GROUND ENGINEERING

Geotechnical Subsurface Exploration Program
Mapleton Hills Development
Boulder, Colorado

Prepared for:

Rubicon Development
1035 Pearl Street, # 205
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Attention: Mr. Michael Bosma

Job Number: 16-0011 **September 21, 2016**

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PURPOSE AND SCOPE OF STUDY

This report presents the results of a subsurface exploration program performed by GROUND Engineering Consultants, Inc. (GROUND) for the proposed construction of the Mapleton Hills development located near the intersection of Mapleton Avenue and 4th Street in Boulder, Colorado. This study was conducted in general accordance with GROUND's proposal number 1603-0353, dated March 7, 2016

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

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

It should be noted that environmental consulting was not part of GROUND's scope of services for this project. Rubicon Development should retain an environmental consultant as appropriate to provide services such as identification of hazardous materials that may be present, preparation of a materials management plan, etc.

PROPOSED CONSTRUCTION

We understand that proposed construction will consist of an assisted living development consisting of ten structures. Several of the structures are planned to have below-grade levels for parking. Development of the site will include significant grading including large cuts into the hillside slopes on the west of the site. Structure loads for the proposed buildings were unavailable at the time of this report preparation but are assumed to be moderate. Additionally, paved surface parking areas, internal private drives, and underground utilities are also planned. Two separate retaining walls are also planned to be constructed at the west of Building B and northeast of the Wellness Center complex.

According to the provided Site Review Submittal, dated August 1, 2016, the following table provides the approximate existing and final grades for the proposed buildings with their approximate cut/fill depths. Please refer to Figure 1 for the Client-assigned building number and location of each building. These cut/fill depths should be re-evaluated once final grading plans are developed.

Building	Approximate Lowest Level Floor Elevation (feet)	Approximate Maximum Cut/Fill * (feet)
A and A East(shared garage)	5500	-5 to -35
A West	5512	-8
B	5514	-4 to -41
C	5512	+2 to -23
C Basement	5500	-10 to -20
D	5500	+5 to -10
E Basement (east half)	5500	-0 to -12
E (west half)	5512	-0 to -15
F	5500	-2 to -10
G(existing power building)	5500	+3 to -3
H and J(shared garage)	5492	-4 to -11
I	5489	-3 to -9
K	5510	+1
M	5540	-1 to -13
Cottage 1	5494.5	+2
Cottage 2	5493.5	+2
Cottage 3	5492.0	+0
Cottage 4	5491.0	+1
Cottage 5	5489.0	+4
Cottage 6	5488	-2
Cottage 7	5487	+2
Cottage 8	5503	+7
Cottage 9	5555	+2
Cottage 10	5553	+1

* based on data and existing grades at the time of our study.

If proposed construction, including the anticipated site grading, differs from that described above, or changes subsequently, GROUND should be notified to re-evaluate the information in this report. Specific tenant requirements for corporate facilities were not provided for our review. Any requirements should be provided which may result in modifications to the parameters provided herein.

SITE CONDITIONS

At the time of our subsurface exploration, the project area supported several structures including a former hospital, a power/boiler building which served the hospital. Three structures on the order of 1,000 square feet in footprint area were located to the immediate west of the power building. Two garage buildings were noted to the northwest of the power building.

A large portion of the site was in use as paved parking and drive lanes. There was a utility tunnel starting at the south west portion of the power building and traveling southwest to the hospital. Additionally, underground utilities were located throughout the site.

Topographically, the project site falls steeply from the west to the east at varying slopes ranging from in excess of 40 percent near the west portion of the site to less than 5 percent on the east portion of the site.

The project site is bordered by 4th Street to the east, a residential development to the northeast, Mapleton Avenue to the south and open space to the west and northwest.

GEOLOGIC SETTING

Published geologic maps, e.g., Colton (1976)[1] depict the site as underlain largely by deposits of Pleistocene to Holocene Colluvium (**Qc**), Verdos Alluvium (**Qv**) and Louviers Alluvium (**Qlo**). A portion of that map showing the site and its vicinity is provided below.

The surficial units are depicted as underlain by a series of lower to upper Cretaceous formations including the Dakota Sandstones (**Kd**) the Carlisle Shale, Greenhorn Limestone and Graneros Shale (**Kcg**) and the Pierre Shale (lower and middle members, **Kpl** and **Kpm**).

Alluvial (stream-laid) deposits in the area such as the Louviers and Verdos Alluviums consist of sands, gravels, cobbles and boulders. The local colluvium ("slope wash" deposits) typically range from sands to clays, depending on the source rock.

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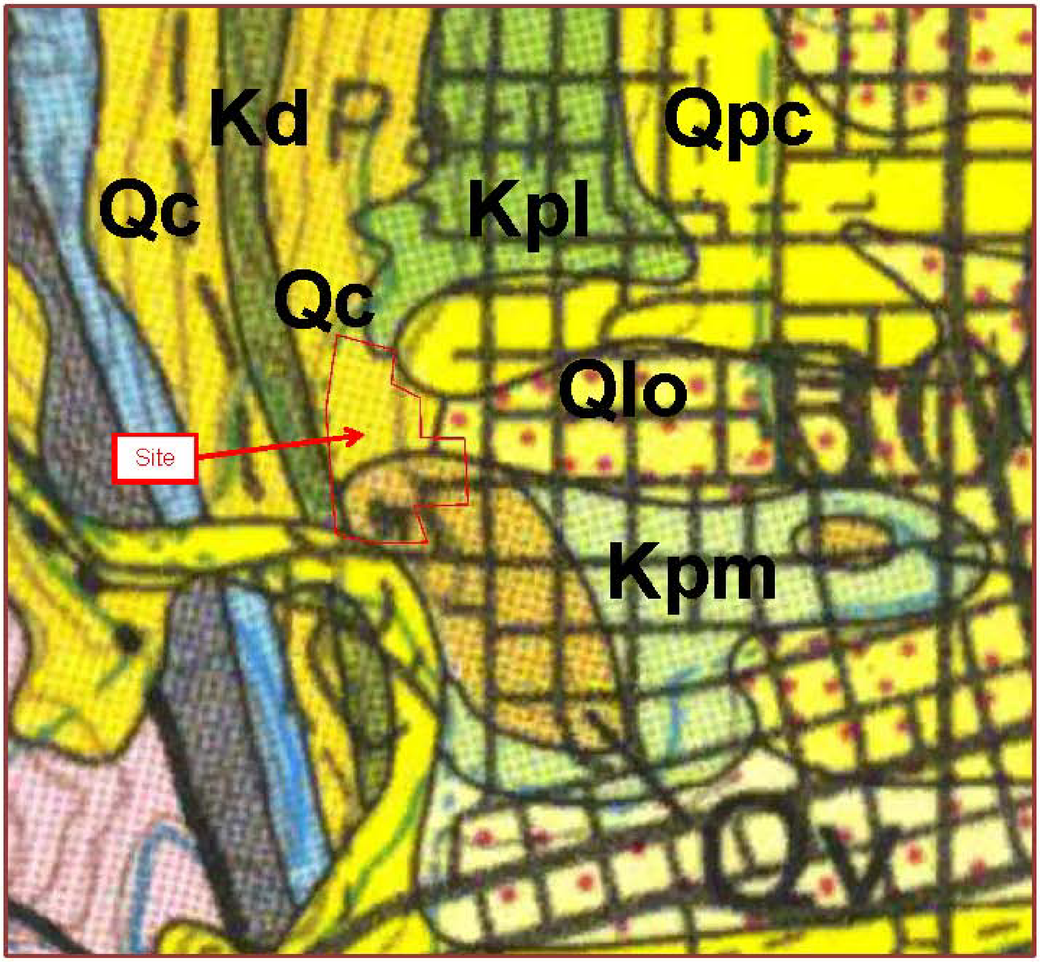
[1] Colton, R.B., 1976, *Geologic Map of Boulder-Fort Collins-Greeley Area, Colorado*, U.S. Geological Survey, Miscellaneous Investigations Series I-855-G.



The Dakota Sandstone is a hard, relatively resistant unit that supports the ridge on the western margin of the site. The other formations are primarily shale with subordinate limestones and sandstones that generally are less resistant to erosion, but still over-consolidated and dense. The shales commonly are moderately to highly expansive. Locally the sandstone beds also can be resistant to excavation.

The bedding (layering) of the bedrock formations dips (tilts) to the east-northeast at the site and in the surrounding area. In the western quarter of the site the beds dip steeply with dip angles approaching vertical. Eastward across the site, the beds dip progressively less steeply.

SUBSURFACE EXPLORATION

The subsurface exploration for the project was conducted in April, 2016. A total of twenty-eight (28) test holes were drilled with a truck/track-mounted, continuous flight power auger rig to evaluate the subsurface conditions as well as to retrieve soil and bedrock samples for laboratory testing and analysis. Of these, eighteen (18) test holes were drilled within the proposed building footprint limits, eight (8) test holes were drilled within/near the preliminary proposed retaining wall locations, and the remaining two (2) test holes were drilled within the proposed pavement areas. Four proposed test holes (B-1, B-2, B-8 and B-9) were not drilled because an area in the eastern portion of the project site was removed from our scope. The foundation/wall test holes were drilled to depths ranging from approximately 15 to 40 feet below existing grade and the pavement test holes were drilled to depths ranging from approximately 9 to 10 feet below existing grades. A GROUND engineer directed the subsurface exploration, logged the test holes in the field, and prepared the soil and bedrock samples for transport to our laboratory.

