GROUND ENGINEERING

Subsurface Exploration Program
Geotechnical Evaluation
Grand Fire Protection District No. 1
North Fire Station
Grand County, Colorado

Prepared For:

Grand Fire Protection District No. 1
P.O. Box 338
60500 US Highway 40
Granby, Colorado 80446

Attention: Mr. Ron Thompson

Job Number: 15-2013 **November 4, 2015**

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PURPOSE AND SCOPE OF STUDY

This report presents the results of a subsurface exploration program performed by GROUND Engineering Consultants, Inc. (GROUND) to provide geotechnical and pavement section parameters for the proposed North Fire Station project to be constructed approximately ½ mile northwest of the intersection of US Highway 34 and Grand County Road 40 in Grand County, Colorado. The site is generally located north of the town of Granby. Our study was conducted in general accordance with GROUND's Proposal No. 1509-1833, dated September 17, 2015.

Field and office studies provided information obtained at the test hole locations regarding surface and subsurface conditions, including the existing site vicinity improvements. Material samples retrieved during the subsurface exploration were tested in our laboratory to assess the engineering characteristics of the site earth materials, and assist in the development of our geotechnical parameters and opinions. Results of the field, office, and laboratory studies are presented below.

This report has been prepared to summarize the data obtained and to present our conclusions 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 two-story fire station facility with an approximate footprint size of 6,800 square feet. The facility will include 4 firefighting equipment bays and apartment units. Associated parking lot, drive lanes, and entrances/exits constructed around the building will also be constructed around the building. We also understand that below grade levels or retaining walls are not planned at this time.

Grading information, anticipated building loads, and vehicular loading were not available at the time of this report preparation. However, based on the existing topography, it

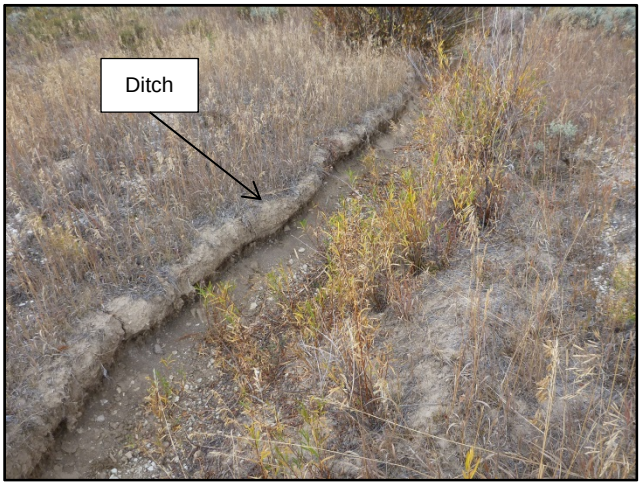
appears that cuts and fills ranging up to approximately 5 feet may be necessary to facilitate the site grading.

SITE CONDITIONS

At the time of our site visits, the project site was an undeveloped lot (see photo to the right) bound by Grand County Road 40 to the north, and undeveloped land to the east, south, and west. The surrounding parcels to the north and east are mapped as being part of the Arapahoe National Recreation Area.

The ground the surface supported native grasses and shrubs in addition to juvenile to mature trees.

Based on Google Earth imagery and field observations, the ground surface sloped gently to the south, west, and east and displayed approximately 15 feet of relief across the project site. Approximately 5 feet of relief was observed across the proposed fire station area.



A ditch was observed traversing the property from northeast to southwest, approximately bisecting it. The ditch continued beyond the northern and western borders of the property. The ditch was approximately 2 to 3 feet deep and was dry at the time of our subsurface exploration program.

SUBSURFACE EXPLORATION

The subsurface exploration for the project was conducted in September, 2015. A total of six (6) test holes were drilled with a truck-mounted drill rig advancing continuous flight auger equipment to evaluate the subsurface conditions as well as to retrieve soil samples for laboratory testing and analysis. Of these, three (3) test holes were advanced within/near the approximate proposed building footprint limits to depths ranging from approximately 25 to 40 feet below existing grade. The remaining three (3) test holes were advanced for the new parking lot and drive to depths ranging up to approximately 5 to 10 feet.

A GROUND engineer directed the subsurface exploration, logged the test holes in the field, and prepared the soil samples for transport to our laboratory.

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

The approximate locations of the test holes are shown in Figure 1. Logs of the exploratory test holes are presented in Figures 2 and 3. Explanatory notes and a legend are provided in Figure 4.

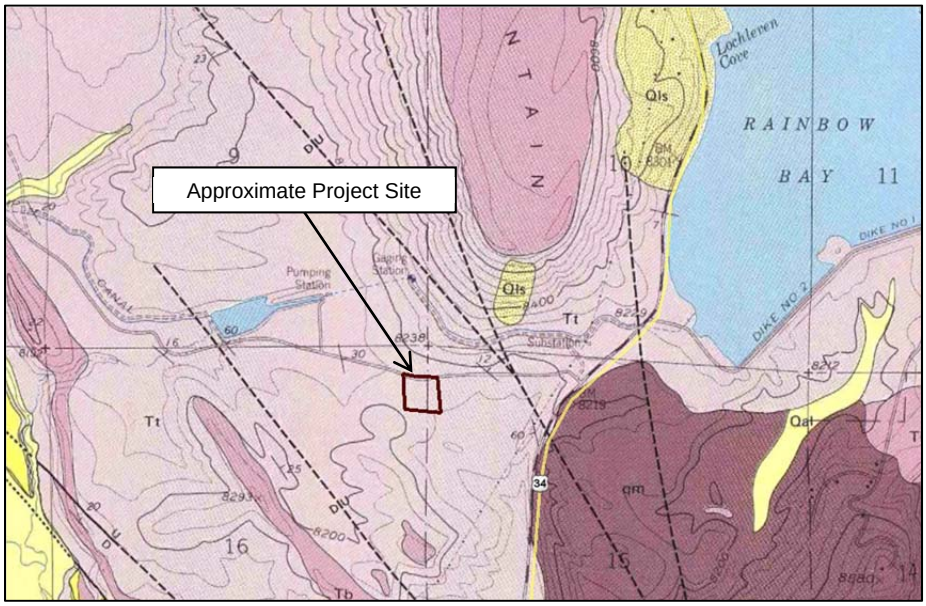
LABORATORY TESTING

Samples retrieved from our test holes were examined and visually classified in the laboratory by the project engineer. Laboratory testing of soil samples obtained from the subject site included standard property tests, such as natural moisture contents, dry unit weights, grain size analyses, Atterberg limits, and swell/consolidation testing. Water-soluble sulfate and corrosivity tests were completed on a selected sample of the soils, as well. Laboratory tests were performed in general accordance with applicable ASTM

protocols. Results of the laboratory testing program are summarized on Tables 1 and 2. Gradation Plots are provided in Figures 5 – 7.

SUBSURFACE CONDITIONS

Regional Geology The bedrock underlying the project was mapped as the Miocene aged Troublesome Formation (**Tt**) (e.g., Izett, 1974[1]). A portion of this geologic map is reproduced below. North of Granby, the Troublesome Formation consists of gray, tuffaceous (ashy) mudstone (termed 'siltstone' herein) and sandstone bedrock with local basalt flows. Conglomerates also are present locally. (see geologic map below).



Subsurface Conditions The subsurface conditions encountered in the test holes generally consisted of a thin layer of topsoil[2] underlain by sands and clays extending approximately to about 1½ to 2 feet below existing grades. These materials were underlain by interbedded sandstone, siltstone, and claystone bedrock that extended to the test hole termination depths. Upper few feet of bedrock was typically severely weathered.

[1] Izett, G.A., 1974, *Geologic map of the Trail Mountain quadrangle, Grand County, Colorado*: U.S. Geological Survey, Geologic Quadrangle Map GQ-1156.

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

Fill soils were not recognized in our test holes but may be present on-site. Delineation of the complete lateral and vertical extents of any fills at the site, or their compositions, was beyond our present scope of services. The actual extents and composition of any fill materials are not known. Therefore, some excavated fill materials may not be suitable for re-use as compacted fill. If fill soil volumes and compositions at the site are of significance, they should be evaluated by the contractor using test pits. GROUND should be contacted if such materials are exposed / encountered during construction.

We interpret the bedrock materials at the site to be Troublesome Formation deposits. We interpret the overlying sand and clay to be residual (developed in-place) soils.

It should be noted that coarse gravel, cobbles and boulders are not well represented in small diameter liner samples collected from 4-inch diameter test holes. Therefore, such materials may be present even where not called out in the material descriptions herein.

Sand and Clay ranged from fine to medium clayey to silty sands to sandy clays with gravels locally. They were low to highly plastic, medium dense to dense or medium to very stiff, dry to slightly moist, and light brown to brown in color. Cobbles and boulders were also noted on the surface.

Weathered Sandstone, Siltstone, and Claystone was interbedded and was fine to coarse grained, low to highly plastic, medium hard to very hard, slightly moist, and light brown to red brown and gray in color.

Sandstone, Siltstone, and Claystone Bedrock was interbedded and was fine to coarse grained, low to highly plastic, medium hard to very hard, slightly moist to wet, and light brown to red brown, gray, and green in color.

Groundwater was not encountered in test holes at the time of drilling. The test holes were backfilled immediately after drilling operations, for safety reasons, and subsequent depth to groundwater measurements were not taken. However, based on our experience with similar projects, groundwater conditions and water seepage in material cuts have been encountered within the project vicinity during construction of retaining structures, walkout-basements, cut slopes, etc. In areas where surface and subsurface drainage has not been properly designed or maintained, groundwater and/or wet conditions were previously encountered. Similarly, where existing natural drainages are

buried during grading operations for the project, a drainage system should be constructed at the bottom of the existing drainage prior to filling so that seasonal water fluctuations can properly drain. Therefore, shallow groundwater conditions, seepage through temporary and permanent cuts, and related soft and wet subgrade conditions should be anticipated by the Project Team and Contractors. Proper drainage measures should be taken during and after construction.

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

Swell and Consolidation Testing of on-site materials encountered in the test holes indicated a potential for heave and consolidation (See Table 1). Consolidations ranging up to 0.8 percent and swells ranging up to 11.4 percent were measured against surcharges approximately equal to anticipated overburden loads or foundation loads.

GEOTECHNICAL CONSIDERATIONS FOR DESIGN

Geotechnical Risk The native site soils and bedrock materials exhibited consolidations up to 0.8 percent and swells up to 11.4 percent under approximate overburden or foundation loads. These conditions, if not properly mitigated, can cause damaging, post-construction, structural movements that will affect nearly all improvements at the site.

Mitigating the expansive materials and/or their effects and controlling the surface waters and shallow subsurface moisture changes are the principal geotechnical design considerations for the site. Specific geotechnical parameters in these regards are provided in subsequent sections of this report. Additional discussion and information regarding these measures and the geotechnical risks that they address are provided below.

The conclusions and parameters provided in this report were based on the data presented herein, our experience in the general project area with similar structures, and our engineering judgment with regard to the applicability of the data and methods of forecasting future performance. A variety of engineering parameters were considered as indicators of potential future soil movements. The parameters provided herein were based on our judgment of "likely movement potentials," (i.e., the amount of movement likely to be realized if site drainage is generally effective, estimated to a reasonable degree of engineering certainty) as well as our assumptions about the owner's willingness to accept geotechnical risk. "Maximum possible" movement estimates necessarily will be larger than those presented herein. They also have a significantly lower likelihood of being realized in our opinion, and generally require more expensive measures to address. We encourage Grand Fire Protection District No. 1, upon receipt of this report, however, to discuss these risks and the geotechnical alternatives with us.

Grand Fire Protection District No. 1 must, therefore, understand the risks and remedial approaches presented in this report (and the risk-cost trade-offs addressed by the civil and structural engineering disciplines) in order to direct the design team to the portion of the Higher Cost / Lower Risk – Lower Cost / Higher Risk spectrum in which this project should be designed. If Grand Fire Protection District No. 1 does not understand these risks, it is critical that additional information or clarification be requested so that expectations reasonably can be met.

