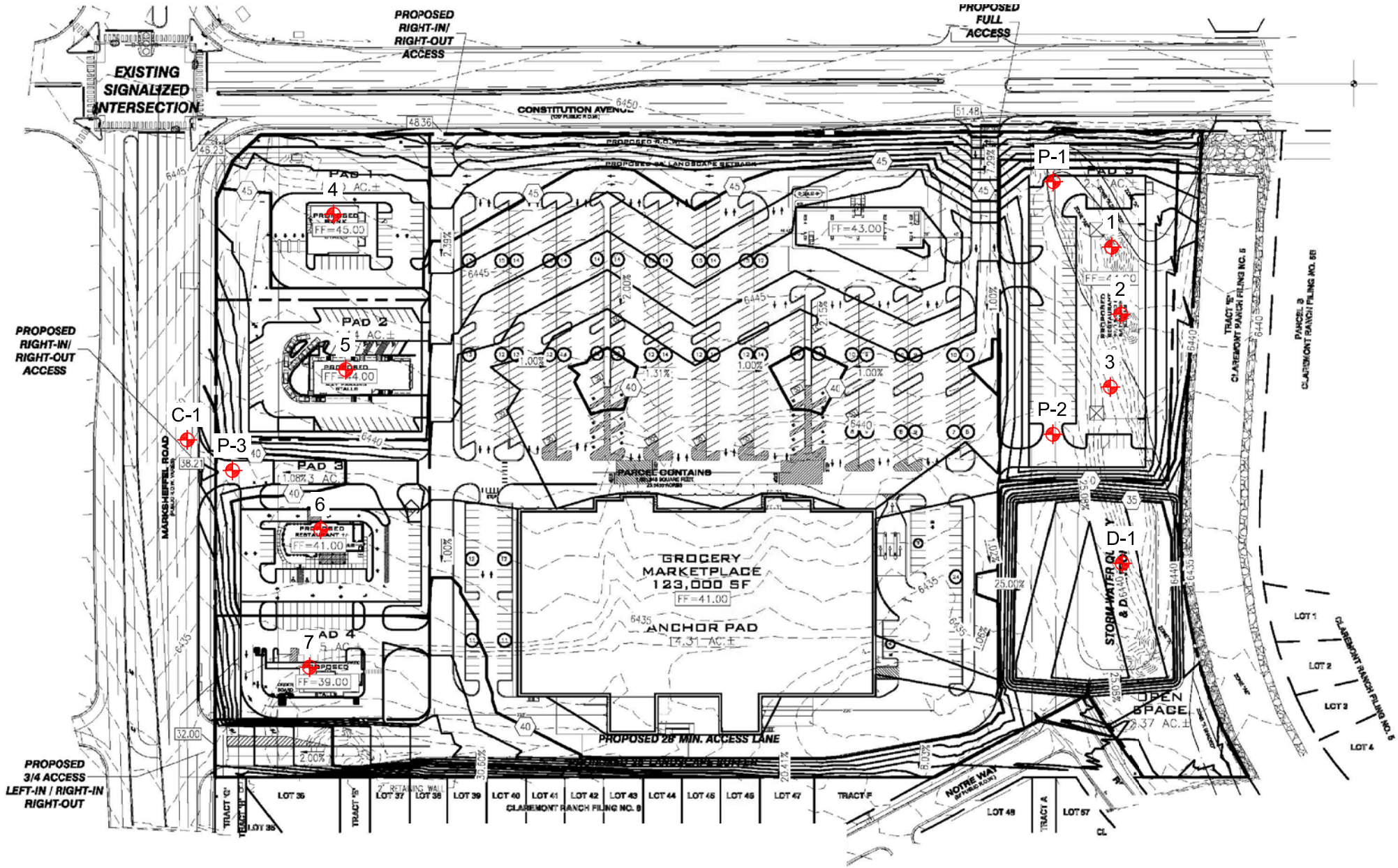
This report was prepared in accordance with generally accepted soil and foundation engineering practice in the project area at the date of preparation. Current applicable codes may contain criteria regarding performance of structures and/or site improvements which may differ from those provided herein.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

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

Sincerely,
GROUND Engineering Consultants, Inc.