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

The approximate locations of the test holes are shown in Figure 1. Logs of the exploratory test holes are presented in Figures 2 through 7. Explanatory notes and a legend are provided in Figures 8 and 9. The test hole locations were professionally surveyed by others for location and ground surface elevation at the time of drilling.

LABORATORY TESTING

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

SUBSURFACE CONDITIONS

Beneath the asphalt paving, 2 to 10 inches[2] in thickness, or topsoil[3], 3 to 6 inches[2] in thickness, the test holes generally consisted of fill, native sands and gravels, and native clays. These materials extended to depths of about 2 to 34 feet below existing grades. We interpret the native soils to be colluvial ("slope wash") soils interbedded with alluvial (stream-laid) deposits of the Verdos and/or Louviers Alluviums.

The soils were underlain by bedrock consisting of clay shales (with locally interbedded sandstones) and limestones that extended to the depths explored.

We interpret the shales and local sandstones to be materials of the lower member of the Pierre Shale, the Niobrara Shale and, in the westernmost portion of the site, the Carlisle Shale and/or shale beds within the Greenhorn Limestone. These dominantly shale formations are not differentiated on our logs or in this text.

We interpret the limestones to be Greenhorn Limestone. This unit crops out locally near the western margin of the site and was encountered at various depths in several of the test holes.

The dip of the bedrock units beneath the site result in different bedrock units underlying the surficial soils across the site. The variation in dip angles both west to east and north to south across the site make forecasting materials and depths with precision difficult. Interpreted, generally west to east, cross sections providing an overall sense of this bedrock geometry are provided in Figures 10 – 14. The cross sections should not be relied upon, however, to provide precise depths to a given material type, etc.

Fill soils were identified in some of the test holes and are likely present elsewhere on site. Delineation of the complete lateral and vertical extents of the fills at the site, or their

[2] Note that these thicknesses are approximate; thicknesses of these materials are difficult to estimate in small diameter test holes.
[3] 'Topsoil' as used herein is defined geotechnically. The materials so described may or may not be suitable for landscaping or as a growth medium for such plantings as may be proposed for the project.

compositions, was beyond our present scope of services. If fill soil volumes and compositions at the site are of significance, they should be evaluated using test pits.

It should be noted that coarse gravel, cobbles, boulders, and similarly sized fragments of debris 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.

Fill consisted of fine to coarse sands and gravels with clay and scattered construction debris. They were non- to medium plastic, loose to medium dense, slightly moist to wet, and brown to black in color.

Sandy Clays were dry to moist, low to medium plastic, stiff to very stiff, and light brown to red-brown in color, with local caliche and iron oxide staining. The sand fractions were fine to medium.

Sands and Gravels were fine to coarse with local clay, low to medium plastic, medium dense to very dense, dry to wet, and brown to red- brown in color. Caliche was noted locally, as was iron oxide staining.

Weathered Clay Shale was medium to highly plastic, weathered, slightly moist, and pale brown to gray in color. Iron oxide staining was common.

Clay Shale was medium to highly plastic, hard to very hard, dry to slightly moist, and pale brown to gray to black in color. Iron oxide staining was noted locally. Sandstone beds and lenses were present locally, as well.

Weathered Sandstone was fine grained, clayey, low to medium plastic, moderately hard, moist, and gray in color. Iron oxide staining was noted commonly.

Sandstone was fine grained, clayey, low to medium plastic, very hard, moist, and gray in color. Iron oxide staining was noted commonly.

Limestone was finely crystalline, thinly to moderately bedded, very hard, slightly moist, and white to pale yellow to pale green in color. Shale beds were present locally.

The limestone was very resistant and recovery from the samplers driven into it (on which the description above was based) generally was very poor.

Groundwater was encountered in some of the test holes at depths ranging from approximately 10 to 29 feet (elevations ranging from 5,476.1 to 5,528.8 feet) below existing grades at the time of drilling and at depths ranging from approximately 3.9 to 23.6 feet (elevations ranging from 5,532.2 to 5,481.5 feet) below existing grades in several of the test holes (B-17, B-22, W-4, W-6, W-7, B-10) when measured approximately 37 to 38 days following drilling. Groundwater levels can be expected to fluctuate, however, in response to annual and longer-term cycles of precipitation, irrigation, surface drainage, nearby rivers and creeks, land use, and the development of transient, perched water conditions.

Swell-Consolidation Testing of samples of the on-site materials encountered in the project test holes indicated a potential for both heave and consolidation. (See Table 1.) Swells ranging from approximately 0.4 to 5.1 percent and consolidations ranging from approximately 0.6 to 2.1 percent were measured at various surcharge loads.

SEISMIC CLASSIFICATION

Based on extrapolation of available data to depth and our experience in the project area, we consider the site likely to meet the criteria for a Seismic Site Classification of **C** according to the 2015 IBC classification (Table 1613.5.2) except in areas of relatively deep fills such as near Test Hole W-6 where a Site Classification of **D** is indicated. If, however, a quantitative assessment of the site seismic properties is desired, then sampling or shear wave velocity testing to a depth of 100 feet or more should be performed.

Utilizing the United States Geological Survey's Seismic Design Maps Tool (http://geohazards.usgs.gov/designmaps/us/application.php), the project area is indicated to possess an S_{DS} and S_{DI} values tabulated below.

Site Classification	S_{DS}	S_{DI}
C	**0.191 g**	**0.067 g**
D	**0.255 g**	**0.094 g**

RADON

Radon is a naturally occurring, colorless, odorless, radioactive gas that can cause lung cancer according to the U.S. Environmental Protection Agency (EPA). Radon collected in an enclosed structure, therefore, can represent a potential hazard. Radon accumulation is not a hazard that can be mitigated by geotechnical measures, however, and testing for the possible presence of radon gas prior to project construction does not yield useful results regarding the potential for future accumulations.

Radon collects typically in basements, crawl spaces or other enclosed portions of buildings constructed in areas underlain at relatively shallow depths by granitic crystalline and/or gneissic bedrock. The likelihood of encountering radon in concentrations exceeding applicable health standards on the subject site, underlain by sedimentary bedrock, is significantly lower. It cannot be excluded, however. 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 local building and/or health departments.

Radon testing should be performed in the proposed buildings, after construction is completed. However, we understand that incorporating sufficient ventilation and other measures into a structure to address radon accumulation during construction typically is significantly less costly than installing them after construction has been completed. Therefore, the architect should consider the potential for radon accumulation in the proposed buildings and incorporate mitigative measures into the design, as appropriate.

GEOTECHNICAL CONSIDERATIONS FOR DESIGN

Geotechnical Risk Variable geotechnical constraints on design and construction were recognized across the roughly 11-acre site. These include the following:

- Where underlying the proposed buildings at shallow to moderate depths, the shales comprising the bulk of the bedrock were significantly expansive. We estimate that a structure bearing directly on the shales at the site likely would experience post-construction vertical movements on the order of 5 inches. Lateral movements will be realized, as well. Movements of this magnitude can damage both the proposed buildings and nearly all other improvements.

- The relief across the site, together with the proposed grading – entailing cuts up to approximately 41 feet in depth and fills up to approximately 7 feet – will yield strongly differential support conditions unless mitigated.

  Prior grading at the site also resulted in significant depths of existing, undocumented fill soils that were as deep as about 14 feet in some test holes. All of the un-documented fill soils should be excavated and replaced as properly compacted fill.

  Eight existing structures on the site are planned for removal or relocation. This also will result in additional fills – that should be replaced with properly compacted fill – and that likely will include construction debris. Although outside the scope of our services, the possible presence of asbestos or other hazardous materials in the buildings to be demolished should be considered.

  Together, the presence of several generations of fill, likely of strongly differential thickness at least across some buildings, will result in likely total and differential settlement of shallow foundations and slab-on-grade floors of 2 to 5 inches which also will be damaging.

- The limestone underlying the western portion of the site at shallow depths (at greater depths farther to the east) was very hard and resistant, as were local beds of sandstone within the shales. Although these materials provide good bearing support, in general, additional efforts by the contractor will be needed to excavate them or advance drilled piers through them.

The conclusions and parameters provided in this report were based on the data presented herein, our experience in the general project area with similar structures, and our engineering judgment with regard to the applicability of the data and methods of forecasting future performance. A variety of engineering parameters were considered as indicators of potential future soil movements. Our parameters and conclusions were based on our judgment of "likely movement potentials," (i.e., the amount of movement likely to be realized if site drainage is generally effective, estimated to a reasonable degree of engineering certainty) as well as our assumptions about the owner's willingness to accept geotechnical risk. "Maximum possible" movement estimates necessarily will be larger than those presented herein. They also have a significantly lower likelihood of being realized in our opinion, and generally require more expensive measures to address. We encourage Rubicon Development, upon receipt of this report, to discuss these risks and the geotechnical alternatives with us.