Depth of Wetting For development of geotechnical parameters for drilled piers, a 'depth of wetting' of 22 feet was used, based on our experience with similar projects. A

depth of wetting of 22 feet is equal to or greater than the depth of wetting found at about 82 percent of the sites evaluated in a study by Walsh and others (2009).[3]

GROUND considers a 22-foot depth of wetting to be appropriately conservative for the proposed project. However, there remains a possibility that it could be exceeded. If Grand Fire Protection District No. 1 prefers that a more conservative (or less conservative) 'depth of wetting' to be used to develop geotechnical parameters, GROUND should be contacted to revise the values provided herein.

Likely Post-Construction Movements Utilizing the above assumptions, data obtained for this study, and our experience on other projects in the vicinity, we estimate that where structure elements, pavements, etc., are supported directly on the existing earth materials, they will be subject to likely post-construction, vertical movements of 4 to 6 inches. Lateral movements will result, as well. As noted above, significant structure distress could result from foundation movements of these magnitudes.

General Foundation and Floor Types Supporting the proposed buildings on deep foundation systems such as drilled piers, and providing them with structural floors also supported on deep foundation systems, will yield the least risk of post-construction movements, likely on the order of ½ inch or less. Details regarding deep foundation systems and structural floors are provided in the *Drilled Pier Foundation Systems* and *Structural Floors and Critical Flatwork* sections of this report.

As a higher risk alternative to structural floors (which may not be acceptable for this facility) the proposed fire station may be constructed with slab-on-grade floors bearing on a remedial fill section. That fill section should consist of 3 feet of properly compacted select granular backfill, such as CDOT Class 1, over 8 feet of properly compacted site derived fill. If the slab-on-grade floor is constructed on such a fill section, we estimate that likely, post-construction, vertical, floor movements will be approximately 1 inch. Likely differential movements likely will be similar over spans of about 40 feet. Additional details in regards to slab-on-grade floors are provided in the *Slab-on-Grade Floor Systems* sections of this report.

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[3] Walsh, K.D., C.A. Colby, W.N. Houston and S.A. Houston, 2009, *Method for Evaluation of Depth of Wetting in Residential Areas*, Journal of Geotechnical and Geoenvironmental Engineering, American Society of Civil Engineers, Vol. 135, No. 2, pp. 169 – 176.

DRILLED PIER FOUNDATION SYSTEMS

Geotechnical Parameters for Drilled Pier Design The geotechnical criteria below should be observed for straight-shaft, drilled pier foundation systems.

Note that the depths indicated herein refer to depths below existing grades at the time of GROUND's field exploration unless otherwise noted. The contractor should account for grade changes between that time and the time of installation. Lowering grades may not be sufficient to shorten piers, however.

1) Drilled piers should bear in 'relatively un-weathered' bedrock underlying the site.

   For design purposes, relatively un-weathered bedrock may be taken to be at and below depths of **6 feet** below existing grades. <u>For bidding purposes, some variation in the depth to relatively un-weathered bedrock should be anticipated</u>.

2) Drilled piers should be at least **18 inches** in diameter and should be designed with a length to diameter ratio no greater than **30 to 1**. Actual length to diameter ratios should be determined by the structural engineer.

3) Drilled piers should have a minimum length of **35 feet**.

   Drilled piers also should penetrate at least **13 feet** into relatively un-weathered bedrock or 3 drilled pier diameters, whichever is.

   Based on the minimum length and bedrock penetration, and taking the top of relatively un-weathered bedrock to be about 6 feet below grade, drilled pier lengths of 35 to 38 feet are anticipated to meet the geotechnical criteria. Actual drilled pier lengths commonly will be greater due to structural considerations, conditions in the drilled pier holes, actual depths to relatively un-weathered bedrock, etc., as well as any grade changes associated with construction of the project.

4) Drilled piers bearing in relatively un-weathered bedrock at and below the depths required above may be designed for an allowable end bearing pressure of **30,000 psf**.

The portion of the drilled pier penetrating relatively un-weathered bedrock at depths greater than 22 feet may be designed for a skin friction value of **2,650 psf**.

100 percent of the skin friction may be used to resist both compressional loads and uplift.

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

6) Drilled piers should be designed for a minimum dead load pressure of **8,000 psf** based on drilled pier cross-section area to a) avoid lengthening of the pier to achieve adequate uplift resistance and b) reduce tensile stresses in the pier.

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

7) Drilled piers should be reinforced as determined by the structural engineer. At a minimum, each drilled pier should be reinforced for its full length to resist the tensile loading created by the deficit between the uplift loads exerted by the swelling soils and bedrock and the actual dead load applied to a pier. Uplift may be estimated as an uplift skin friction of **1,600 psf** applied to the upper **22 feet** of the drilled pier.

8) A **10-inch** or thicker continuous void should be provided beneath grade beams, drilled pier caps, and foundation walls to limit the potential of swelling soil and bedrock from exerting uplift forces on these elements and to concentrate drilled pier loadings. The void space should be protected from backfill intrusion.

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

10) Penetration of relatively un-weathered bedrock in drilled pier shafts should be roughened artificially to assist the development of peripheral shear between the drilled pier and bedrock. Artificially roughening of drilled pier holes should

consist of installing shear rings **3 inches high** and **2 inches deep** in the portion of each drilled pier penetrating relatively un-weathered bedrock below **a depth of 22 feet** (from existing grades). The shear rings should be installed **18 inches on center**.

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

11) Groups of closely spaced drilled piers will require an appropriate reduction of the estimated capacities. Reduction of axial capacity generally can be avoided by spacing drilled piers at least 3 diameters center to center. At this spacing or greater, no reduction in axial capacities or horizontal soil modulus values is required. The capacities of drilled piers spaced more closely than 3 diameters center to center should be reduced. Reduction factors can be obtained from the plot provided on Figure 8.

    Linear arrays of drilled piers, however, must be spaced at least 8 diameters center to center to avoid reductions in lateral capacity when loaded in line with the array (parallel to the line connecting the drilled pier centers). The lateral capacities of piers in linear arrays spaced more closely than 8 diameters should be reduced. Reduction factors can be obtained from the plot provided on Figure 9.

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

12) The depth to relatively un-weathered bedrock should be determined in the field at each drilled pier location and may differ from other information provided herein.

13) Lenses or beds of relatively soft bedrock and/or lignite or coal not suitable for foundation support may be encountered within the relatively un-weathered bedrock section, which may result in lengthening the drilled piers.

14) The bedrock in the project area is known to include very hard and resistant beds and lenses locally. The pier-drilling contractor should mobilize equipment of

sufficient size and operating capability to achieve the design lengths and bedrock penetration. The contractor should have the equipment on-site to core hard, bedrock materials, where necessary, in order to advance the pier holes to sufficient depths.

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

15) Groundwater was not encountered during subsurface exploration to the depths explored. Groundwater may be encountered during drilled pier installation, however, and casing may be required in the drilled pier holes to reduce water infiltration. In the event that casing is seated into the bedrock, the minimum bedrock penetration should be taken from the bottom of the casing.

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

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

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

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

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

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

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

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

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

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

STRUCTURAL FLOORS AND CRITICAL FLATWORK

The fire station floor and all critical flatwork should be constructed as structural floors supported on grade beams and drilled piers in the same manner as the building structure and should span over well-ventilated crawl spaces. Requirements for the number and position of additional piers to support the floors will depend upon the span, design load, and structural design, and should be developed by the structural engineer. Geotechnical parameters for design and installation of drilled piers are provided in the *Drilled Pier Foundation System* section of this report. (Exterior flatwork that is sufficiently sensitive to post-construction movements to be constructed as a structural floor should be identified by the Fire Protection District.)

The crawl space beneath a structural floor should be at least 3 feet in depth, but also adequately sized to allow utility lines to be installed above the swelling materials and to allow access to and maintenance of utility piping.

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

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

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

The Fire Protection District must be willing to accept the risks of potential mold, fungus, and mildew growth when electing to utilize a structural floor system. Additionally, the Contractor is solely responsible for the construction means and methods, and any

observation or testing performed by a representative of the geotechnical engineer during construction does not relieve the Contractor of that responsibility.

Mold Growth Areas/Conditions for Growth for Structural Floors

1) Water damaged building materials or high moisture/humidity areas where cellulose-containing materials are used:
   i. Wallboard/Sheetrock
   ii. MDF/OSB/Plywood
   iii. Fibrous Ceiling Tiles
   iv. Paper-backed Insulation
   v. Jute-backed Carpet
   vi. Hardwood Flooring
2) Condensation inside buildings from pipes, baths, heaters, and dryer vents
3) Relative humidity greater than 55%
4) Temperatures of 36 to 104 ºF.
5) “Wet” areas that do not dry out after 24 hours.

Mold does not require a light source in order to grow and can grow inside walls, behind tubs/showers, under carpet and flooring undetected.

Piping connections through the floor should allow for differential movement between the piping and floor and positive bond breaks provided because utility lines can be displaced by soils and bedrock movements, as discussed in the *Geotechnical Considerations for Design* section of this report, which are not reflected in the structural floor.

SLAB-ON-GRADE FLOORS

A slab-on-grade floor represents a higher risk alternative to a structural floor. If selected, then where a slab-on-grade floor is designed and constructed in accordance with the following criteria, the potential for slab movement will not be eliminated. However, the magnitude of the movements will be reduced and generally more uniform. This will reduce but not eliminate the resultant damage where movements occur.

Slab-on-grade construction should be used only if the Fire Protection District understands and accepts the risk of post-construction slab movements. Post-construction slab movements are directly related to changes in moisture contents of the underlying soils after construction is completed.

1) A slab-on-grade floor should bear on a section of properly compacted fill as discussed in the *Geotechnical Considerations for Design* section of this report. The above fill section should extend at full depth at least **10 feet** laterally beyond the slab perimeter.

   The thickness of the re-worked section should be taken from the bottom of the slab + gravel layer system. (If the gravel layer is not installed, the re-worked section should be correspondingly thickened.)

   Parameters for fill placement and compaction are provided in the *Project Earthwork* section of this report. Exposed loose, soft, or otherwise unsuitable materials should be excavated and replaced with properly compacted fill.

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

2) Floor slabs should be adequately reinforced. Floor slab design, including slab thickness, concrete strength, jointing, and slab reinforcement should be developed by a structural engineer.

3) A vertical modulus of subgrade reaction **(Kv) of 100 tcf (116 pci)** may be used for design of a concrete, slab-on-grade floor bearing on properly compacted granular fill section.

   This value is for a 1-foot x 1-foot plate; it should be adjusted for slab dimension.

4) The slab should be separated from all bearing walls and columns with slip joints, which allow unrestrained vertical movement. \\Slip joints should be observed periodically, particularly during the first several years after construction. Slab movement can cause previously free-slipping joints to bind. Measures should be taken to assure that slab isolation is maintained in order to reduce the likelihood

of damage to walls and other interior improvements, including door frames, plumbing fixtures, etc.

5) Concrete slabs-on-grade should be provided with properly designed control joints.

ACI, AASHTO and other industry groups provide guidelines for proper design and construction concrete slabs-on-grade and associated jointing. The design and construction of such joints should account for cracking as a result of shrinkage, curling, tension, loading, and curing, as well as proposed slab use. Joint layout based on the slab design may require more frequent, additional, or deeper joints, and should reflect the configuration and proposed use of the slab.

Particular attention in slab joint layout should be paid to areas where slabs consist of interior corners or curves (e.g., at column blockouts or reentrant corners) or where slabs have high length to width ratios, significant slopes, thickness transitions, high traffic loads, or other unique features. The improper placement or construction of control joints will increase the potential for slab cracking.

6) Interior partitions resting on floor slabs should be provided with slip joints so that if the slabs move, the movement cannot be transmitted to the upper structure. This detail is also important for wallboards and doorframes. Slip joints should allow **2 inches or more** of vertical, differential movement. Accommodation for differential movement also should be made where partitions meet bearing walls.

7) 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 **3 or more inches** of vertical movement should be provided for slab-bearing mechanical equipment.