Amy Crandall, P.E.

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



SITE PLAN PROVIDED BY CLIENT

1

Indicates test hole number and approximate location.

(Not to Scale)

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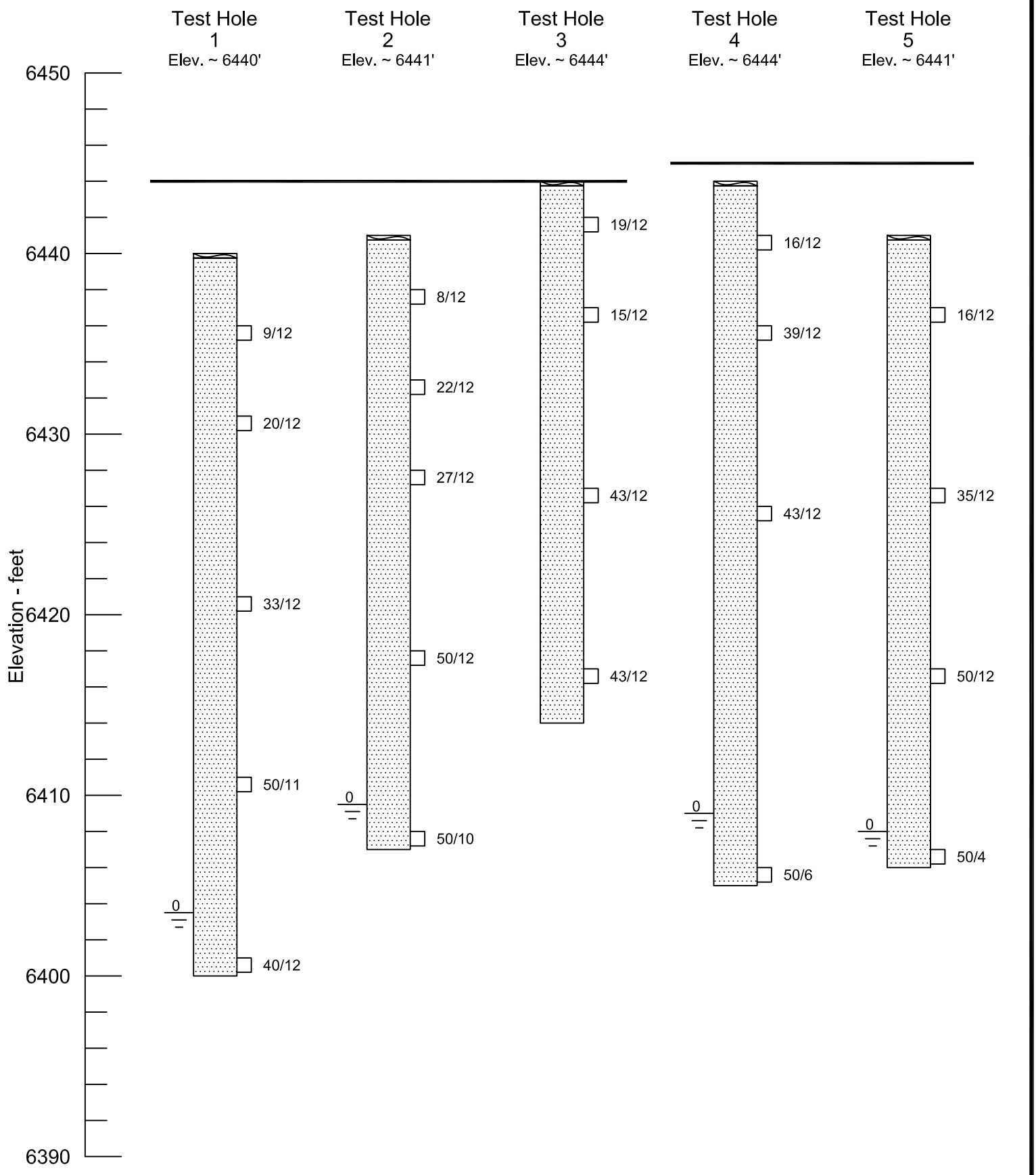
LOCATION OF TEST HOLES