General Foundation and Floor Types

Deep Foundation Systems and Structural Floors In GROUND's opinion, supporting the proposed buildings on deep foundations will provide the lowest estimates of likely post-construction foundation movement (about ½ inch, with similar differential movements over spans of about 40 feet) and the least risk of excessive foundation movements. Because of the proximity of other structures to the project, drilled piers appear to be the best deep foundation system option. Geotechnical parameters for design and installation of a drilled pier foundation are provided in the *Drilled Pier Foundation System* section of this report.

Constructing the lowest level floor of a building as a structural floor, also supported on drilled piers, will yield similarly low post-construction floor movement estimates. Geotechnical parameters for structural floors are provided in the *Floor Systems* section of this report.

Because of the proposed use and the presence of expansive clay shales and relatively shallow depths beneath the proposed lowest floor elevations (and, commonly, the presence of an existing building to be demolished) the following buildings should be supported on drilled piers and provided with structural floors also supported on drilled piers:

Drilled Pier Foundations and Structural Floors

Building A	**Building E**	**Building M**
Building A West	**Building H**	**Cottage 9**
Building B	**Building I**	**Cottage 10**
Building C	**Building J**	

Where the lowest level of a building will be constructed as a parking garage, and post-construction floor movements up to about 5 inches and the associated maintenance are acceptable, that floor may be constructed as a concrete slab-on-grade. Detailed parameters slab-on-grade floor design and construction are provided in the *Slab-on-Grade Floors* sections of this report.

Shallow Foundations The existing 3-story, brick building in the western portion of the site (345 Maxwell Ave.) appeared to supported on shallow foundations bearing on Greenhorn Limestone. This building appeared to be performing well, considering its age. The smaller, single-story stone building to the south (331 Maxwell Ave.) also may be supported in the same manner. No proposed buildings appear to be planned where shallow foundations could be anticipated to bear only on the limestone, however. Instead, they appear to straddle between the limestone and potentially expansive clay shales.

Other proposed building locations at the site, however, are underlain by significant depths of native sands and gravels, commonly 12 or more feet in thickness. At those locations, buildings may be supported on shallow foundations and provided with slab-on-grade concrete floors with unusual risk of post-construction movements. If the measures outlined in this report are implemented effectively, then we estimate likely post-construction foundation and floor movements to be about 1 inch, with differential movements of about ½ inch over spans of about 40 feet.

The following buildings may be supported on shallow, spread footing foundations and provided with slab-on-grade floors bearing on firm, native soils:

Shallow Foundations and Slab-on-Grade Floors bearing on Firm, Native Soils

Cottages 1 – 8

Detailed parameters for shallow foundation and slab-on-grade floor design and construction are provided in the *Shallow Foundations* and *Slab-on-Grade Floors* sections of this report.

(The above buildings also may be supported on drilled piers and/or provided with a structural floor also supported on drilled piers. If this is done, then likely post-construction movements will be about ½ inch with similar differential movements over spans of about 40 feet.)

If existing, un-documented fill is exposed under one of the above buildings, it should be removed in accordance with the comments below.

The following buildings may be supported on shallow, spread footing foundations and provided with slab-on-grade floors after removal of debris from demolition of existing structures and construction of a uniform section of properly compacted fill <u>at least equal in depth to the depth to which demolition was taken</u>:

Shallow Foundations and Slab-on-Grade Floors bearing on a Remedial Fill Section

Building A East	**Building F**	**Building K**
Building D *	**Building G**	

* Note that for Building D, it is not construction debris and relic foundations, etc., but <u>all the existing, un-documented fill soils</u> that must be excavated and replaced with properly compacted fill. Therefore, at Building D, the section of properly compacted fill must extend to a depth of at least 15 feet.

<u>In all cases, the maximum depth of excavation to remove construction debris, relic foundation elements, etc., beneath a building should be the minimum depth of the section of properly compacted fill beneath that building</u>. The fill section should extend laterally beyond the building at full depth a distance equal to the depth of the fill section beneath the footings. Detailed parameters for shallow foundation and slab-on-grade floor design and construction are provided in the *Shallow Foundations* and *Slab-on-Grade Floors* sections of this report.

(The above buildings also may be supported on drilled piers and/or provided with a structural floor also supported on drilled piers. If this is done, then likely post-construction movements will be about ½ inch with similar differential movements over spans of about 40 feet.)

Detailed parameters for shallow foundation and slab-on-grade floor design and construction are provided in the *Shallow Foundations* and *Slab-on-Grade Floors* sections of this report.

Other Considerations In all cases where fill underlies a building, earthwork should be undertaken so that a fill section of uniform depth and laterally uniform fill materials is constructed to reduce differential movements.

Groundwater was encountered at variable depths across the site. Because of the complex stratigraphy, and prior and proposed cut/fill grading, groundwater including perched groundwater should be anticipated in excavations of almost any depth, anywhere across the site.

Cobbles and boulders in the shallow soils, hard limestones, sandstones and cemented shales may make drilled pier installation locally more difficult than is typical in the area. Construction debris left on site may cause similar difficulties.

'Slope creep' is the slow, broad, downslope movement of surficial soils due to freeze/thaw cycling, variations in soil moisture content, local soil disturbance, etc. In our experience, slope creep typically involves approximately the top 4 feet of soil on a slope. The local slopes, both natural and graded, should be anticipated to exhibit creep over time. Sidewalk stones, fencing, light poles and other improvements supported at shallow depths on slopes and near the tops-of-slope will be displaced over time.

Detailed stability analyses of project slopes was beyond GROUND's present scope of services. Proposals for analyses of specific slopes can be provided upon request.

DRILLED PIER FOUNDATIONS

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

1) Drilled piers should bear in 'relatively un-weathered' bedrock underlying the site. For design purposes, the depth to 'relatively un-weathered' bedrock may be obtained from the lowest 'top-of-bedrock' elevation shown on the logs of nearby test holes. (GROUND can provide assistance in this regard.) For bidding purposes, these elevations may vary.

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

3) Drilled piers should have a minimum length of **30 feet**. The actual drilled pier lengths should be determined by the structural engineer based on loading, etc.,

with further increases in length possibly required by the conditions encountered during installation at each drilled pier location.

4) Drilled piers also should penetrate at least **10 feet** into relatively unweathered bedrock or 3 drilled pier diameters, whichever is greater.

Actual drilled pier lengths commonly will be greater than the 30-foot minimum due to structural considerations, conditions in the drilled pier holes, actual depths to relatively un-weathered bedrock depths, etc.

5) Drilled piers bearing in comparatively un-weathered bedrock may be designed for an allowable end bearing pressure of **35,000 psf**.

The portion of the drilled pier penetrating comparatively unweathered bedrock may be designed for an allowable skin friction value of **2,625 psf** for the portion of the pier penetrating relatively un-weathered bedrock. 100 percent of the skin friction may be used to resist both compressional loads and uplift.

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

7) Drilled piers should be designed for a minimum dead load pressure of **7,000 psf** based on drilled pier cross-section area.

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

8) 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 swelling soils and bedrock. Tension may be estimated as an uplift skin friction of **1,400 psf** applied to the **upper 20 feet** of the drilled pier.

Reinforcement design also should include any deficit between the dead load applied in design and the minimum dead load provided above.

9) 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.

10) Geotechnical parameters for resisting lateral loading of drilled piers are provided in the *Lateral Loads* section of this report.

11) 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 and below a depth of 20 feet, from top of pier. 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.

12) Groups of closely spaced drilled piers placed to support concentrated loads will require an appropriate reduction of the estimated capacities. Reduction of axial capacity generally can be avoided by spacing drilled piers at least 3 diameters center to center. At this spacing or greater, no reduction in axial capacities or horizontal soil modulus values is required. The capacities of drilled piers spaced more closely than 3 diameters center to center should be reduced. Reduction factors can be obtained from Figure 15.

13) 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 16.

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

14) The depth of comparatively unweathered bedrock should be determined in the field at each drilled pier location and may differ from other information provided herein.

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

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

    Boulders, cobbles and coarse construction debris may be encountered in soils penetrated by the drilled pier holes, making drilling more difficult. The contractor should anticipate these conditions.

    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.

17) Groundwater was encountered in some of the test holes at depths ranging from approximately 10 to 29 feet below existing grades (elevations ranging from about 5,476 to 5,529 feet) at the time of drilling and at depths ranging from approximately 4 to 23½ feet (elevations from 5,482 to 5,532 feet) in several of the test holes (B-17, B-22, W-4, W-6, W-7, B-10) when re-measured approximately 37 to 38 days following drilling. Drilled piers likely will be encountered in the drilled pier holes.

    Groundwater where with granular soils, often results in caving during pier installation.

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

18) 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.

19) 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.

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

21) 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**.

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

23) 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.

24) 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.

25) Sonic integrity testing (sonic echo or cross-hole sonic) should be considered to be performed for an appropriate percentage of the drilled piers (e.g., 10 percent, at least initially) to assess the effectiveness of the drilled pier construction methods. Additional information on sonic integrity testing can be provided upon request.

SHALLOW FOUNDATIONS

The geotechnical parameters indicated below may be used for design of shallow, spread footing foundations for the buildings so indicated in the *Geotechnical Considerations for Design* section of this report.

Geotechnical Parameters for Shallow Foundation Design

1) Footings should bear on firm native soils or a section of properly compacted fill fill prism consisting of properly compacted as outlined in the *Geotechnical Considerations for Design* section of this report.