8) Moisture can be introduced into a slab subgrade during construction and additional moisture will be released from the slab concrete as it cures. A properly compacted layer of free-draining gravel, 6 or more inches in thickness, should be placed beneath the slabs. This layer will help distribute floor slab loadings, ease construction, reduce capillary moisture rise, and aid in drainage. \\The free-draining gravel should contain less than 5 percent material passing the No. 200

Sieve, more than 50 percent retained on the No. 4 Sieve, and a maximum particle size of 2 inches. \\The capillary break and the drainage space provided by the gravel layer also may reduce the potential for excessive water vapor fluxes from the slab after construction as mix water is released from the concrete. \\We understand, however, that professional experience and opinion differ with regard to inclusion of a free-draining gravel layer beneath slab-on-grade floors. If these issues are understood by the owner and appropriate measures are implemented to address potential concerns including slab curling and moisture fluxes, then the gravel layer may be deleted.

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

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

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

In the event a vapor barrier is utilized, it should consist of a minimum 15 mil thickness, extruded polyolefin plastic (no recycled content or woven materials), maintain a permeance less than 0.01 perms per ASTM E-96 or ASTM F-1249, and comply with ASTM E-1745 (Class "A"). Vapor barriers should be installed in accordance with ASTM E-1643.

Polyethylene ("poly") sheeting (even if 15 mils in thickness which polyethylene sheeting commonly is not) does not meet the ASTM E-1745 criteria and is not

recommended for use as vapor barrier material. It can be easily torn and/or punctured, does not possess necessary tensile strength, gets brittle, tends to decompose over time, and has a relatively high permeance.

Construction Considerations for Slab-on-Grade Floors

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

11) The fill section beneath a slab should be of uniform thickness.

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

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

14) A geotechnical engineer should be retained to observe the prepared surface on which the floor slab will be cast prior to placement of reinforcement. Loose, soft, or otherwise unsuitable materials should be excavated and replaced with properly compacted fill, placed in accordance with the parameters in the *Project Earthwork* section of this report.

NONCRITICAL EXTERIOR FLATWORK

Proper design, drainage, construction, and maintenance of the areas between individual buildings and parking/driveway areas are critical to the satisfactory performance of the project. Sidewalks, entranceway slabs and roofs, fountains, raised planters, and other highly visible improvements commonly are installed within these zones, and distress in or near these improvements are common. Commonly, soil preparation in these areas receives little attention because they fall between the building and pavement (which are typically built with heavy equipment). Subsequent landscaping and hardscape installation often is performed by multiple sub-contractors with light or hand equipment, and over-excavation / soil processing is not performed. The design team, contractor, and pertinent subcontractors should take particular care with regard to proper subgrade preparation around the structure exteriors.

Similar to slab-on-grade floors, exterior flatwork and other hardscaping placed on the soils encountered on-site may 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 as the soils experience volume change as the moisture content varies. Distress to rigid hardscaping likely will result. The following measures will help to reduce damages to these improvements. However, if performance like a slab-on-grade floor or structural floor is desired, than the exterior flatwork should be supported on deep foundations and span over a void (for performance like structural floor) or constructed on a fill section similar to the one provided for slab-on-grade floors (for performance like a slab-on-grade floor).

Provided the owner understands the risks identified above, we believe that subgrade under exterior flatwork or other (non-building) site improvements could be scarified and/or excavated to a minimum depth of **2 feet** and the material re-compacted as properly compacted fill. Greater depths of re-working (i.e. 3 to 4 feet) will result in improved long term performance. For performance similar to a slab-on-grade floor, however, a similar depth of remedial earthwork would be necessary, however. The re-worked soils 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. ACI parameters should be followed regarding construction and/or control joints. Based on our experience, concrete sidewalks are often 4 inches in thickness. Actual thicknesses should be based on project design or governing municipal standards.

In no case should exterior flatwork extend to under any portion of the building where there is less than 3 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.

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 3 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 3½ 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 information 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 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.

LATERAL LOADS

Deep Foundations Resisting Lateral Loads Based on the data obtained for this study and our experience with similar sites and conditions, lateral load analysis using the Terzaghi method may take the values tabulated below for the modulus of horizontal subgrade reaction (K_h) to be characteristic of the soils and bedrock underlying the site, based on a simplified soil / bedrock profile relative to existing grades at the time of subsurface exploration. The upper 4 feet of embedment should be neglected for resistance to lateral loading.

HORIZONTAL MODULUS SUBGRADE REACTION (K_h) – TERZAGHI METHOD

Material	**Approximate Depth Range** (below grade)	**K_h based on Foundation Element Width / Diameter**	
		18-inch	**24-inch**
Sands and Clays and Weathered Sandstone, Siltstone, and Claystone	4 to 6 feet	40 tcf (46 pci)	30 tcf (35 pci)
Sandstone, Siltstone, and Claystone Bedrock	> 6 feet	200 tcf (232 pci)	150 tcf (174 pci)

Note that the K_h values tabulated above are dependent on deep foundation element width or diameter. If values for other diameters are required, please contact this office.

If "L-Pile" or a similar computer program were used for lateral analysis of the piles, geotechnical parameters for input into that program are tabulated below for the same simplified soil / bedrock profile. These include, unit wet weights (γ'), internal friction angle (ϕ), cohesion (**c**), for the earth materials, as well as values for strain at 50 percent of failure stress (ε_{50}) and horizontal soil modulus (**k**). Again, lateral resistance should be neglected for soils shallower than 4 feet below existing grades.

GEOTECHNICAL PARAMETERS FOR LATERAL LOAD ANALYSIS USING L-PILE

Soil / Bedrock Material	Approximate Depth Range	Parameter	Value
Sands and Clays and Weathered Sandstone, Siltstone, and Claystone (modeled as Sand above the water table)	4 - 6 feet	γ'	120 pcf (0.0694 pci)
		ϕ	28 degrees
		k	0.077×10^6 pcf (45 pci)
Sandstone, Siltstone, and Claystone Bedrock (modeled as Stiff Clay without Free Water)	> 6 feet	γ'	120 pcf (0.0694 pci)
		c	3,500 psf (27.8 psi)
		ε_{50}	0.004

Active and At-Rest Lateral Earth Pressures Project foundation elements, such as thrust blocks for utilities, may be designed to resist lateral earth pressures computed on the basis of the following equivalent fluid unit weights:

Material	At-Rest EFP	Active EFP	Passive EFP	Friction Coefficient
On-Site Materials	68 pcf	47 pcf	310 pcf	0.33
CDOT Class 1 Structure Backfill	59 pcf	38 pcf	470 pcf	0.45

The earth pressures provided above assume a horizontal backfill surface. Shoring wall and foundation wall design should incorporate any upward sloping backfills, live loads such as construction equipment, material stockpiles, etc., and other surcharge pressures. The values were based on a moist unit weight (γ') of 120 pcf and an angle of internal friction (ϕ) of 26 degrees for the on-site materials, and a moist unit weight (γ') of 132 pcf and an angle of internal friction (ϕ) of 34 degrees for the structure backfill. These values are un-factored. Appropriate factors of safety should be included in design calculations.

WATER-SOLUBLE SULFATES

The concentration of water-soluble sulfates measured in selected samples obtained from the test holes ranged up to 0.01 percent (Table 2). Such concentrations of water-soluble sulfates represent a **'negligible'** environment for sulfate attack on concrete exposed to these materials. Degrees of attack are based on the scale of 'negligible,' 'moderate,' 'severe' and 'very severe' as described in the "Design and Control of Concrete Mixtures," published by the Portland Cement Association (PCA). The Colorado Department of Transportation (CDOT) utilizes a corresponding scale with 4 classes of severity of sulfate exposure (Class 0 to Class 3) as described in the published table below.

Based on these data, no special, sulfate-resistant cement need be used in project concrete.

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

SOIL CORROSIVITY

Data were obtained to support an initial assessment of the potential for corrosion of ferrous metals in contact with earth materials at the site, based on the conditions at the time of GROUND's evaluation. The test results are summarized in Table 2.

Reduction-Oxidation testing indicated red-ox potentials of -76 and -93 millivolts. Such a low potentials typically create a more corrosive environment.

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

Soil Resistivity In order to assess the "worst case" for mitigation planning, samples of materials retrieved from the test holes were tested for resistivity in the laboratory, after being saturated with water, rather than in the field. Resistivity also varies inversely with temperature. Therefore, the laboratory measurements were made at a controlled temperature. Measurement of electrical resistivity indicated values of approximately 16,046 and 16,644 ohm-centimeters in samples of site soils.

pH Where pH is less than 4.0, soil serves as an electrolyte; the pH range of about 6.5 to 7.5 indicates soil conditions that are optimum for sulfate reduction. In the pH range above 8.5, soils are generally high in dissolved salts, yielding a low soil resistivity.[4] Testing indicated pH values of about 8.1 and 8.4.

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

Table A.1 Soil-test Evaluation

Soil Characteristic / Value	Points
Redox Potential	
< 0 (negative values)	5
0 to +50 mV	4
+50 to +100 mV	3½
> +100 mV	0
Sulfide Reactivity	
Positive	3½
Trace	2
Negative	0
Soil Resistivity	
<1,500 ohm-cm	10
1,500 to 1,800 ohm-cm	8
1,800 to 2,100 ohm-cm	5
2,100 to 2,500 ohm-cm	2
2,500 to 3,000 ohm-cm	1
>3,000 ohm-cm	0

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

pH

0 to 2.0	5
2.0 to 4.0	3
4.0 to 6.5	0
6.5 to 7.5	0 *
7.5 to 8.5	0
>8.5	3

Moisture

Poor drainage, continuously wet	2
Fair drainage, generally moist	1
Good drainage, generally dry	0

* If sulfides are present <u>and</u> low or negative redox-potential results (< 50 mV) are obtained, add three (3) points for this range.

We anticipate that drainage at the site after construction will be effective. Nevertheless, based on the values obtained for the soil parameters, the overburden soils and bedrock appear to comprise a moderately corrosive environment for ferrous metals (7 points).

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

RADON POTENTIAL

Radon is a naturally occurring, colorless, odorless, radioactive gas that can cause lung cancer, according to the U.S. Environmental Protection Agency (EPA). The occurrence of radon is difficult to predict, and structures with all types of foundations can be affected by radon build up. Radon allowed to concentrate in an enclosed structure represents a potential hazard. It is not a hazard that can be mitigated by geotechnical measures, however.

Testing for the possible presence of radon gas prior to project development does not yield useful results regarding the potential accumulation of radon in completed structures. Radon accumulations most typically are found in basements, crawl spaces or other enclosed portions of buildings built in areas underlain at relatively shallow depths by granitic crystalline and gneissic bedrock. Therefore the likelihood of encountering radon in concentrations exceeding applicable health standards on the

subject site should be considered. Additional information regarding radon and radon-resistant building design can be obtained from the EPA (e.g., www.epa.gov/radon) as well as from many local building and/or health departments.

It is GROUND's opinion that radon testing be performed in the building, on-site, after construction is completed. However, we understand that incorporating sufficient ventilation and other measures into a structure to address radon accumulation during construction is significantly less costly than installing them after construction has been completed. The Architect should consider radon mitigation measures for the proposed residence and incorporates appropriate systems into the design, as needed.

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, existing asphalt/concrete, 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. Soft soils under the ditch should be excavated completely and replaced with properly compacted fill.

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.

Tree trunks and roots may be present within, under, or adjacent to the proposed building footprint and adjacent to the existing building. The contractor should take care to assure that all tree roots and organic materials, if present, are removed prior to placement of fill or foundation/floor slab elements. Relatively deep excavations may be required to accomplish proper removal of roots and organic materials. The geotechnical engineer should be retained to observe the removal process and the subsequent fill placement.

Cobbles and boulders may be encountered locally. Therefore, such materials (commonly found in in alluvial sand and clay deposits, for example) may be present even where not called out in the material descriptions herein.

Drainage During Construction The contractor should take pro-active measures to control surface waters during construction, to direct them away from excavations and into appropriate drainage structures. Wetting of foundation soils during construction can have adverse effects on the performance of project improvements.

Filled areas should be graded to drain effectively at the end of each work day.