JOB NO.: 15-3527	FIGURE: 1
CADFILE NAME: 3527SITE.DWG	



Indicates finished floor elevation (FFE).

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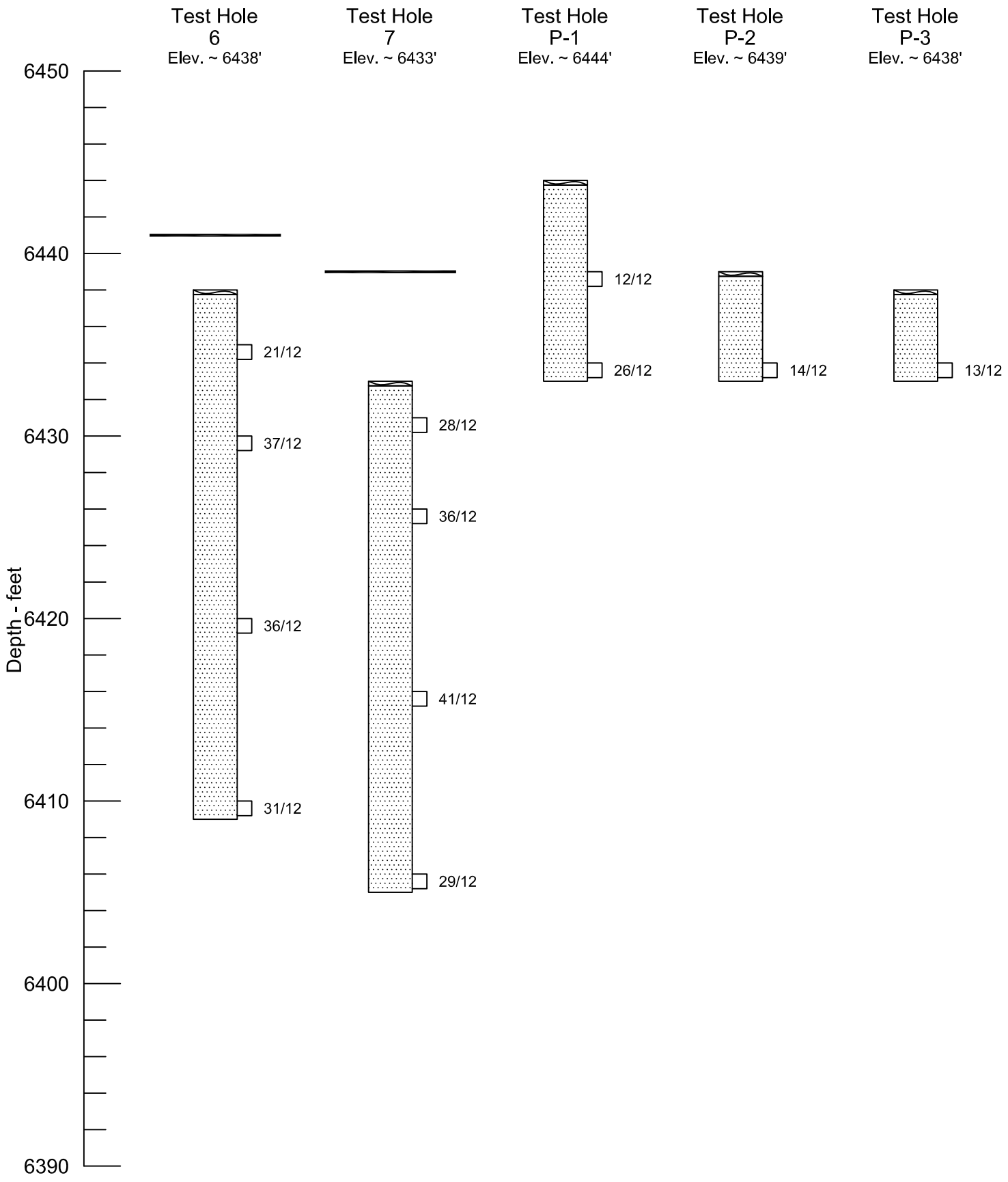
LOGS OF TEST HOLES