   The fill section should extend laterally at full depth across and beyond the building perimeter as described in the *Geotechnical Considerations for Design* section of this report.

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

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

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

2) Footings bearing on firm, native soils or a section of properly compacted fill may be designed for an allowable soil bearing pressure of **2,000 psf** for footings up to 8 feet in width. For larger footings, a lower allowable bearing pressure may be appropriate.

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

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

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

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

4) Footings/structures (slabs) should bear at an elevation **3 or more feet** below the lowest adjacent exterior finish grades to have adequate soil cover for frost protection. Interior footings in heated areas not subject to frost heave may bear at **1½ feet or more** below adjacent grades.

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

6) Geotechnical parameters for lateral resistance to foundation loads are provided in the *Lateral Earth Pressure* section of this report.

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

Shallow Foundation Construction

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

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

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

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

12) In areas where shallow groundwater is encountered, footing excavations may expose wet soils. In those excavations, a layer of lean concrete, coarse gravel, stabilization geo-textile or other means of stabilization should be used to reduce disturbance of the natural soils caused by construction operations. Disturbing the native soils will increase potential settlements.

13) Foundation elevations may be slightly above local groundwater levels. Therefore, it may be necessary to de-water some footing excavations during construction. De-watering should not be conducted by pumping from inside footing excavations. This may decrease the supporting capacity of the soils.

14) All foundation subgrades should be properly compacted prior to placement of concrete.

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

STRUCTURAL FLOORS

A structural floor should be supported on grade beams and drilled piers. Requirements for the number and position of piers to support the floor should be developed by the structural engineer. Geotechnical parameters for design and installation of drilled piers are provided in the *Drilled Pier Foundations* section of this report. Separate, entryway floor slabs and similar areas also should be constructed as structural floors to reduce the potential for differential movements between such structures and the building.

It should be noted that structural floors tend to be flexible and move elastically under live loads. Building design also should account for floor movements of this type.

Structural floors should be constructed either to span above a well-ventilated crawl space. Floor system design should be coordinated with any radon-mitigations systems, etc.

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 close proximity of finished building pad grades. Persistently warm, humid conditions in the presence of cellulose, which is the base material found in many typical construction products, creates an ideal environment for the growth of fungi, molds, and mildew. Published data suggest links between molds and negative health effects. Therefore, GROUND recommends that crawl spaces beneath structural floors be provided with adequate, positive <u>active</u> ventilation systems or other active mechanisms such as specially designed HVAC systems (as well as properly constructed and maintained underdrains) 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.

If utility lines are placed in the crawl space above the soil, the crawl space should be adequate to allow access to and maintenance of the utility piping. Utility lines can be displaced by soil movements which are not reflected in the building. Design and installation of associated fixtures should accommodate this potential differential movement, which could be on the order of 5 inches.

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 recommended. 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 the necessary tensile strength, gets brittle, tends to decompose over time, and has a relatively high permeance.

Crawl spaces typically are vulnerable to moisture accumulation. Proper installation and maintenance of the underdrains, as outlined in the *Subsurface Drainage* section of this report, will assist drainage of free water and assist the ventilation system to reduce crawl space moisture.

SLAB-ON-GRADE FLOORS

The geotechnical parameters below may be used for design of slab-on-grade floors for the buildings so indicated in the *Geotechnical Considerations for Design* section of this report. ACI Sections 301/302/360 provide guidance regarding concrete slab-on-grade design and construction.

Geotechnical Parameters for Slab-on-Grade Floors

16) A slab-on-grade floor should bear on either firm, native soils or a section of properly compacted fill as outlined in the *Geotechnical Considerations for Design* section of this report.

    The remedial fill section should extend laterally at full depth across and beyond the building perimeter as described in the *Geotechnical Considerations for Design* section of this report.

The thickness of a remedial fill section should be taken from the bottom of the slab + gravel layer system of that building. (If the gravel layer is not installed, the remedial fill section should be correspondingly thickened.)

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

17) 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.

18) An allowable vertical modulus of subgrade reaction (Kv) of **75 tcf** (86.8 pci) may be used for design of a concrete, slab-on-grade floor bearing on the recommended section of properly compacted fill.

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

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

Slip joints should be observed periodically, particularly during the first several years after construction. Slab movement can cause previously free-slipping joints to bind. Measures should be taken to assure that slab isolation is maintained in order to reduce the likelihood of damage to walls and other interior improvements.

20) 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.

21) 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.

22) Moisture can be introduced into a slab subgrade during construction and additional moisture will be released from the slab concrete as it cures. It can be beneficial to place a properly compacted layer of free-draining gravel, **4 or more inches** in thickness, 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.

23) 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. ACI and other industry groups provide guidance regarding the use of vapor barriers.

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.

Construction Considerations for Slab-on-Grade Floors

24) 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.

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

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

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

FOUNDATION WALLS

Wall Design Parameters Equivalent fluid pressures for use in design of foundation walls are provided in the *Lateral Loads* section of this report.

If select, granular fill is placed as foundation wall backfill, then the select, granular fill should be placed behind the wall to a minimum distance equal or greater than half of the wall height. In such cases, a relatively low permeability soil (rather than the select, granular soil) should comprise the upper **1 foot** of the wall backfill to reduce infiltration into the backfill or other measures taken to reduce surface water infiltration. The local clayey soils and excavated clay shale are suitable, in general for this purpose.

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

Parameter for underdrains and wall drainage are provided in the *Subsurface Drainage* section of this report.

Foundation Wall Construction Considerations Wall backfill soils should be compacted properly, but the contractor should take care not to over-compact the backfills because excessive lateral pressures on the walls could result.

Some settlement of wall backfill will occur even where the material was placed and compacted correctly. This settlement likely will be differential, increasing with depth of fill.

Where shallowly founded structures or pavements are be placed on backfilled zones, the associated risks should be understood by the owner. Structural design, pipe connections, etc., should take into account (differential) foundation wall backfill settlements. A geotechnical engineer should be retained to provide design parameters where improvements are placed in backfilled areas.

MECHANICAL ROOMS / MECHANICAL PADS

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

RETAINING WALLS

Because of the presence of expansive clay shales and relatively deep, un-documented fills, rigid types of retaining walls (e.g., cast-in-place concrete walls) should be supported on drilled pier foundations. Geotechnical parameters for drilled pier design and construction are provided in the *Drilled Pier Foundations* section of this report.

Alternatively, flexible-type retaining walls, such as "mechanically stabilized earth" (MSE) walls, are able to accommodate significant strains with minimal adverse effects to the walls. Because of the flexibility of the wall system, an MSE wall can be supported at shallow depths.

Wall Design Parameters Equivalent fluid pressures for use in design of foundation walls are provided in the *Lateral Loads* section of this report.

MSE retaining walls bearing on firm native soils or properly compacted fill may be designed for an allowable soil bearing pressure of **2,000 psf**. Where wall excavation bottoms expose soft, loose or otherwise deleterious materials, such materials should be excavated and replaced with properly compacted fill.

For estimation purposes, the length of the geotextile reinforcing zone of an MSE wall can be taken as 0.7 to 0.8 times the wall height, but will depend on the final design as provided others.

MSE walls should bear at least 3 feet below lowest adjacent grade to provide adequate soil cover above the bearing elevation if frost protection is a design consideration.

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

Wall drainage provisions should be developed by the retaining wall designer.

Retaining Wall Construction Considerations Wall backfill soils should be compacted properly, but the contractor should take care not to over-compact the backfills because excessive lateral pressures on the walls could result.

Some settlement of wall backfill will occur even where the material was placed and compacted correctly. This settlement likely will be differential, increasing with depth of fill.

Where shallowly founded structures or pavements are be placed on backfilled zones, the associated risks should be understood by the owner. Structural design, pipe connections, etc., should take into account (differential) foundation wall backfill settlements. A geotechnical engineer should be retained to provide design parameters where improvements are placed in backfilled areas.

LATERAL EARTH PRESSURES

Shallow Foundations Resisting Lateral Loads Footings and similar elements designed for frictional resistance to lateral loads may be designed using a friction coefficient between the foundation element and the local soils or bedrock of **0.27**.

Passive soil pressure may be calculated at this site using an equivalent fluid pressure of **270 pcf** for drained conditions, to a **maximum of 2,700 psf**. The upper **1 foot** of embedment should be neglected for passive resistance, however. Where this passive soil pressure is used to resist lateral loads, it should be understood that significant lateral strains will be required to mobilize the full value indicated above, likely 1 inch or more. A reduced passive pressure can be used for reduced anticipated strains, however.

Active and At-Rest Lateral Earth Pressures Retaining structures such as foundation walls which are laterally supported can be expected to undergo only a limited amount of deflection, i.e., an **at-rest** condition, should be designed to resist lateral earth pressures calculated on the basis of an equivalent fluid unit weight of **79 pcf** where on-site materials are placed as backfill. (Where CDOT Class 1 Structure Backfill is placed as backfill behind a retaining structure, an at-rest equivalent fluid pressure of **59 pcf** may be used.)

Retaining walls designed to deflect sufficiently to mobilize the full, **active** earth pressure condition may be designed for an active lateral earth pressure calculated on the basis of an equivalent fluid unit weight of **58 pcf** where the backfill consists of on-site materials. (Where CDOT Class 1 Structure Backfill is placed as backfill behind a retaining structure, an active equivalent fluid pressure of **38 pcf** may be used.)