Use of Existing Fill Soils Fill soils were not recognized in the test holes, but may be present on-site. Actual contents and composition of all 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. Existing fill soils should be evaluated on a case by case basis regarding possible re-use.

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

Fragments of rock, cobbles, and inert construction debris (e.g., concrete or asphalt) larger than **3 inches** in maximum dimension will require special handling and/or placement to be incorporated into project fills. In general, such materials should be placed as deeply as possible in the project fills. A geotechnical engineer should be consulted regarding appropriate information for usage of such materials on a case-by-case basis when such materials have been identified during earthwork. Standard parameters 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. Soils imported for use as common fill at the site should exhibit **30 percent or less** passing the No. 200 Sieve and should have a plasticity index of **15 or lower.** Representative samples of the

materials proposed for import should be tested and approved prior to transport to the site.

Select, Granular Fill Soil imported as select, granular fill should meet the criteria for CDOT Class 1 Structure Backfill as tabulated below, and be approved for the proposed use. All imported soils should be tested and approved prior to transport to the site.

CDOT CLASS 1 STRUCTURE BACKFILL

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

Fill Platform Preparation Prior to filling, the top 12 inches of in-place materials on which fill soils will be placed (except for trench bottoms where bedding 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. A surface to receive fill must be effectively stable prior to placement of fill, including trench bottoms prior to placement of bedding.

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

Soils that classify as ML, MH, CL or CH should be compacted to **at least 95 percent** of the maximum dry density at moisture contents from **optimum moisture content to 4 percent above the optimum moisture content** as determined by ASTM D698, the 'standard Proctor.'

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 specified 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, this procedure should not be used for the following reasons:

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

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

Whenever possible, excavations should 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. For a 12-foot fill, for example, that corresponds to settlement of about 2 inches. If fill placement is performed properly, 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 specified 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, continuous 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.

The contractor should be prepared to handle cobbles and large boulders, where encountered.

Temporary, un-shored excavation slopes up to 15 feet in height be cut no steeper than 1.5:1 (horizontal : vertical) in the on-site soils in the absence of seepage. Some surface sloughing may occur on the slope faces at these angles. The risk of slope instability will be significantly increased in areas of seepage along excavation slopes.

Should site constraints prohibit the use of the specified slope angles, temporary shoring should be used. The shoring should be designed to resist the lateral earth pressure exerted by structure, traffic, equipment, and stockpiles. GROUND can provide shoring design upon request.

Groundwater was not encountered in the test holes at the time of drilling. Therefore, 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, a geotechnical engineer should be retained to evaluate the conditions and provided additional information, as appropriate.

Good surface drainage should be provided around temporary excavation slopes to direct surface runoff away from the slope faces. A properly designed swale should be provided at the top of the excavations. In no case should water be allowed to pond at the site. Slopes should be protected against erosion. Erosion along the slopes will result in sloughing and could lead to a slope failure. Any excavations in which personnel will be working must comply with all OSHA Standards and Regulations (CFR 29 Part 1926). 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

Pipe Support The bearing capacity of the site soils appeared adequate, in general, for support of the proposed utilities. The pipe + contents 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, the allowable passive soil pressure and sliding friction provided above in the *Lateral Loads* section should be considered.

Trench Backfilling Some settlement of compacted soil trench backfill materials should be anticipated, even where all the backfill is placed and compacted correctly. Typical settlements are on the order of 1 to 2 percent of fill thickness. However, the need to compact to the lowest portion of the backfill must be balanced against the need to protect the pipe from damage from the compaction process. Some thickness of backfill may need to be placed at compaction levels lower than specified (or smaller compaction equipment used together with thinner lifts) to avoid damaging the pipe. Protecting the pipe in this manner can result in somewhat greater surface settlements. Therefore, although other alternatives may be available, the following options are presented for consideration:

Controlled Low Strength Material Because of these limitations, the entire depth of the trench (both bedding and common backfill zones) should be backfilled with "controlled low strength material" (CLSM), i.e., a lean, sand-cement slurry, "flowable fill," or similar material along all trench alignment reaches with low tolerances for surface settlements.

CLSM used as pipe bedding and trench backfill should exhibit a 28-day unconfined compressive strength between 50 to 150 psi so that re-excavation is not unusually difficult.

Placement of the CLSM in several lifts or other measures likely will be necessary to avoid 'floating' the pipe. Measures also should be taken to maintain pipe alignment during CLSM placement.

Compacted Soil Backfilling 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.

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

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

As discussed above, CLSM or similar material should be used 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, then with regard to potential migration of fines into the pipe bedding, design and installation should follow ASTM D2321, Appendix X1.8. If the granular bedding does not meet filter criteria for the enclosing soils, then non-woven filter fabric (e.g., TenCate Mirafi® 140N, or the equivalent) should be placed around the bedding to reduce migration of fines into the bedding which can result in severe, local surface settlements. Where this protection is not provided, settlements can develop/continue several months or years after completion of the project.

In addition, clay or concrete cut-off walls should be installed at the margins of the pad to interrupt the granular bedding section to reduce the rates and volumes of water transmitted along utility alignments which can contribute to migration of fines. We anticipate that cut-off walls also will be beneficial with regard to limiting migration of methane along the pipe bedding.

If granular bedding is specified, the contractor should not anticipate that significant volumes of on-site soils will 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 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 site slopes, including retained slopes, descend toward a building, and the toe-of-slope (-wall) is less than 3 times the total slope (wall) height from the building, then an interceptor drain should be installed between the building and the slope. Ideally, the interceptor drain should be installed at least 10 feet from the building or along the axis of the swale between the building and the toe-of-

slope (-wall). Geotechnical comments for interceptor drain systems are provided in the *Subsurface Drainage* section of this report.

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.

SUBSURFACE DRAINAGE

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

Although inclusion of an underdrain system is common for buildings like the proposed fire station, 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. However, an underdrain system should be used at the contact between the select, granular fill materials and the native clayey fill materials.

Additionally, if a below grade level or a partially below-grade level is added to any of the proposed buildings, then an underdrain system should be included. Crawl spaces typically are not considered below-grade 'levels' in this sense, but provision of an underdrain for the crawl space may be beneficial in this case, given the potential for water infiltration from the use and washing of fire trucks and other vehicles at the facility. Damp-proofing also 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.

Wall drain elements (if any) should be in hydraulic connectivity with an underdrain system. Otherwise, collected surface waters should not be routed into an underdrain.

An underdrain system should be tested by the contractor after installation and after placement and compaction of the overlying backfill to verify that the system functions properly. Like other components of the structure, periodic maintenance of an underdrain system after completion should be anticipated to keep it functioning as intended.

Geotechnical Parameters for Underdrains Where a granular fill section underdrain system or where any other type of underdrain system is included in project drainage design, it should be designed in accordance with the parameters provided below. The actual granular fill section drain layout, outlets, and locations should be developed by a civil engineer. A typical, cross-section detail of a granular fill section drain for this project is provided on Figure 10. A typical, cross-section detail of a deep foundation system underdrain for this project is provided on Figure 11.

1) An underdrain system for a building should consist of perforated, rigid, PVC collection pipe at least **4 inches** in diameter, non-perforated, rigid, PVC discharge pipe at least **4 inches** in diameter, free-draining gravel, and filter fabric.

2) The free-draining gravel should contain **less than 5 percent** passing the No. 200 Sieve and **more than 50 percent** retained on the No. 4 Sieve, and have a maximum particle size of **2 inches**.

   Each collection pipe should be surrounded on the sides and top (only) with **6 or more inches** of free-draining gravel.

3) The gravel surrounding the collection pipe(s) should be wrapped with filter fabric (Tencate MiraFi 140N® or the equivalent) to reduce the migration of fines into the drain system.

4) The granular fill drain should be placed at the margins of the (base of the) granular fill section and with laterals extending into the interior of the building footprint so that no portion of the select, granular fill section bottom is more than **75 feet** from the perimeter drain or a lateral.

5) The high point(s) of collection pipe flow lines for the granular fill section underdrain system should be at least **2 inches** below the interface between the select granular fill and the underlying clayey soils, whether native materials or site-derived fill, depending on the fill section selected. Multiple high points may 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 **2 inches** of differential movement after installation **along a 50-foot run**.

6) The granular fill section underdrain system should be designed to discharge at least **10 gallons per minute** of collected water.

7) Underdrain 'clean-outs' should be provided at intervals of no more than **200 feet** to facilitate maintenance of the underdrains. Clean-outs also should be provided at collection and discharge pipe elbows of **60 degrees or more**.

8) The underdrain discharge pipes should be connected to one or more sumps from which water can be removed by pumping, or to outlet(s) for gravity discharge. We suggest that collected waters be discharged directly into the storm sewer system, if possible.

PAVEMENT SECTIONS

A pavement section is a layered system designed to distribute concentrated traffic loads to the subgrade. Performance of the pavement structure is directly related to the physical properties of the subgrade soils and traffic loadings. Pavement designs for the project parking and driveways were developed in general accordance with the design guidelines and procedures of the American Association of State Highway and Transportation Officials (AASHTO), and the Colorado Department of Transportation (CDOT) and local pavement construction practice.

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

Subgrade Materials Based on the results of our field and laboratory studies, project subgrade materials in the proposed access road alignment areas consisted largely of sands and clays. These materials typically classified as A-2-6 to A-7-5 in the pavement test hole soils in accordance with the AASHTO classification system, with Group Index values generally between 0 and 14. These types of soils generally provide relatively poor to moderate pavement support.

For the site soils, an R-value of 15 was estimated to be representative of site soils. The R-value was converted to a resilient modulus of 4,195 psi and used to develop the pavement sections.

It is important to note that significant decreases in soil support as quantified by the resilient modulus have been observed as the moisture content increases above the optimum. Therefore, pavements that are not properly drained may experience a loss of the soil support and subsequent reduction in pavement life.

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

An ESAL value of 21,900 (corresponding to an EDLA value of 3 for a 20-year design life) was assumed for parking stalls and driveways for light vehicles (automobiles and similar). An ESAL value of 365,000 (corresponding to an EDLA value of 50 for a 20-year design life) was assumed for the fire truck routes.

If design traffic loadings differ significantly from these assumed values, GROUND should be notified to re-evaluate the pavement sections below.

Pavement Sections The soil resilient modulus and the anticipated ESAL values were used to determine the required structural number for the project pavements. The required structural number was then used to develop minimum pavement sections for the subject facility. Pavement sections were based on the DARWin™ computer program that solves the 1993 AASHTO pavement design equation. Pavement parameters and calculations are summarized in *Appendix A*. A Reliability Level of 90 percent was utilized develop the pavement sections, together with a Serviceability index loss of 2.0. An overall standard of deviation of 0.44 also was used (0.34 for concrete pavement). Structural coefficients of 0.40 for hot bituminous asphalt and 0.12 for aggregate base course were used, respectively. The resultant minimum pavement sections that should be used at the facility are tabulated below.

MINIMUM PAVEMENT SECTIONS

Location:	Fire Truck Routes	Light Vehicle Parking
Flexible Sections:	-	6 inches Asphalt *or* 4 inches Asphalt / 6 inches Aggregate Base
Concrete Section:	7 in. PCC / 6 in. Ag. Base [1]	(6 in. PCC / 6 in. Ag. Base)

[1] We anticipate that areas where fire trucks are parked, however, or the weight taken on outriggers, etc., will require thicker concrete sections to avoid punching

failures or other breaks in the concrete. Significant reinforcement of the concrete likely will be needed, as well. We suggest a 9-inch, reinforced, concrete section over 6 inches of CDOT Class 6 Aggregate Base Course for such areas, but that section should be established by a structural engineer. A vertical modulus of subgrade reaction of **100 tcf (116 pci)** may be used to evaluate a concrete section where the concrete + base course bear on properly compacted granular fill.

All concrete sections should be underlain by 6 inches of properly compacted CDOT Class 5 ot Class 6 Aggregate Base Course.

Pavement Materials Asphalt pavement should consist of a bituminous plant mix composed of a mixture of aggregate and bituminous material. Asphalt mixture(s) should meet the requirements of a job-mix formula established by a qualified engineer as well as applicable municipal design requirements.