JOB NO.: 15-3527	FIGURE: 2
CADFILE NAME: 3527LOG01.DWG	



Indicates finished floor elevation (FFE).

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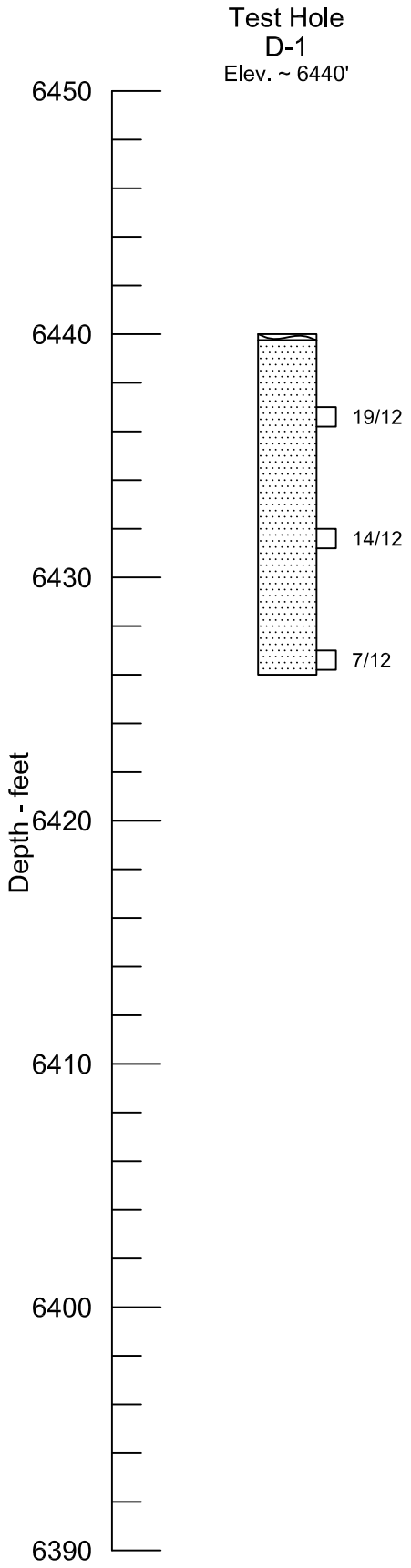
LOGS OF TEST HOLES

JOB NO.: 15-3527	FIGURE: 3
CADFILE NAME: 3527LOG02.DWG	



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LOGS OF TEST HOLES

JOB NO.: 15-3527	FIGURE: 4
CADFILE NAME: 3527LOG03.DWG	

LEGEND:

Topsoil

Sand: Silty, with interbedded layers of silt, was fine to coarse grained with occasional gravel, non-plastic to medium plastic, loose to dense, slightly moist to moist, brown to tan in color, and occasionally iron stained.

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

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

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

NOTES:

1) Test holes were drilled on 03/23 and 03/24/2015 with 4-inch diameter continuous flight augers.

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

3) Elevations of the test holes were estimated utilizing client provided topographic map 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 level readings shown on the logs were made at the time and under the conditions indicated. Fluctuations in the water level may occur with time.