Other Considerations Note that the values provided above were based on a moist unit weight (γ') of 126 pcf and an angle of internal friction (ϕ) of 22 degrees (132 pcf and 34 degrees, respectively for CDOT Class 1 Structure Backfill) and are un-factored. Appropriate factors of safety should be included in design calculations.

To utilize these values for Class 1 Structure Backfill, the select, granular fill section should be placed to a minimum distance behind the wall equal or greater than half of the wall height. The upper 1 foot of the wall backfill, however, should be a relatively impermeable soil (such as the on-site clays) or otherwise protected to reduce surface water infiltration into the granular backfill.

The active and at-rest earth pressures indicated above assume well drained conditions behind the wall and a horizontal backfill surface. The contractor should evaluate/verify the values for the materials actually placed as wall backfill. Wall design should incorporate any upward sloping backfills, live loads such as construction equipment, material stockpiles, etc., and other surcharge pressures. The build-up of hydrostatic pressures behind a wall also will increase lateral earth pressures on the walls.

If additional values are necessary, such as for soils in a submerged condition, GROUND can provide them upon request.

Global stability analysis may be needed after retaining wall design. GROUND can provide a proposal for global stability analyses upon request.

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. Resistance to lateral loads by deep foundations should be neglected in the **upper 3 feet** of soils, whether fill or native.

HORIZONTAL MODULUS SUBGRADE REACTION (K_h) – TERZAGHI METHOD

Material	K_h based on Foundation Element Width / Diameter	
	18-inch	24-inch
Soils and Weathered Bedrock	56 tcf (64.8 pci)	42 tcf (48.6 pci)
Bedrock	300 tcf (347 pci)	225 tcf (260 pci)

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

If “L-Pile” or a similar computer program is used for lateral analysis of the piles, recommended geotechnical parameters for input into that program are tabulated below for the same simplified soil / bedrock profile. These include, unit wet weights (γ'), angles of internal friction (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, resistance to lateral loads should be neglected in the upper 3 feet of soils, whether fill or native.

Note that below the water table, if present, the unit weights must be adjusted for buoyancy by reducing the values by 62.4 pcf.

GEOTECHNICAL PARAMETERS FOR LATERAL LOAD ANALYSIS USING L-PILE

Soil / Bedrock Material	Parameter	Recommended Value
Soils and Weathered Bedrock (model as Sand)	γ'	123 pcf (0.0712 pci)
	ϕ	28 degrees
	k	67.4 tcf (78 pci)
Bedrock (model as Stiff Clay without Free Water)	γ'	126 pcf (0.0729 pci)
	c	2,000 psf (13.9 psi)
	ε_{50}	0.010

Again, resistance to lateral loads by deep foundations should be neglected in the upper 3 feet of soils, whether fill or native.

WATER-SOLUBLE SULFATES

The concentrations of water-soluble sulfates measured in selected samples retrieved from the test holes ranged from approximately less than 0.01 to 0.06 percent by weight (See Table 2). Such concentration of water-soluble sulfates represents a negligible environment for sulfate attack on concrete exposed to these materials. Degrees of attack are based on the scale of 'negligible,' 'moderate,' 'severe' and 'very severe' as described in the "Design and Control of Concrete Mixtures," published by the Portland Cement Association (PCA). The Colorado Department of Transportation (CDOT) utilizes a corresponding scale with 4 classes of severity of sulfate exposure (Class 0 to Class 3) as described in the published table below.

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

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

Based on these data no special sulfate-resistant cement appears necessary in project concrete.

SOIL CORROSIVITY

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

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

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

Measurements of electrical resistivity indicated values from approximately 1,886 to 9,385 ohm-centimeters in samples of the site earth materials.

pH Where pH is less than 4.0, soil serves as an electrolyte; the pH range of about 6.5 to 7.5 indicates soil conditions that are optimum for sulfate reduction. In the pH range above 8.5, soils are generally high in dissolved salts, yielding a low soil resistivity (AWWA, 2010). Testing indicated pH values of approximately 8.6 to 10.6.

Reduction-Oxidation testing indicated negative potentials: -92 to -213 millivolts. Such low potentials typically create a more corrosive environment.

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

Corrosivity Assessment The American Water Works Association (AWWA, 2010[4]) has developed a point system scale used to predict corrosivity. The scale is intended for protection of ductile iron pipe but is valuable for project steel selection. When the scale equals 10 points or higher, protective measures for ductile iron pipe are suggested. The AWWA scale (Table A.1 Soil-test Evaluation) is presented below. The soil characteristics refer to the conditions at and above pipe installation depth.

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

Table A.1 Soil-test Evaluation

Soil Characteristic / Value	Points
Resistivity	
<1,500 ohm-cm	10
1,500 to 1,800 ohm-cm	8
1,800 to 2,100 ohm-cm	5
2,100 to 2,500 ohm-cm	2
2,500 to 3,000 ohm-cm	1
>3,000 ohm-cm	0
pH	
0 to 2.0	5
2.0 to 4.0	3
4.0 to 6.5	0
6.5 to 7.5	0 *
7.5 to 8.5	0
>8.5	3
Redox Potential	
< 0 (negative values)	5
0 to +50 mV	4
+50 to +100 mV	3½
> +100 mV	0
Sulfide Content	
Positive	3½
Trace	2
Negative	0
Moisture	
Poor drainage, continuously wet	2
Fair drainage, generally moist	1
Good drainage, generally dry	0

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

We anticipate that drainage at the site after construction will be good. With effective drainage, the site soils appear to comprise a severely corrosive environment for ferric materials. (16½ points)

Corrosive conditions can be addressed by use of materials not vulnerable to corrosion, heavier gauge materials with longer design lives, polyethylene encasement, or cathodic protection systems. If additional information is needed regarding soil corrosivity, the American Water Works Association or a corrosion engineer should be contacted.

Structure-specific soil corrosivity studies should be performed to evaluate the conditions in support of utility design. It should be noted, however, that changes to the conditions at a site conditions during construction, such as the import of other soils, or the intended or unintended introduction of off-site water, may alter corrosion potentials significantly. Additional testing may be appropriate during construction.

PROJECT EARTHWORK

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

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

Prior to earthwork construction, vegetation and other deleterious materials should be removed and disposed of off-site. Relic underground utilities should be abandoned in accordance with applicable regulations, removed as necessary, and properly capped. Remnant foundation elements should be entirely removed and the resultant excavation filled with properly compacted backfill.

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.

Drainage During Construction The contractor should take proactive 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 the proposed buildings and other improvements.

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

Existing Fill Soils Un-documented fill soils were encountered at several of the test holes. Because of prior construction at the site, un-documented fill soils likely are present in many areas of the site. We anticipate that the majority of the existing fill soils can be re-used as compacted fill. However, because the contents and composition of all fill soils at the site are not known, some of the excavated existing fill soils may not be suitable for re-use as fill. A geotechnical engineer should be retained during site excavations to observe the excavated fill materials and provide parameters for its suitability for reuse.

At Test Holes B-10 and W-6, existing fill was encountered that contained construction debris, was gray to black in color and had a hydrocarbon odor. Environmental assessment of potentially contaminated soils and their suitability for re-use as compacted fill, as well as any handling requirements, etc., should be provided by an environmental consultant.

Existing Native Soils and Bedrock The native soils and bedrock materials excavated from the site that are free of trash, organic material, construction debris, and other deleterious materials, are suitable, in general, for placement as compacted fill. Organic materials, including excavated lignite or coal if encountered, should not be incorporated into project fills.

Cobbles and fragments of rock (as well as inert construction debris, e.g., concrete or asphalt) up to 6 inches in maximum dimension may be included in project fills, in general. However, such materials should be placed as deeply as possible in the project fills. Such materials should be assessed on a case-by-case basis as they are identified during earthwork. The presence of cobbles in project fills may complicate drilled pier installation, however. Coarser cobbles and boulders, however, should not be incorporated into project fills.

All excavated bedrock to be replaced as compacted fill should be processed into a soil-like mass.

Where limestone and sandstone bedrock are excavated coarse fragments that require crushing to break down likely will be generated. It may be cost effective to export, rather than process such materials.

We anticipate that the excavated clay shales will require more than typical effort to process, place, and compact properly; significant volumes of water likely will be necessary. The excavated material should be disked or otherwise processed until it is broken down into fragments no larger than 3 inches in maximum dimension and moisture-conditioned prior to compaction.

Because of the capacity of the bedrock fragments to absorb water into the structures of the clay mineral grains, sufficient applied water to bring them to desired moisture contents at the time of initial placement may not be sufficient for them to remain at those moisture levels. Some of the excavated bedrock materials will require processing, moisture conditioning, placement and compaction more than once into order to comply the moisture content and compaction criteria provided in this report. The contractor should anticipate this and plan his means and methods accordingly.

Imported Fill Materials If it is necessary to import material to the site, the imported soils should be free of organic material, and other deleterious materials. Imported material should consist of soils that exhibit **between 35 and 70 percent passing the No. 200 Sieve** and should have a plasticity index of **15 or less**. Representative samples of the materials proposed for import should be tested and approved prior to transport to the site.