Where composite flexible sections are used, the aggregate base material should meet the criteria of CDOT Class 6 aggregate base course. Base course should be placed in and compacted in accordance with the parameters and considerations in the *Project Earthwork* section of this report.

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

These concrete mix design criteria should be coordinated with other project requirements including any criteria for sulfate resistance presented in the *Water-Soluble Sulfates* section of this report. To reduce surficial spalling resulting from freeze-thaw cycling, we suggest that pavement concrete meet the requirements of CDOT Class P

concrete. In addition, the use of de-icing salts on concrete pavements during the first winter after construction will increase the likelihood of the development of scaling. Placement of flatwork concrete during cold weather so that it is exposed to freeze-thaw cycling before it is fully cured also increases its vulnerability to scaling. Concrete placing during cold weather conditions should be blanketed or tented to allow full curing. Depending on the weather conditions, this may result in 3 to 4 weeks of curing, and possibly more.

Concrete pavements should contain sawed or formed joints. CDOT and various industry groups provide guidelines for proper design and concrete construction and associated jointing. In areas of repeated turning stresses, concrete pavement joints should be fully tied and doweled. We suggest that civil pavement design consider joint layout in accordance with CDOT's M standards, found at the CDOT website: www.dot.state.co.us/DesignSupport/.

Subgrade Preparation Although subgrade preparation to a depth of 12 inches is typical in the project area, pavement performance commonly can be improved by a greater depth of moisture-density conditioning of the soils. Therefore, the following subgrade preparation is being provided:

Remedial Earthwork Shortly before paving, the pavement subgrade should be overexcavated to a depth of **2 feet or more**, moisture-conditioned and properly re-compacted. However, if performance like a slab-on-grade floor or structural floor is desired, than the exterior flatwork should be supported on deep foundations and span over a void (for performance like structural floor) or constructed on a fill section similar to the one provided for slab-on-grade floors (for performance like a slab-on-grade floor.

Subgrade preparation should extend the full width of the pavement from back-of-curb to back-of-curb (or 3 feet beyond the pavement section if curbs are not planned). The subgrade for sidewalks and other project hardscaping also should be prepared in the same manner.

Criteria and standards for fill placement and compaction are provided in the *Project Earthwork* section of this report. The contractor should be prepared either to dry the subgrade materials or moisten them, as needed, prior to compaction.

Where adequate drainage cannot be achieved or maintained, excavation and replacement should be undertaken to a greater depth, in addition to the edge drains discussed below.

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

Additional Considerations The collection and diversion of surface drainage away from paved areas is extremely important to 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 should be used 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. Where the subgrade soils are expansive, wetting also typically results in increased pavement heave. In relatively flat areas where design drainage gradients necessarily are small, subgrade settlement or heave can obstruct proper drainage and yield increased infiltration, exaggerated distress, etc. (These considerations apply to project flatwork, as well.)

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

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

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

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

the pavement sections, however, pavement maintenance measures will be needed on an accelerated schedule.

CLOSURE

Geotechnical Review The author of this report or a GROUND principal should be retained to review project plans and specifications to evaluate whether they comply with the intent of the measures discussed in this report. The review should be requested in writing.

The geotechnical conclusions and parameters presented in this report are contingent upon observation and testing of project earthworks by representatives of GROUND. If another geotechnical consultant is selected to provide materials testing, then that consultant must assume all responsibility for the geotechnical aspects of the project by concurring in writing with the parameters in this report, or by providing alternative parameters.

Materials Testing Grand Fire Protection District No. 1 should consider retaining a geotechnical engineer to perform materials testing during construction. The performance of such testing or lack thereof, however, in no way alleviates the burden of the contractor or subcontractor from constructing in a manner that conforms to applicable project documents and industry standards. The contractor or pertinent subcontractor is ultimately responsible for managing the quality of his work; furthermore, testing by the geotechnical engineer does not preclude the contractor from obtaining or providing whatever services that he deems necessary to complete the project in accordance with applicable documents.

Limitations This report has been prepared for Grand Fire Protection District No. 1 as it pertains to design and construction of the proposed facility and related improvements as described herein. It may not contain sufficient information for other parties or other purposes.

In addition, GROUND has assumed that project construction will commence by Summer, 2016. Any changes in project plans or schedule should be brought to the attention of a geotechnical engineer, in order that the geotechnical conclusions in this report may be re-evaluated and, as necessary, modified.

The geotechnical conclusions and criteria 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, then a geotechnical engineer should be retained at once, so that re-evaluation of the conclusions for this site may be made in a timely manner. In addition, a contractor who 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.

ALL DEVELOPMENT CONTAINS INHERENT RISKS. It is important that ALL aspects of this report, as well as the estimated performance (and limitations with any such estimations) of proposed improvements are understood by the Client or properly conveyed to any future owners. Utilizing these criteria and measures herein for planning, design, and/or construction constitutes understanding and acceptance of the conclusions with regard to risk and other information provided herein, associated improvement performance, as well as the limitations inherent within such estimates.

If any information referred to herein is not well understood, then the Client, or anyone using this report, should contact the author or a GROUND principal immediately. We will be available to meet to discuss the risks and remedial approaches presented in this report, as well as other potential approaches, upon request.

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 conclusions contained herein.

This document, together with the concepts and conclusions presented herein, as an instrument of service, is intended only for the specific purpose and client for which it was prepared. Reuse of or improper reliance on this document without written authorization and adaption by GROUND Engineering Consultants, Inc., shall be without liability to GROUND Engineering Consultants, Inc.

GROUND appreciates the opportunity to complete this portion of the project and welcomes the opportunity to provide the Grand Fire Protection District No. 1 with a cost proposal for construction observation and materials testing.

Sincerely,

GROUND Engineering Consultants, Inc.

Ben Fellbaum

COLORADO LICENSED
BRIAN H. RECK
48265
PROFESSIONAL ENGINEER
11/05/2015

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

MAIN TRAVELED WAY

CR 40

N 88°46'35" E
18.78'

S 00°35'59

(P.O.

S 77°14'06" E
140.10'

30' FROM
CENTERLINE
(TYPICAL)

C1

30'

1

P-1

PROPOSED
BUILDING

2

40'

S 29°55'02" E
100.00'

P-2

3

N 60°05'06" E
120.00'

P-3

N 03°08'58" E 445.45'

S 00°35'59" W 476.56'

BASIS OF BEARING

220,112.77 SQ. FT.
AND/OR
5.05 ACRES+/-

N 79°11'44" W 506.15'

S 00°35'59" W 1936.28'

SITE PLAN PROVIDED BY OTHERS

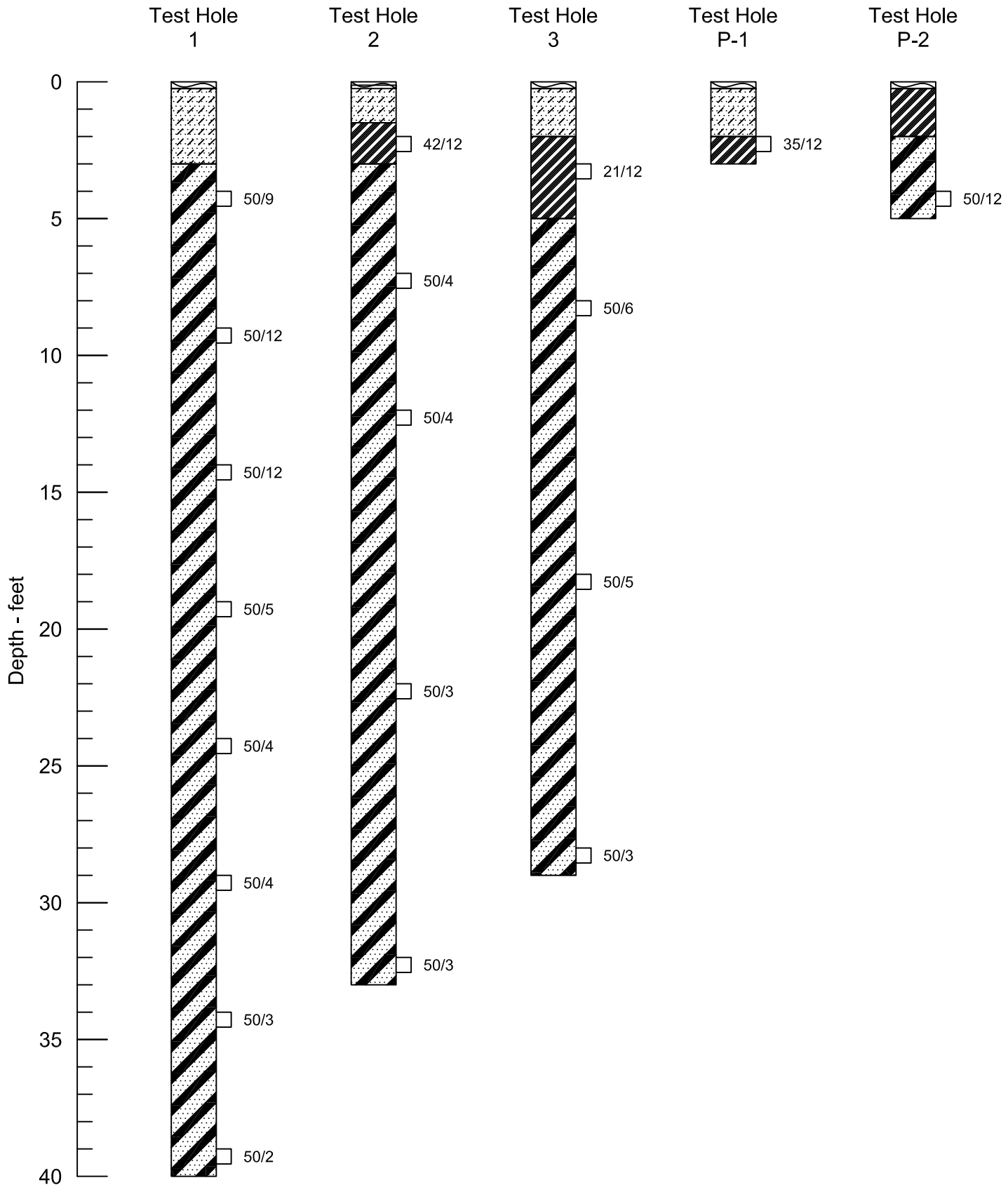
1
Indicates test hole number and approximate location.

(Not to Scale)

GROUND
ENGINEERING CONSULTANTS

LOCATION OF TEST HOLES

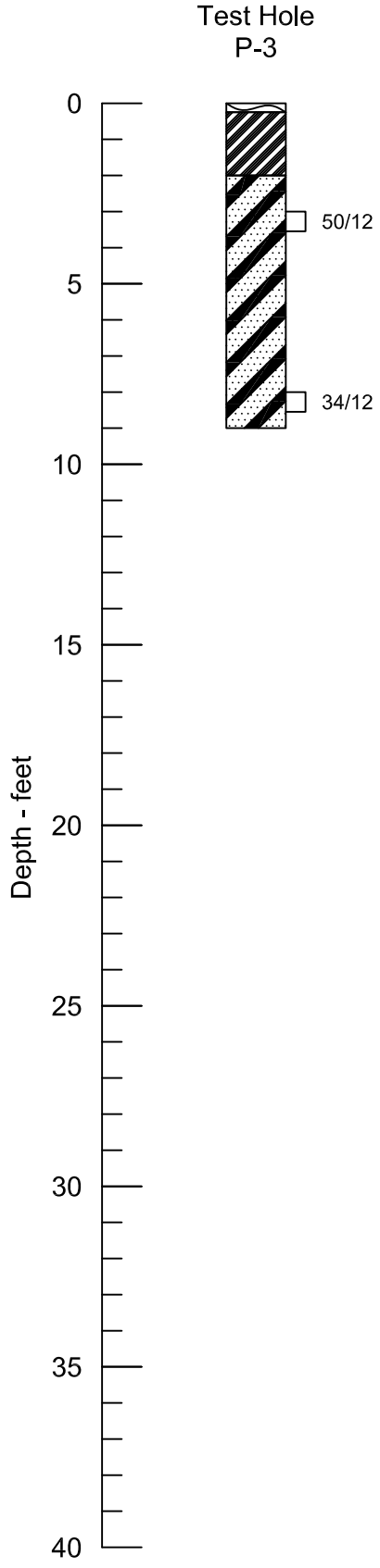
JOB NO.: 15-2013	FIGURE: 1
CADFILE NAME: 2013SITE.DWG	



GROUND
ENGINEERING CONSULTANTS

LOGS OF TEST HOLES

JOB NO.: 15-2013	FIGURE: 2
CADFILE NAME: 2013LOG01.DWG	



GROUND
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LOGS OF TEST HOLES

JOB NO.: 15-2013	FIGURE: 3

CADFILE NAME: 2013LOG02.DWG

LEGEND:

Topsoil

Sand and Clay: Fine to medium clayey to silty sands to sandy clays and contained local gravels. They were low to highly plastic, medium dense to dense or medium to very stiff, dry to slightly moist, and light brown to brown in color. Cobbles and boulders were also noted on the surface.