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

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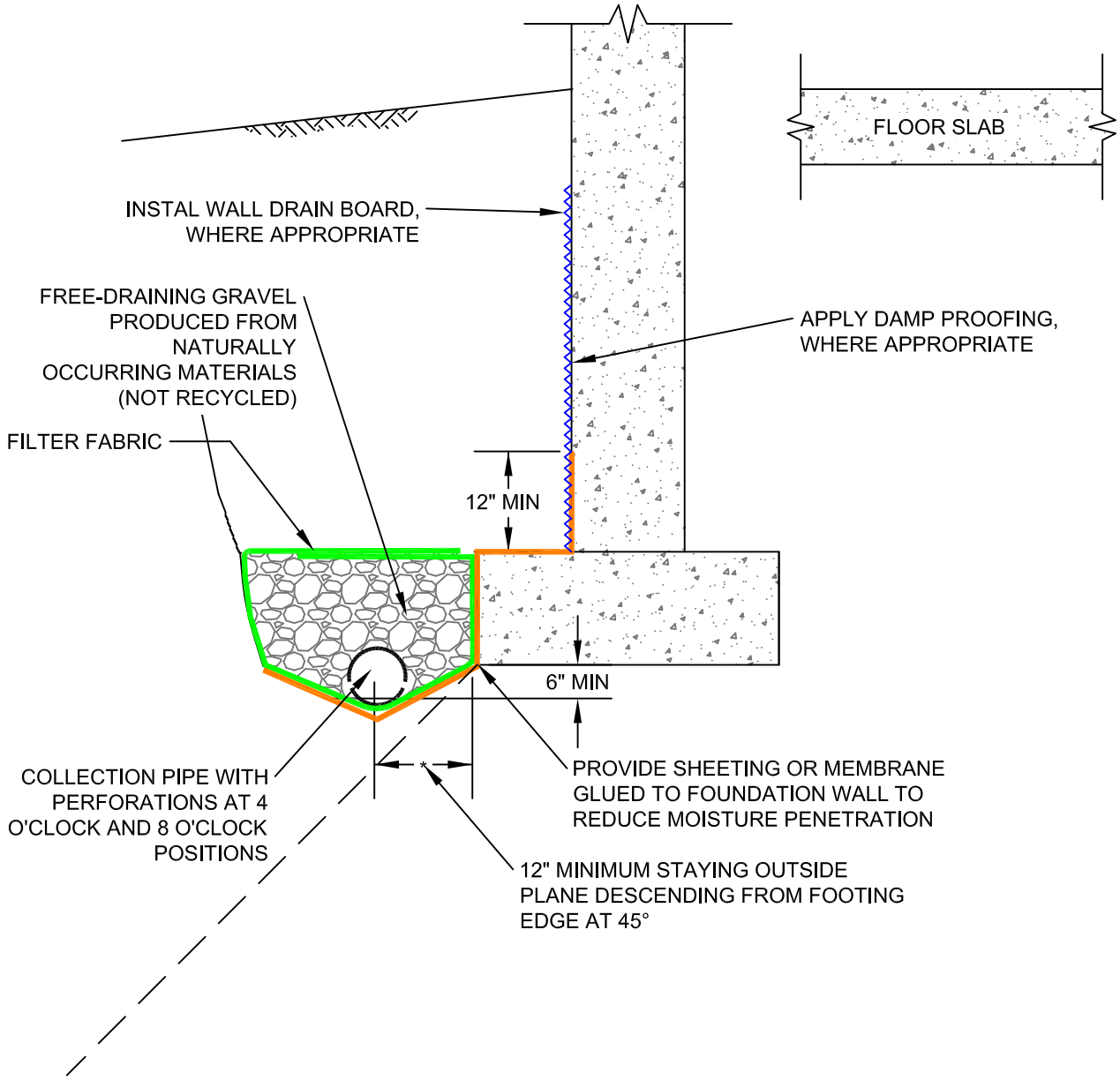
LEGEND AND NOTES