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

CDOT CLASS 1 STRUCTURE BACKFILL

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

Materials proposed for import as select, granular fill should be tested and approved prior to transport to the site.

Fill Platform Preparation Prior to filling, the top **12 inches** of in-place materials on which fill soils will be placed (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. Where over-excavation is to be performed, then these parameters for subgrade preparation are for the subgrade below the bottom of the specified overexcavation depth.

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

Wet, Soft or Unstable Subgrades Where wet, soft or unstable subgrades are encountered, the contractor must establish a stable platform for fill placement and achieving compaction in the overlying fill soils. Therefore, excavation of the unstable soils and replacing them with relatively dry or granular material, possibly together with the use of stabilization geo-textile or geo-grid, may be necessary to achieve stability. Whereas the stabilization approach should be determined by the contractor, GROUND offers the alternatives below for consideration. Proof-rolling can be beneficial for identifying unstable areas.

- Replacement of the existing subgrade soils with clean, coarse, aggregate (e.g., crushed rock or "pit run" materials) or road base. Excavation and replacement to a depth of 1 to 2 feet commonly is sufficient, but greater depths may be necessary to establish a stable surface.

  On very weak subgrades, an 18- to 24-inch "pioneer" lift that is not well compacted may be beneficial to stabilize the subgrade. Where this approach is employed, however, additional settlements of up to ½ inch may result.

- Where coarse, aggregate alone does not appear sufficient to provide stable conditions, it can be beneficial to place a layer of stabilization geo-textile or geo-grid (e.g., Tencate Mirafi® HP370 or RS 580*i*, or Tensar® BX 1100) at the base of the aggregate section.

The stabilization geo-textile / geo-grid should be selected based on the aggregate proposed for use. It should be placed and lapped in accordance with the manufacturer's recommendations.

Geo-textile or geo-grid products can be disturbed by the wheels or tracks of construction vehicles. We suggest that appropriate care be taken to maintain the effectiveness of the system. Placement of a layer of aggregate over the geo-textile / geo-grid prior to allowing vehicle traffic over it can be beneficial in this regard.

When a given remedial approach has been selected, we suggest constructing a test section to evaluate the effectiveness of the approach prior to use over a larger area.

Benching into Existing Slopes Slopes that are steeper than 5:1 (horizontal : vertical) should be benched prior to placement of fill on them.

Benches should be at least 8 feet in width and have (near-) vertical risers between them no more than 3½ feet in height.

All topsoil and loose, soft or low density soil should be removed by the benching, if not previously removed by stripping.

Benches should be sloped back into the slope at angles of about 3 percent (or roughly 3 inches in 8 feet).

Where indications of groundwater are encountered, every second bench should be provided with a back drain at the toe of the riser. Geotechnical parameters for back drain systems are provided in the *Subsurface Drainage* section of this report.

General Considerations for Fill Placement Fill materials should be mixed thoroughly to achieve a uniform moisture content, placed in uniform lifts not exceeding **8 inches** in loose thickness, and properly compacted. It may be necessary to re-work the fill soils more than once in order to achieve the compaction criteria provided below.

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 ranges are obtained.

Where soils supporting foundations or on which foundation will be placed are exposed to freezing temperatures or repeated freeze – thaw cycling during construction – commonly due to water ponding in foundation excavations – bearing capacity typically is reduced and/or settlements increased due to the loss of density in the supporting soils. After periods of freezing conditions, the contractor should re-work areas affected by wetting or the formation of ice to re-establish adequate bearing support.

Compaction Standards 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 Proctor dry density at moisture contents within 2 percent of optimum moisture content as determined by ASTM D1557, the 'modified Proctor.'

Soils that classify as ML, MH, CL or CH should be compacted to **95 percent** of the maximum Proctor density at moisture contents from 1 percent below to 3 percent above the optimum moisture content as determined by ASTM D698, the 'standard Proctor.'

Use of Squeegee Relatively uniformly graded fine gravel or coarse sand, i.e., "squeegee," or similar materials commonly are proposed for backfilling foundation excavations, utility trenches (excluding approved pipe bedding), and other areas where employing compaction equipment is difficult. In general, squeegee should not be used on this project.

Instead, excavations not ackfilled with properly compacted fill should be backfilled with "Controlled Low Strength Material" (CLSM), i.e., a lean, sand-cement slurry ("flowable fill") or a similar material.

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 6-foot fill, that corresponds to settlement of about 1 inch. 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 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 into appropriate drainage pathways or structures.

Bulkage and Shrinkage The estimates below for bulking and shrinkage of excavated site materials replaced as compacted fill may be used for preliminary earthwork volume calculations. The actual volume changes realized over the course of project grading necessarily are highly dependent upon the average depth of earthworking, the compaction methods used, and the average degree of compaction achieved.

Excavated Sandstone, Limestone and Shale likely will exhibit between 0 and 8 percent bulking.

Overburden Soils likely will exhibit from 3 to 6 percent shrinkage.

Slope Creep Slopes more than 10 feet in height are proposed for this project. The shallow soils supporting the slope will tend to ‘relax’ or ‘creep’ downslope with time. Improvements placed in proximity to the tops-of-slope can be displaced and/or rotated by these soil movements with associated (typically limited) distress. Curbs, pavements, flatwork, fences, and other improvements that are intolerant of this displacement and potential distress should be set back from the tops-of-slope or anchored below the zone of ‘active’ or ‘creeping’ soils: typically about 4 feet. GROUND can provide additional discussion in this regard upon request.

EXCAVATION CONSIDERATIONS

Excavation Difficulty The test holes were advanced to the depths indicated on the test hole logs by means of conventional truck-mounted drilling equipment. We anticipate no unusual excavation difficulties, across most of the site, in the site soils and bedrock with conventional, heavy-duty excavating equipment in good working condition, although excavation likely will be relatively slow.

Very high penetration resistance values, however, were obtained in the Greenhorn Limestone and locally within the bedrock shales. Practical drill rig refusal was encountered locally (Test Holes B-11, and W-3). Where encountered in project excavations, these very hard beds and lenses will be more difficult to excavation or to advance drilled pier holes through. The contractor should be prepared to excavate very hard, resistant bedrock and to handle, process, and, if necessary, export such materials. The use of very heavy duty excavation equipment, e.g., a Caterpilar D10 or other larger dozer with a single-shank ripper may be beneficial where resistant bedrock is encountered. Specialized breaking equipment, or limited, local blasting may be cost effective to facilitate project excavations and to break local masses of resistant rock. We anticipate that the volumes of excavation for which blasting will be necessary will be low, however.

Temporary Excavations and Personnel Safety Excavations in which personnel will be working must comply with all applicable OSHA Standards and Regulations, particularly CFR 29 Part 1926, OSHA Standards-Excavations, adopted March 5, 1990. The contractor’s “responsible person” should evaluate the soil exposed in the excavations as part of the contractor’s safety procedures. GROUND has provided the information in this report solely as a service to Rubicon Development and is not assuming responsibility for construction site safety or the contractor’s activities.

The contractor should take care when making excavations not to compromise the bearing or lateral support for any adjacent, existing improvements.

We recommend temporary, un-shored excavation slopes up to **20 feet** in height be cut no steeper than **1½ : 1** (horizontal : vertical) in the site soils in the absence of seepage or adversely oriented bedding conditions. The portions of temporary slopes supported by relatively un-weathered bedrock likely can be cut as steeply as **¾:1** (horizontal : vertical) in the absence of seepage or adversely oriented bedding planes. Such slopes should be evaluated on a case-by-case basis, however. Some surface sloughing may occur on the slope faces at these angles. Local conditions encountered during construction, such as groundwater seepage and loose sand, will require flatter slopes.

Stockpiling of materials should not be permitted closer to the tops of temporary slopes than 5 feet or a distance equal to the depth of the excavation, whichever is greater.

Should site constraints prohibit the use of the recommended slope angles, temporary shoring should be used. Lateral earth pressures provided in the *Lateral Loads* section of this report may be used for planning purposes. However, a qualified engineer should be retained by the contractor to confirm these values at individual shoring locations or provide alternative parameters, and to design the shoring.

Groundwater and Surface Water Groundwater was encountered at the time of drilling in some test holes at depths ranging from approximately 10 to 29 feet below existing grades (elevations from about 5,476 to 5,529 feet). Approximately 37 to 38 days after drilling, groundwater was noted in the some test holes at depths ranging from approximately 4 to 24 feet (elevations from about 5,532 to 5,482 feet).

Like the existing and proposed site elevations, the depths to groundwater were significantly variable. Groundwater likely will be encountered in some project excavations but not in others. Water may be encountered at higher elevations or shallower depths than noted above, at least seasonally. De-watering may be needed to complete some earthwork, including for utility line installation, etc. Limited volumes of perched water will be encountered at still higher elevations.

Should un-anticipated seepage or flowing groundwater be encountered in project excavations, a geotechnical engineer should be retained to evaluate the conditions and provided additional recommendations, as appropriate. The risk of slope instability will be significantly increased in areas of seepage along excavation slopes.

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

Temporary slopes should also be protected against erosion. Erosion along the slopes will result in sloughing and could lead to a slope failure.