Weathered Sandstone, Siltstone, and Claystone: Interbedded and was fine to coarse grained, low to highly plastic, medium hard to very hard, slightly moist, and light brown to red brown and gray in color.

Sandstone, Siltstone, and Claystone Bedrock: Interbedded and was fine to coarse grained, low to highly plastic, medium hard to very hard, slightly moist to wet, and light brown to red brown, gray, and green in color.

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

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

NOTES:

1) Test holes were drilled on 09/29 and 09/30/2015 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 not measured and the logs of the test holes are drawn to depth.
4) The test hole locations and elevations should be considered accurate only to the degree implied by the method used.
5) The lines between materials shown on the test hole logs represent the approximate boundaries between material types and the transitions may be gradual.
6) Groundwater was not encountered during drilling. Ground water levels can fluctuate seasonally and in response to landscape irrigation.
7) The material descriptions on this legend are for general classification purposes only. See the full text of this report for descriptions of the site materials and related information.
8) All test holes were immediately backfilled upon completion of drilling, unless otherwise specified in this report.

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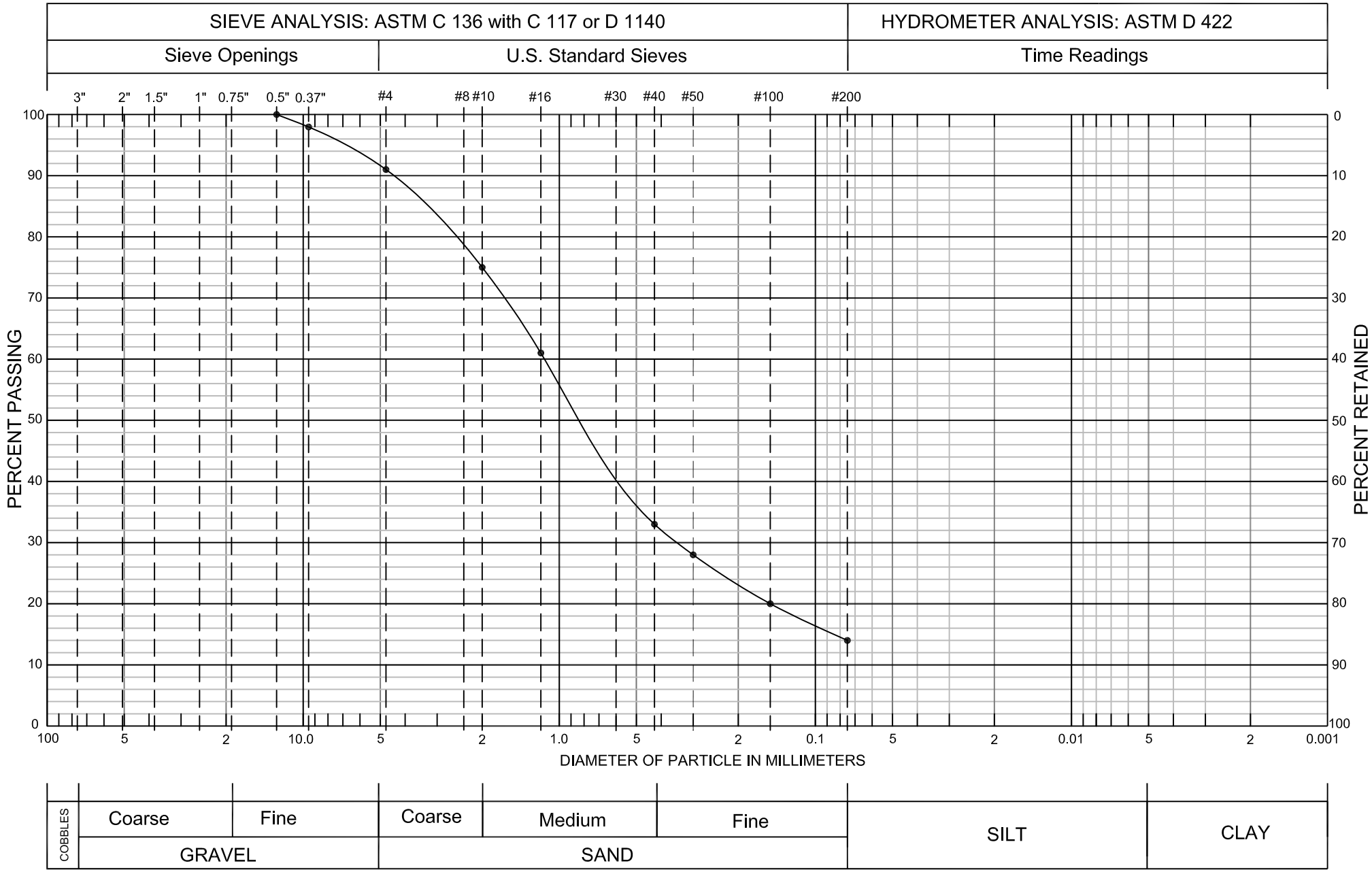
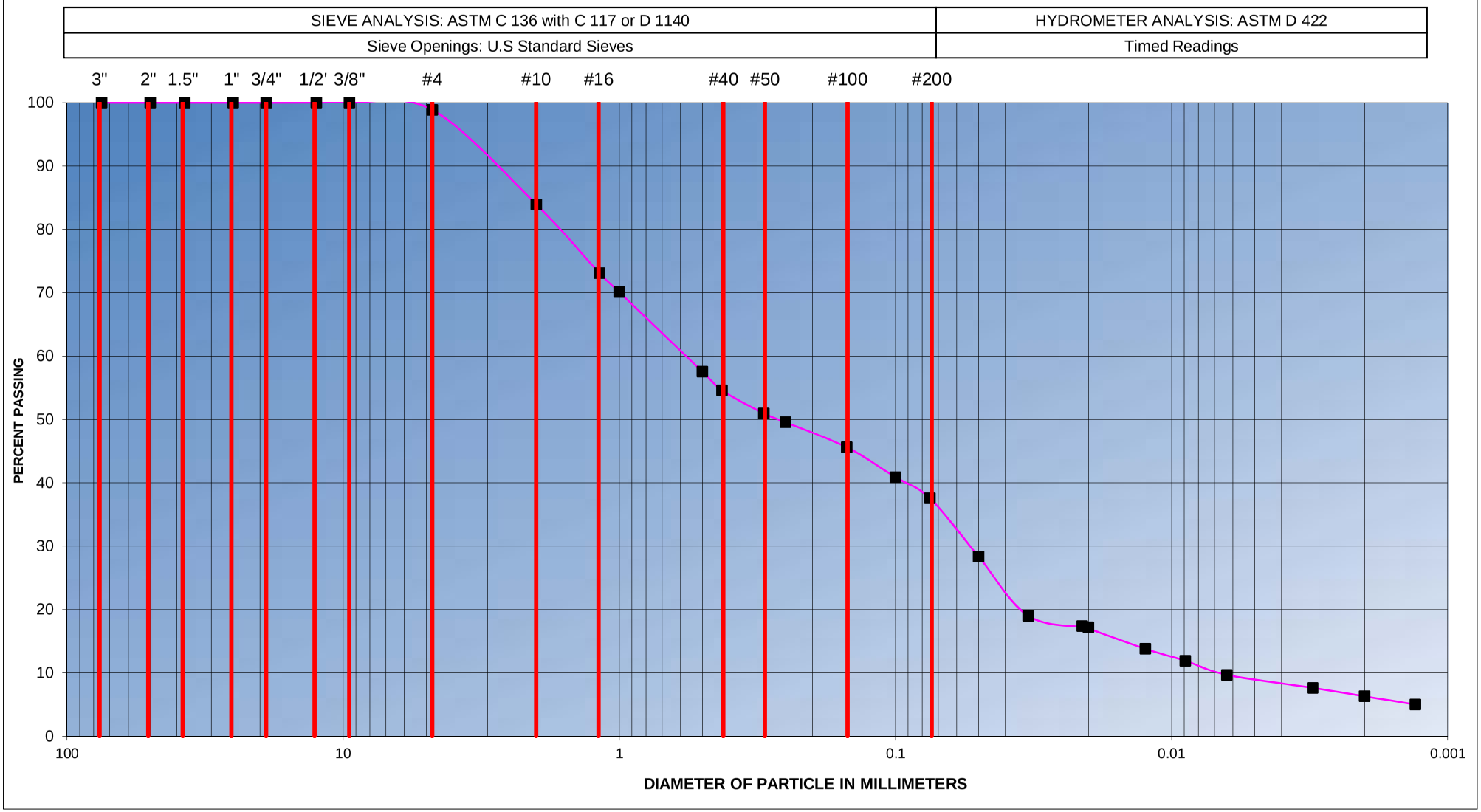
LEGEND AND NOTES

JOB NO.: 15-2013	FIGURE: 4
CADFILE NAME: 2013LEG.DWG	



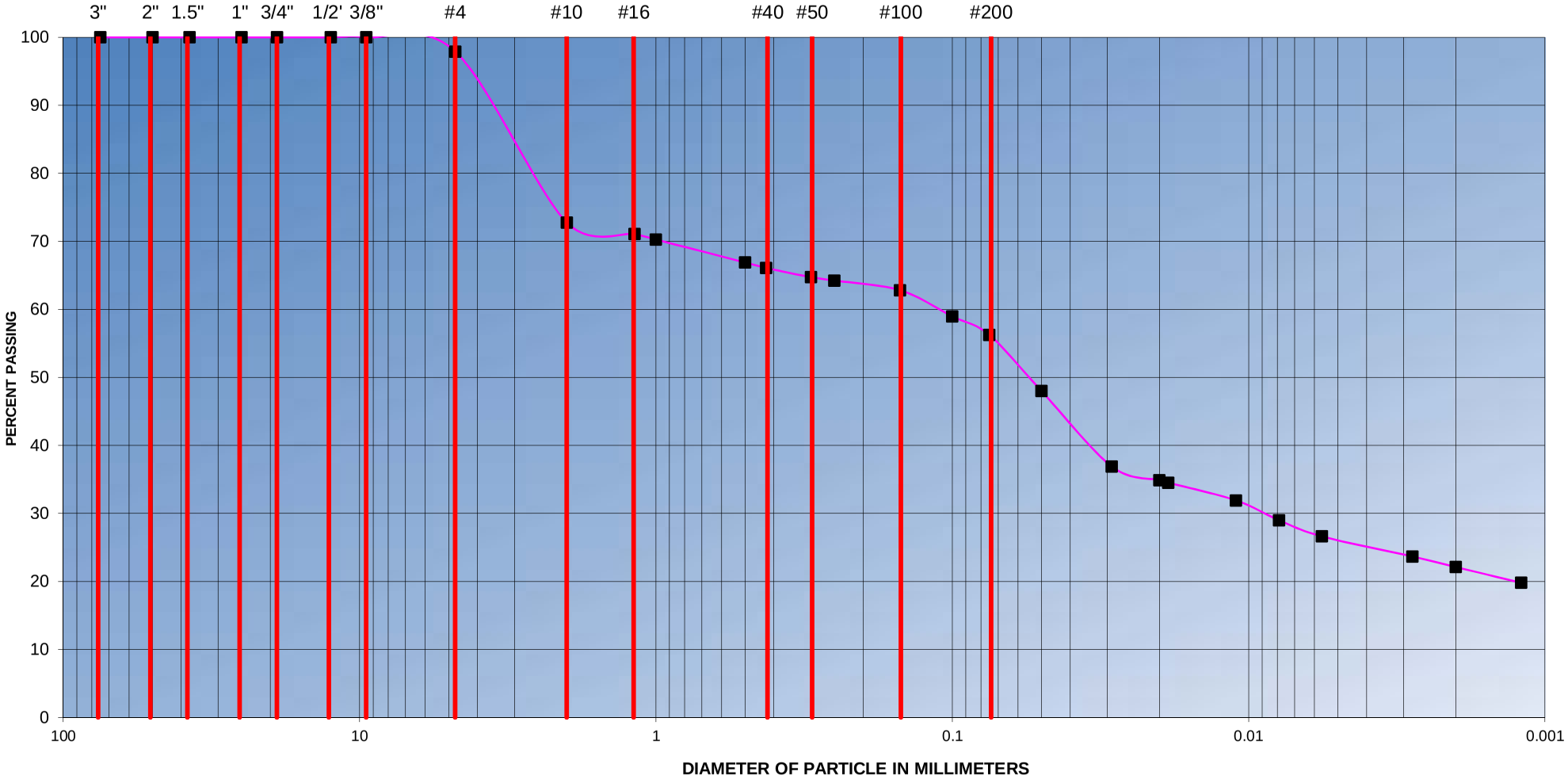
Sample Location: Test Hole 1 at 4 feet	Gravel: 9 % Sand: 77 % Silt and Clay: 14 %	GROUND ENGINEERING CONSULTANTS
Sample Description: Clayey SAND; A-2-6(0)	LL = 36 PI = 14	GRADATION TEST RESULTS
		JOB NO.: 15-2013 FIGURE: 5
		CADFILE NAME: 2013GRAD.DWG