JOB NO.: 15-3527	FIGURE: 5
CADFILE NAME: 3527LEG.DWG	



SEE TEXT FOR ADDITIONAL INFORMATION

NOTES:
1. THIS IS NOT A DESIGN-LEVEL DRAWING. IT SHOULD BE USED SOLELY FOR GENERAL INFORMATION PURPOSES ONLY. ACTUAL UNDERDRAIN DESIGN SHOULD BE COMPLETED BY OTHERS.
2. THE UNDERDRAIN SYSTEM MUST BE TESTED BY THE CONTRACTOR AFTER INSTALLATION AND BACKFILLING TO VERIFY THAT IT FUNCTIONS PROPERLY.
3. INCLUSION OF THIS FIGURE IN CONSTRUCTION DOCUMENTS IS DONE SO AT THE DOCUMENT PREPARER'S RISK.
4. REPRODUCTION OF THIS DOCUMENT SHOULD BE IN COLOR.

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TYPICAL UNDERDRAIN DETAIL

JOB NO.: 15-3527	FIGURE: 6
CADFILE NAME: 3527DRAIN.DWG	

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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)*	Classifi-cation	Classifi-cation (GI)	Soil or Bedrock Type
1	4	3.3	93.9			18	NV	NP	-0.3 (500 psf)		SM	A-1-b (0)	Silty SAND
1	39	32.6	87.5			70	61	19		4,900	MH	A-7-5 (16)	SILT with Sand
2	3	5.0	116.9	7	88	5	24	6	-0.5 (400 psf)		SP-SC	A-1-b (0)	SAND with Silt and Clay
2	13	11.3	111.0			7	NV	NP			SP-SM	A-1-b (0)	SAND with Silt
3	7	2.6	96.5			3	NV	NP			SP	A-1-b (0)	SAND
3	27	5.2	96.9			6	NV	NP			SP-SM	A-1-b (0)	SAND with Silt
4	3	2.0	SD			10	NV	NP			SP-SM	A-1-b (0)	SAND with Silt
4	18	4.5	96.5	8	87	5	NV	NP			SP-SM	A-1-b (0)	SAND with Silt
5	14	5.2	112.7	10	83	7	NV	NP			SP-SM	A-1-b (0)	SAND with Silt
5	24	5.4	117.3			11	22	8			SP-SM	A-1-b (0)	SAND with Silt
6	8	3.3	108.5			7	NV	NP			SP-SM	A-1-b (0)	SAND with Silt
6	28	4.0	97.6			7	NV	NP			SP-SM	A-1-b (0)	SAND with Silt
7	2	4.5	103.5			9	22	8			SP-SM	A-1-b (0)	SAND with Silt
7	7	6.6	116.3	4	84	12	NV	NP			SP-SM	A-1- b (0)	SAND with Silt
P1	10	7.9	108.2			7	NV	NP			SP-SM	A-1- b (0)	SAND with Silt
P2	5	1.8	SD	13	85	2	NV	NP			SW	A-1-a (0)	SAND with Silt
D1	3	9.6	113.2			27	21	5			SC-SM	A-2-4 (0)	Silty, Clayey SAND
D1	13	10.0	107.5			16	22	6			SC-SM	A-1-b (0)	Silty, Clayey SAND
P1-P2	0-5	122.3*	9.6*			22	21	3			SM	A-2-4(0)	Silty SAND

SD = Sample Disturbed; NV = Non-Viscous; NP = Non-Plastic

** Negative swell indicates consolidation*

Job No. 15-3527

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TABLE 2
SUMMARY OF SOIL CORROSION TEST RESULTS

Sample Location		Water Soluble Sulfates	pH	Redox Potential	Sulfides Content	Resistivity	USCS Classifi-cation	Soil or Bedrock Type
Test Hole No.	Depth *(feet)*	*(%)*		*(mV)*		*(ohm-cm)*		
1	4	<0.01	8.2	-69	Trace	1,430	SM	Silty SAND
7	7	<0.01	8.6	-94	Trace	6,690	SP-SM	SAND with Silt

Job No. 15-3527