UTILITY LATERAL INSTALLATION

Pipe Support The bearing capacity of the site soils appeared adequate, in general, for support of anticipated water lines. The pipe + water are less dense than the soils which will be displaced for installation. Therefore, GROUND anticipates no significant pipe settlements in these materials where properly bedded. Because of the differential fill thicknesses, etc., at the site with the resultant potential for differential settlements, gravity lines should be set as steeply as possible.

Excavation bottoms may expose soft, loose or otherwise deleterious materials, including debris. Firm materials may be disturbed by the excavation process. All such unsuitable materials should be excavated and replaced with properly compacted fill. Areas allowed to pond water will require excavation and replacement with properly compacted fill. The contractor should take particular care to ensure adequate support near pipe joints, which are less tolerant of extensional strains.

Where thrust blocks are needed, they may be designed utilizing the values provided in the *Lateral Loads* section of this report.

Manholes The following geotechnical criteria may be used for manhole design:

1) Manhole footings should bear on firm, native soils or bedrock.

   If soft or wet soils, or fill soils are exposed at footing bearing elevation, then those materials should be excavated and the footing deepened, or the footing should bear on at least 2 feet of clean, 1-inch nominal, crushed rock.

The crushed rock section, if used, should extend at least 2 feet laterally beyond each manhole footing. The crushed rock section should be wrapped in filter fabric (Tencate Mirafi® 140N, or the equivalent).

2) Footings bearing on firm, native soils or bedrock, or on a section of crushed rock as outlined above may be designed for an allowable bearing pressure of 2,000 psf, for footings up to 6 feet in width (minimum lateral dimension).

3) Manhole footings should have a minimum lateral dimension of 24 or more inches. Actual footing dimensions, however, should be determined by the structural engineer.

4) The resistance of manholes to lateral loads may be evaluated using the geotechnical parameters provided in the *Lateral Loads* section of this report.

5) Where local soils are placed as fill over portions of the manhole structure, a moist unit weight of the fill soil of 125 pcf may be assumed for design purposes.

6) Footing excavations may expose wet soils locally. A layer of lean concrete or gravel can be placed in the bottom of foundation excavations prior to steel and concrete placement. This "mud mat" will reduce disturbance of the natural soils caused by construction operations. Disturbing the native soils will increase potential settlements.

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

If used, the CLSM used as pipe bedding and trench backfill should exhibit a 28-day unconfined compressive strength **between 50 and 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 Rubicon Development.

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. Where resistant bedrock is excavated, however, unsuitably coarse fragments may comprise a large fraction of the excavated materials.

If it is necessary to import material for use as backfill, the imported soils should be free of vegetation, organic debris, and other deleterious materials and meet the criteria for imported soils provided in the *Project Earthwork* section of this report.

Criteria for fill placement and compaction are provided in the *Project Earthwork* section of this report. Note that where pipes are bedded, the bottom of the trench need not be scarified and re-compacted.

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

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

If a granular bedding material is specified, then design and installation follow ASTM D2321, Appendix X1.8, with regard to potential migration of fines into the pipe bedding. 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 to interrupt the granular bedding section to reduce the rates and volumes of water transmitted along the sewer alignment which can contribute to migration of fines.

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

SURFACE DRAINAGE

The site soils are relatively stable with regard to moisture content – volume relationships at their existing moisture contents. Other than the anticipated, post-placement settlement of fills, post-construction soil movement will result primarily from the introduction of water into the soil underlying the proposed structures, 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 followed both during construction and as part of project design. The facility should be observed periodically to evaluate the surface drainage and identify areas where drainage is ineffective. Routine maintenance of site drainage should be undertaken throughout the design life of the project. If these measures are not implemented and maintained effectively, the movement estimates provided in this report could be exceeded. Particular attention should be paid to generally north-facing areas where evaporation commonly is less.

1) Wetting or drying of the foundation excavations and underslab areas should be avoided during and after construction. 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 areas not covered with pavement or concrete slabs, or a minimum 3 percent in the first 10 feet in 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 <u>and maintained</u> to direct water away from sidewalks and other hardscaping as well as utility trench alignments which are not tolerant of increased, post-construction movements.

The ground surface near foundation elements should be able to convey water away readily. Cobbles and similar materials will tend to act as baffles, directing water downward, rather than away. 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) Existing and constructed slopes at the site will descend toward buildings locally. 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 a graded or retained slopes descending toward a building, surface drainage swales and/or interceptor drains should be installed between the improvements and the slope.

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

6) Roof downspouts and drains should discharge well beyond the perimeter of the structure foundations (**minimum 15 feet**) and backfill zones and be provided with positive conveyance off-site for collected waters.

   If roof downspouts and drains are not used, then surface drainage design should accommodate concentrated volumes of water adjacent to the building.

7) Irrigation water – both that applied to landscaped areas and over-spray – is a significant cause of distress to improvements. Where (near-)saturated soil conditions are sustained, distress to nearby improvements should be anticipated.

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.

The 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 into soils supporting improvements, 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 or required 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, local, shallow underdrains beneath the planter beds can be used in lieu of water-tight planters.

Colorado Geological Survey – Special Publication 43 provides additional guidelines for landscaping and reducing the amount of water that infiltrates into the ground.

8) 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.

9) Regular maintenance will be required to maintain effective surface drainage. Maintenance should be anticipated to include removal and replacement of sidewalk stones, reaches of curb and gutter, sections of pavement, etc., as well as local re-grading 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 saturated subsurface waters. Underdrains will not collect water infiltrating under unsaturated (vadose) conditions, or moving via capillarity, however. In addition, if not properly constructed and maintained, underdrains can transfer water into foundation soils, rather than remove it. This will tend to induce heave or settlement of the subsurface soils, and may result in distress. Underdrains can, however, provide an added level of protection against relatively severe post-construction movements by draining saturated conditions near individual structures should they arise, and limiting the volume of wetted soil.

At the subject site, because of the relief across the site, the differential subgrade conditions, and the dipping bedrock underlying the site, each building should be provided with a perimeter underdrain system. Edge underdrains along the upslope sides of pavements, flatwork areas, etc., also can reduce distress to those elements.

If a below-grade level of limited area extends to greater depths than other portions of a building, e.g., an elevator pit or partial basement, then it may be cost-effective to construct a local, lower underdrain system for that element.

Damp-proofing should be applied to the exteriors of foundation walls for buildings with below-grade or partially below-grade levels. 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.

Geotechnical Parameters for Perimeter Underdrain Design Where underdrain systems are included in project drainage design, they should be designed in accordance with the parameters below. The actual underdrain layout, outlets, and locations should be developed by a civil engineer. Typical, cross-section details of underdrains are provided in Figures 17 and 18. Other typical details can be provided upon request.

Each underdrain system should be tested by the contractor after installation and after placement and compaction of the overlying backfill to verify that the system functions properly.

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

2) The free-draining gravel should be naturally occurring (not recycled) with **5 percent or less** 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**.

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

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

4) Each underdrain system above the water table should be designed to discharge **at least 10 gallons per minute** of collected water.

5) The high point(s) for the collection pipe(s) should be below the grade beam or footing as shown in the details. Multiple high points for a single system can be beneficial for reducing the depth to which a system would be installed.

   The collection and discharge pipe for the system should be installed at the slope determined by the underdrain designer. Pipe slopes should be selected to accommodate **at least ½ inch** of differential movement over spans of **50 feet**.

   Underdrain 'clean outs' should be provided for each underdrain system at intervals of no more than **150 feet**, at collection and discharge pipe elbows of **60 degrees or more**, and elsewhere as appropriate to facilitate underdrain maintenance.

6) 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.

7) Maintenance of the underdrain systems should be performed regularly to ensure that the systems continue to work effectively.

Geotechnical Parameters for Back Drain Design Back drains should be designed in accordance with the parameters above for underdrain systems, as modified below. The actual back drain layouts, outlets, etc., should be developed by a civil engineer. A typical, cross-section detail of a back drain for this project is provided on Figure 19.

- Each back drain system should be designed to discharge **at least 3 gallons per minute** of collected water per 150 feet of slope length.
- A back drain should be laid along the toe of the benching riser after removal of soft, loose, and low density materials.
- Between high points, the back drain collection pipes should be laid on slopes of **about 2 percent** toward a discharge pipe.
- The back drain discharge pipe(s) should convey collected water through the slope face to discharge point(s) well beyond the toe of slope.
- No waterproof membrane is needed as a component of an interceptor drain system.
- Back drain 'clean-outs' should be provided at regular intervals to facilitate maintenance of the underdrains. In general, GROUND recommends that cleanouts be placed at **approximately 200-foot centers** along the system. Cleanouts also should be located at pipe elbows that entail angles **greater than 60 degrees**.

PAVEMENT SECTIONS

A pavement section is a layered system designed to distribute concentrated traffic loads to the subgrade. Performance of the pavement structure is directly related to the physical properties of the subgrade soils and traffic loadings.

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

Pavement designs for the private pavements were developed in general accordance with the design guidelines and procedures of the American Association of State Highway and Transportation Officials (AASHTO).

Subgrade Materials Based on the results of our field exploration and laboratory testing, the potential pavement subgrade materials classify as A-2-6 to A-7-6 soils in accordance with the American Association of State Highway and Transportation Officials (AASHTO) classification system.