COBBLE	Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY
	GRAVEL		SAND				

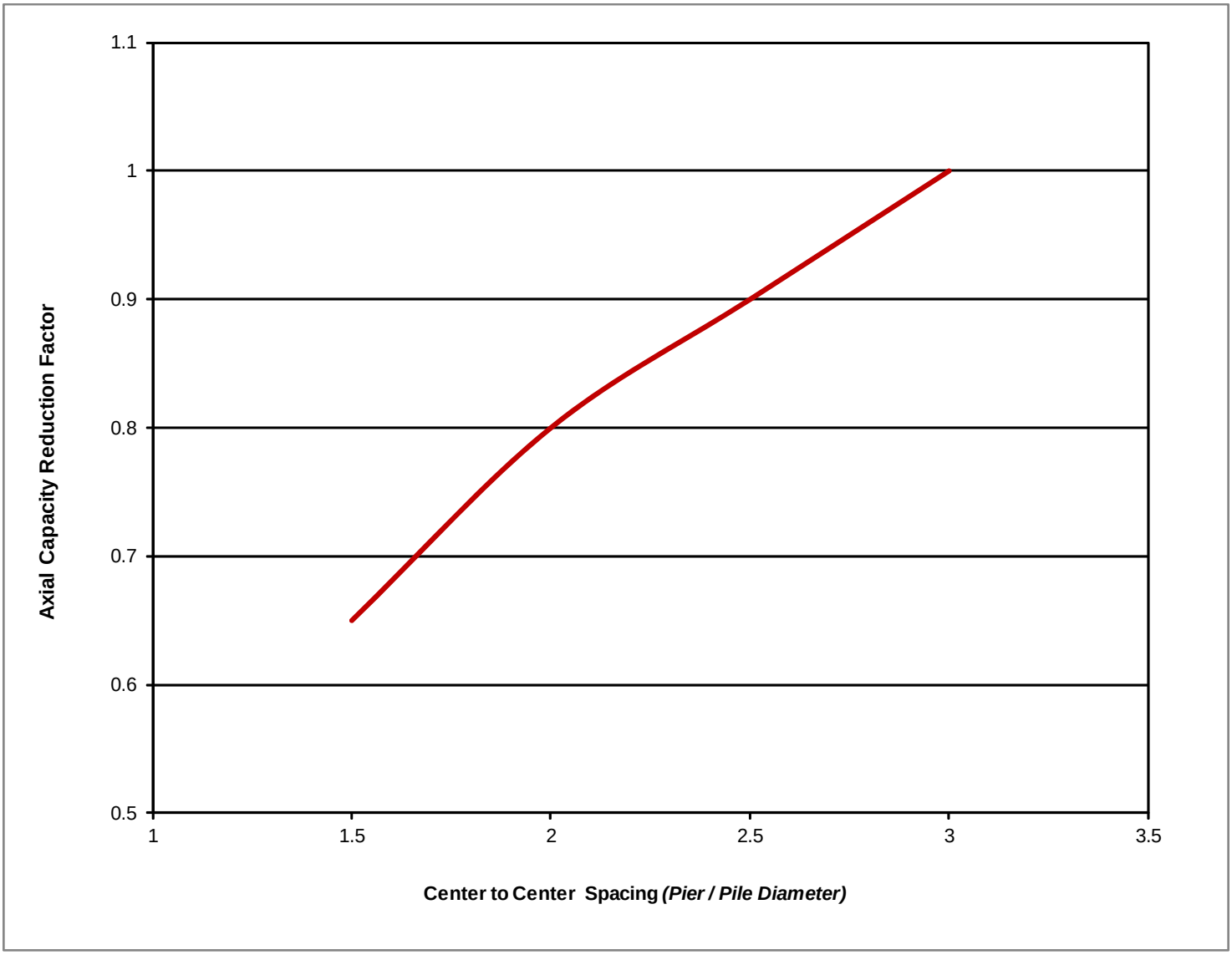
Sample of: Silty SAND	Gravel 1% Sand 61% Silt and Clay 38%	GROUND ENGINEERING CONSULTANTS
From: TH 2 at 2 feet	Liquid Limit 47 Plasticity Index 11	GRADATION TEST RESULTS
		JOB NO.: 15-2013
		FIGURE: 6

SIEVE ANALYSIS: ASTM C 136 with C 117 or D 1140	HYDROMETER ANALYSIS: ASTM D 422
Sieve Openings: U.S Standard Sieves	Timed Readings



COBBLE	Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY
	GRAVEL		SAND				

Sample of: CLAYSTONE Bedrock	Gravel 2% Sand 42% Silt and Clay 56%	GROUND ENGINEERING CONSULTANTS
From: TH 3 at 8 feet	Liquid Limit 48 Plasticity Index 24	GRADATION TEST RESULTS
		JOB NO.: 15-2013
		FIGURE: 7



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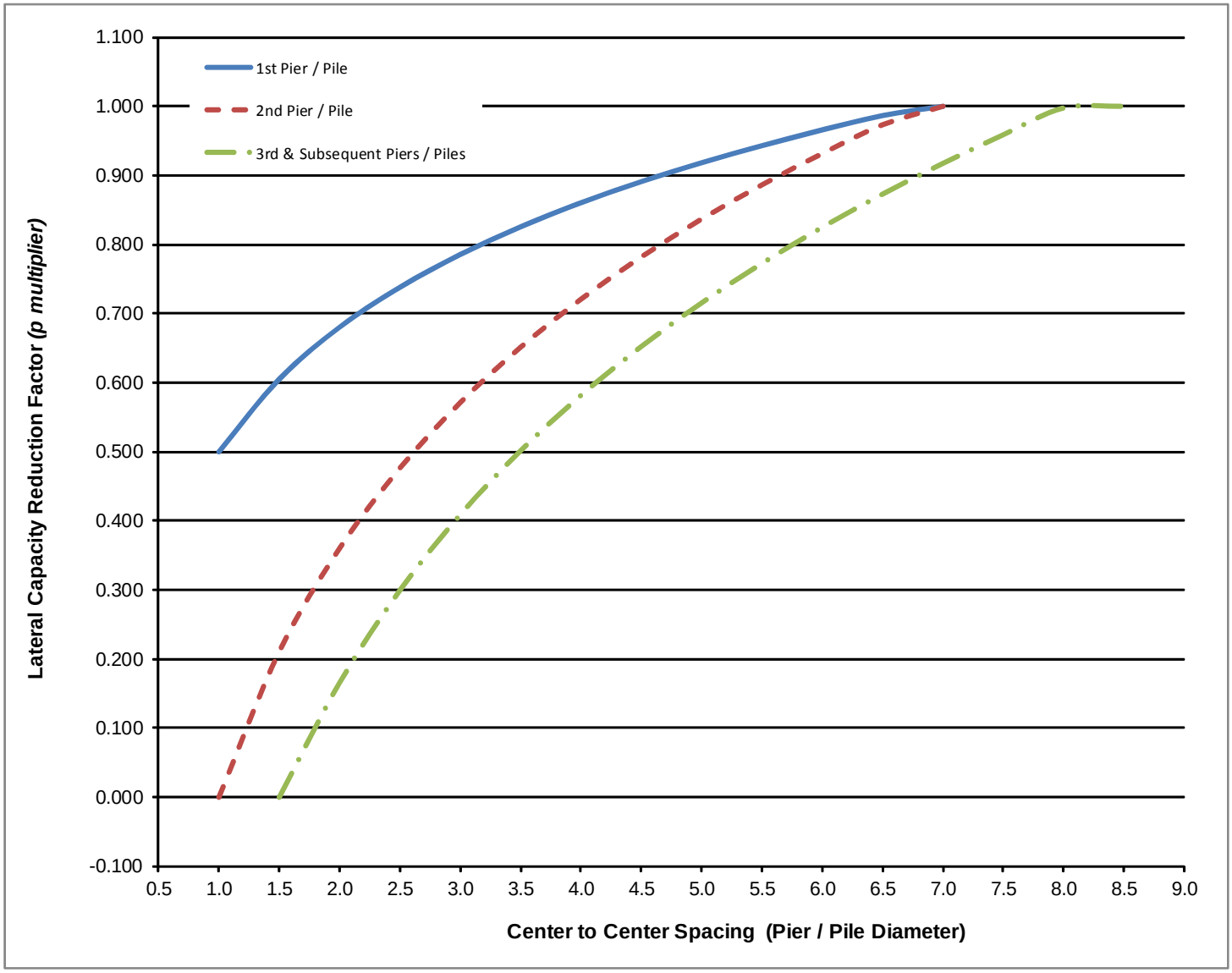
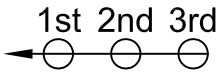
AXIAL CAPACITY
REDUCTION FACTORS
FOR CLOSELY SPACED PIERS / PILES

JOB NO.: 15-2013	FIGURE: 8

CADFILE NAME: 2013APR.DWG





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

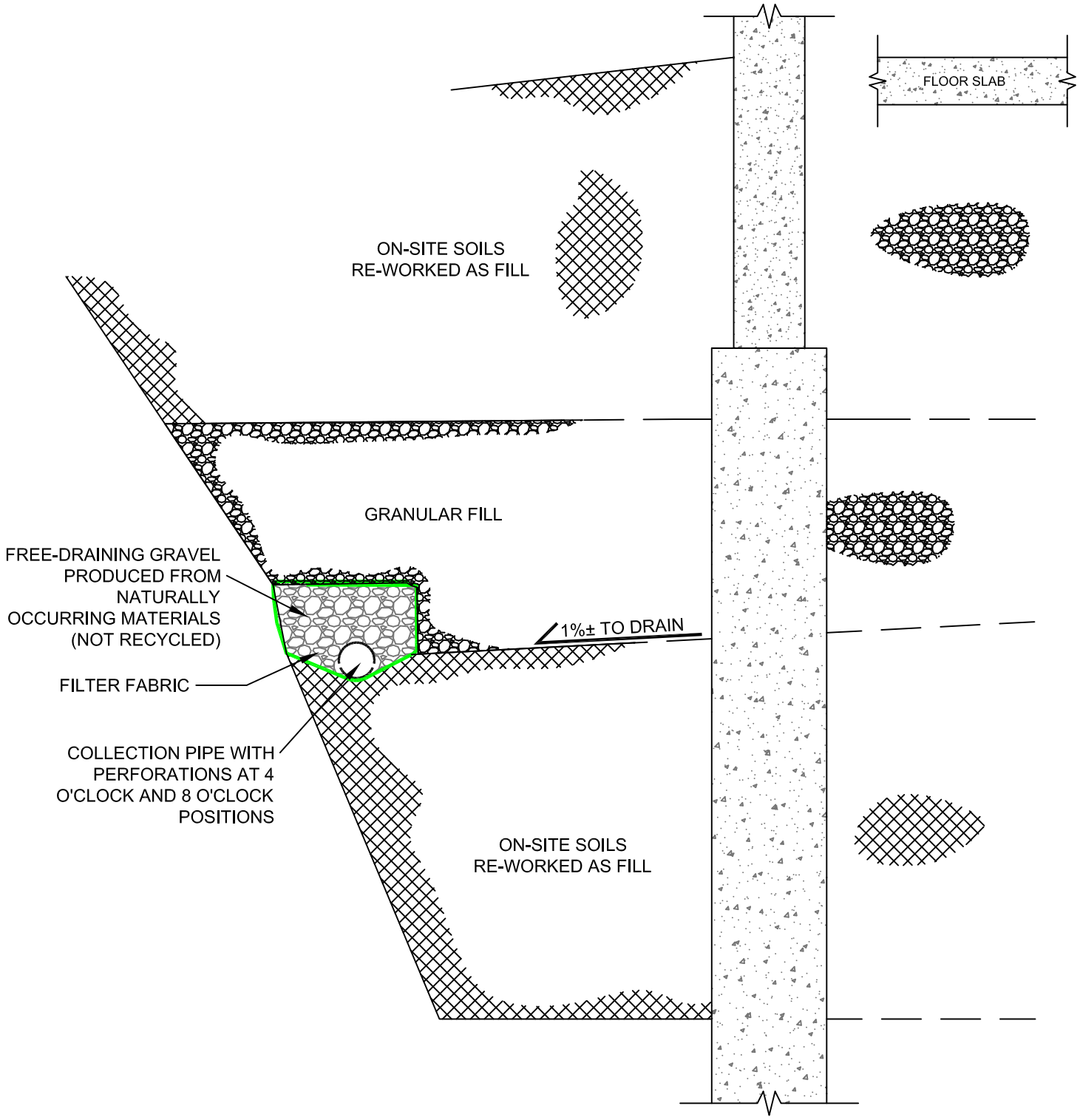
For lateral loads oriented perpendicular to the row of piers / piles, use the 1st pier / pile p-multiplier.

GROUND
ENGINEERING CONSULTANTS

LATERAL CAPACITY
REDUCTION FACTORS
FOR CLOSELY SPACED PIERS / PILES

JOB NO.: 15-2013	FIGURE: 9

CADFILE NAME: 2013LLPR.DWG



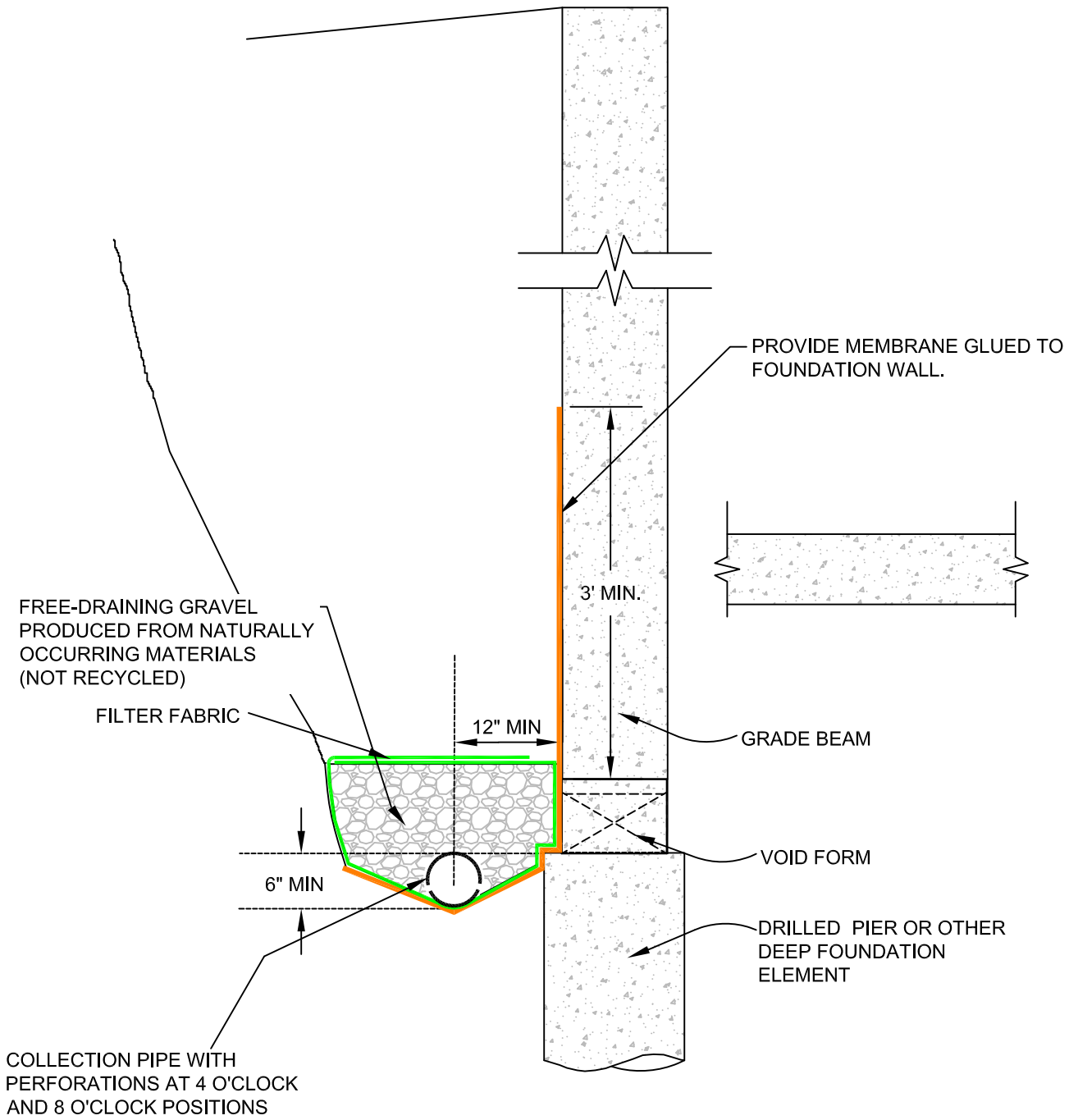
SEE TEXT FOR ADDITIONAL INFORMATION

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.

GROUND
ENGINEERING CONSULTANTS

TYPICAL GRANULAR DRAIN DETAIL

JOB NO.: 15-2013	FIGURE: 10
CADFILE NAME: 2013DRAIN01.DWG	



SEE TEXT FOR ADDITIONAL INFORMATION

AT THE TIME OF PREPARATION OF THIS TYPICAL DETAIL, BUILDING AND DRAINAGE PLANS HAD NOT BEEN DEVELOPED FULLY. MODIFICATION OF THE DETAIL MAY BE NECESSARY AS PART OF FINAL DESIGN. GROUND CAN BE CONTACTED BY THE CIVIL ENGINEER IN THIS REGARD

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

GROUND
ENGINEERING CONSULTANTS

TYPICAL UNDERDRAIN DETAIL

JOB NO: 15-2013	FIGURE: 11
CADFILE NAME: 2013DRAIN02.DWG	

GROUND
ENGINEERING CONSULTANTS

TABLE 1
SUMMARY OF LABORATORY TEST RESULTS

Sample Location		Natural	Natural	Gradation		Passing	Atterberg Limits		Percent	Unconfined	USCS	AASHTO	
Test Hole No.	Depth *(feet)*	Moisture Content *(%)*	Dry Density *(pcf)*	Gravel *(%)*	Sand *(%)*	No. 200 Sieve *(%)*	Liquid Limit	Plasticity Index	Swell* *(Surcharge Pressure)*	Compressive Strength *(psf)*	Classifi-cation	Classifi-cation (GI)	Soil or Bedrock Type
1	4	6.2	129.5	9	77	14	36	14		8,630	SC	A-2-6 (0)	SANDSTONE Bedrock
1	9	17.5	100.7			16	44	13	-0.8 (200 psf)		SM	A-2-7 (0)	SANDSTONE Bedrock
2	2	14.7	94.3	1	61	38	47	11		1,600	SM	A-2-7 (1)	Weathered SANDSTONE
2	12	18.9	99.9			63	54	24	-1.3 (1,500 psf)		MH	A-7-5 (14)	SILTSTONE Bedrock
3	3	21.5	82.1			8	51	11			SM	A-2-7 (0)	Weathered SANDSTONE
3	8	15.0	124.1	2	42	56	48	24	11.4 (100 psf)		CL	A-7-6 (11)	CLAYSTONE Bedrock
P-1	2	21.2	87.0			12	48	11			SM	A-2-7 (0)	Weathered SANDSTONE
P-2	4	20.5	89.0			22	56	21	4.0 (200 psf)		SM	A-2-7 (0)	SILTSTONE Bedrock
P-3	3	15.7	99.1			30	47	16			SM	A-2-7 (0)	SANDSTONE Bedrock

** Negative swell indicates consolidation*

Job No. 15-2013

GROUND
ENGINEERING CONSULTANTS

TABLE 2
SUMMARY OF SOIL CORROSION TEST RESULTS

Sample Location		Water Soluble Sulfates	pH	Redox Potential	Sulfide Reactivity	Resistivity	Soil or Bedrock Type
Test Hole No.	Depth *(feet)*	*(%)*		*(mV)*		*(ohm-cm)*	
2	2	0.01	8.6	-93	Trace	16,644	Silty SAND
P-1	2	<0.01	8.3	-76	Negative	16,046	Silty SAND

Job No. 15-2013

APPENDIX A

Pavement Section Calculations

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare
Computer Software Product
Network Administrator

Flexible Structural Design Module

Job No. 15-2013
Light Vehicle Parking
Full Depth Asphalt

Flexible Structural Design

18-kip ESALs Over Initial Performance Period	21,900
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability Level	90 %
Overall Standard Deviation	0.44
Roadbed Soil Resilient Modulus	4,195 psi
Stage Construction	1
Calculated Design Structural Number	2.27 in

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(in)	Width (ft)	Calculated SN (in)
1	Asphalt	0.4	1	6	-	2.40
Total	-	-	-	6.00	-	2.40

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare
Computer Software Product
Network Administrator

Flexible Structural Design Module

Job No. 15-3022013
Light Vehicle Parking
Composite Section

Flexible Structural Design

18-kip ESALs Over Initial Performance Period	21,900
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability Level	90 %
Overall Standard Deviation	0.44
Roadbed Soil Resilient Modulus	4,195 psi
Stage Construction	1
Calculated Design Structural Number	2.27 in

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(in)	Width (ft)	Calculated SN (in)
1	Asphalt	0.4	1	4	-	1.60
2	Aggregate Base Course	0.12	1	6	-	0.72
Total	-	-	-	10.00	-	2.32

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare
Computer Software Product
Network Administrator

Rigid Structural Design Module

Job No. 15-2013
Fire Truck Routes
Concrete Section

Rigid Structural Design

Pavement Type	JPCP
18-kip ESALs Over Initial Performance Period	365,000
Initial Serviceability	4.5
Terminal Serviceability	2.5
28-day Mean PCC Modulus of Rupture	650 psi
28-day Mean Elastic Modulus of Slab	3,400,000 psi
Mean Effective k-value	14 psi/in
Reliability Level	90 %
Overall Standard Deviation	0.34
Load Transfer Coefficient, J	3.6
Overall Drainage Coefficient, Cd	1
Calculated Design Thickness	6.86 in