An R-value of 5 was estimated based on the laboratory testing of the “worst case” pavement subgrade materials. R = 5 correlates to a resilient modulus of 3,025 psi based on the CDOT correlation. 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.

Estimated Traffic Specific traffic data was unavailable at the time of our report preparation. Based on our experience with similar projects, equivalent 18-kip daily load application (EDLA) values of 3, 10, and 50 were assumed for the general parking lot areas, the proposed roadway/drive lanes, and any heavy vehicle/fire lanes, respectively. The EDLA values of 3, 10, and 50 were converted to equivalent 18-kip single axle load (ESAL) values of 21,900, 73,000, and 365,000 respectively for 20-year design lives. If design traffic loadings differ significantly from these assumed values, GROUND should be notified to re-evaluate the pavement sections below.

Pavement Sections The soil resilient modulus and the ESAL values were used to determine the required structural number for the project pavements which then was then used to develop the pavement sections based on the DARWin™ computer program that solves the 1993 AASHTO pavement equations. A reliability level of 80 percent and a terminal serviceability of 2.0 were utilized for design of the pavement sections. A structural coefficient of 0.40 was used for hot bituminous asphalt and 0.12 was used for aggregate base course. The resultant minimum pavement sections are tabulated below.

Minimum Pavement Sections

Location	***Full Depth Asphalt*** *(inches Asphalt)*	***Composite Section*** *(inches Asphalt / inches Aggregate Base)*	***Rigid Section*** *(inches Concrete / inches Aggregate Base)*
Parking Lot	**5½**	**4 / 6**	**6 / 6**
Drive Lanes	**6½**	**4 / 9**	**6 / 6**
Heavy Truck Traffic Areas	-	-	**7 / 6**

Truck loading and unloading areas, trash collection areas, as well as other pavement areas subjected to high turning stresses or heavy truck traffic should be provided with rigid pavements consisting of 7 or more inches of portland cement concrete. All concrete sections should be underlain by 6 inches of properly compacted CDOT Class 6 Aggregate Base Course.

A flexible section that is theoretically equivalent to the rigid section indicated for the heavy vehicle traffic areas would be 5½ inches of asphalt over 11 inches of aggregate base course. In GROUND's experience, however, a rigid (concrete) pavement will provide significantly superior performance relative to a flexible pavement along the routes where trash trucks and other heavy vehicles turn repeatedly, etc.

Pavement Materials

Asphalt 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 City of Boulder design requirements.

Aggregate gradation **S** (nominal ¾-inch) and binder type **PG64-22** should be used for the lower lift(s), and gradation **SX** (nominal ½-inch) and binder type **PG64-22** for the top lift.

For the lower (S) lift(s), lift thicknesses generally should be between **2¼ and 3½ inches**. The top (SX) lift generally should be between **2 and 3 inches** in thickness.

Aggregate base material should meet the criteria of CDOT Class 6 Aggregate Base Course. Base course should be placed in and compacted in accordance with the standards in the *Project Earthwork* section of this report.

Concrete Pavement concrete should consist of a plant mix composed of a mixture of aggregate, portland cement and appropriate admixtures meeting the requirements of a job-mix formula established by a qualified engineer as well as applicable City of Littleton design requirements design requirements. Concrete should have a minimum modulus of rupture of third point loading of **650 psi**. Normally, concrete with a 28-day compressive strength of 4,500 psi should develop this modulus of rupture value. The concrete should be air-entrained with approximately 6 percent air and should have a minimum cement content of **6 sacks per cubic yard**. Maximum allowable slump should be **4 inches**.

These concrete mix design criteria should be coordinated with other project requirements including any criteria for sulfate resistance presented in the *Water-Soluble Sulfates* section of this report. To reduce surficial spalling resulting from freeze-thaw cycling, we suggest that pavement concrete meet the requirements of CDOT Class P concrete. In addition, the use of de-icing salts on concrete pavements during the first winter after construction will increase the likelihood of the development of scaling. Placement of flatwork concrete during cold weather so that it is exposed to freeze-thaw cycling before it is fully cured also increases its vulnerability to scaling. Concrete placing during cold weather conditions should be blanketed or tented to allow full curing. Depending on the weather conditions, this may result in 3 to 4 weeks of curing, and possibly more.

Concrete pavements should contain sawed or formed joints. CDOT and various industry groups provide guidelines for proper design and concrete construction and associated jointing. In areas of repeated turning stresses, such as truck loading and unloading areas, the concrete pavement joints should be fully tied and doweled. Example layouts for joints, as well as ties and dowels, which may be applicable, can be found in CDOT's M standards, found at the CDOT website: http://www.dot.state.co.us/DesignSupport/. PCA, ACI and ACPA publications also provide useful guidance in these regards. Joint spacings less than the 15-foot maximum indicated in in CDOT's M standards, e.g., 10 feet or 12 feet, may be beneficial to reduce concrete cracking.

Aggregate Base Course Aggregate base course should consist of natural (not recycled) materials meeting the criteria of CDOT Class 6 Aggregate Base Course. Aggregate base course should be placed and compacted in accordance with the criteria in the Project Earthwork section of this report.

Subgrade Preparation Although subgrade preparation to a depth of 12 inches is common in the general project area, pavement performance commonly can be improved by a greater depth of moisture-density conditioning of the soils, as indicated below. Remedial earthwork will not prevent premature pavement distress, but will tend to reduce it.

Remedial Earthwork Due to potential for heave in the shallow site soils, the pavements should be constructed on a section of properly moisture-conditioned and compacted fill at least **2 feet** or a thickness that completely removes and replaces all the undocumented fill soils, whichever is greater. This section assumes that a) traffic speeds in the parking areas and driveways will be relatively slow, and b) the facility owner will be tolerant of significant total and differential pavement post-construction movements and the associated maintenance costs that that are necessary to re-establish effective drainage, replace distressed pavement, etc.

If the owner opts to leave some of the existing fill soils in-place beneath paved areas, additional settlements, accelerated pavement distress, and additional maintenance should be anticipated. Similarly, where existing utility lines or other site constraints limit the depth to which remedial earthwork can be accomplished, additional maintenance should be anticipated.

If a performance like a slab-on-grade floor is desired, then pavements should be constructed over a similar section of properly compacted fill.

Subgrade preparation of the selected depth should extend the full width of the pavement from back-of-curb to back-of-curb. The subgrade for any sidewalks and other project hardscaping also should be prepared in the same manner.

Geotechnical criteria for fill placement and compaction are provided in the *Project Earthwork* section of this report. The contractor should be prepared to either dry the subgrade materials or moisten them, as needed, prior to compaction. In particular, wet soils should be anticipated where existing pavements are removed. It may be difficult for the contractor to achieve and maintain compaction in some on-site soils encountered without careful control of water contents. Likewise, some site soils likely will “pump” or deflect during compaction if moisture levels are not carefully controlled. The contractor should be prepared to process and compact such soils to establish a stable platform for paving, including use of chemical stabilization, if necessary.

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 moisture content.

Additional Observations The collection and diversion of surface drainage away from paved areas is extremely important to the satisfactory performance of the pavements. The subsurface and surface drainage systems should be designed carefully to ensure removal of the water from paved areas and subgrade soils. Allowing surface waters to pond on pavements will cause premature pavement deterioration. Where topography, site constraints, or other factors limit or preclude adequate surface drainage, pavements should be provided with edge drains to reduce loss of subgrade support. The long-term performance of the pavement also can be improved greatly by proper backfilling and compaction behind curbs, gutters, and sidewalks so that ponding is not permitted and water infiltration is reduced.

Landscape irrigation in planters adjacent to pavements and in "island" planters within paved areas should be carefully controlled or differential heave and/or rutting of the nearby pavements will result. Drip irrigation systems are suggested for such planters to reduce over-spray and water infiltration beyond the planters. Enclosing the soil in the planters with plastic liners and providing them with positive drainage also will reduce differential moisture increases in the surrounding subgrade soils.

In our experience, infiltration from planters adjacent to pavements is a principal source of moisture increase beneath those pavements. This wetting of the subgrade soils from infiltrating irrigation commonly leads to loss of subgrade support for the pavement with resultant accelerating distress, loss of pavement life and increased maintenance costs. This is particularly the case in the later stages of project construction after landscaping has been emplaced but heavy construction traffic has not ended. Heavy vehicle traffic over wetted subgrade commonly results in rutting and pushing of flexible pavements, and cracking of rigid pavements. In relatively flat areas where design drainage gradients necessarily are small, subgrade settlement can obstruct proper drainage and yield increased infiltration, exaggerated distress, etc. (These considerations apply to project flatwork, as well.)

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

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

GROUND's experience indicates that longitudinal cracking is common in asphalt-pavements generally parallel to the interface between the asphalt and concrete structures such as curbs, gutters or drain pans. Distress of this type is likely to occur even where the subgrade has been prepared properly and the asphalt has been compacted properly. The use of thick base course or reinforced concrete pavement can reduce this. Our office should be contacted if these alternates are desired.

The assumed traffic loading does not include excess loading conditions imposed by heavy construction vehicles. Consequently, heavily loaded concrete, lumber, and building material trucks can have a detrimental effect on the pavement. An effective program of regular maintenance should be developed and implemented to seal cracks, repair distressed areas, and perform thin overlays throughout the life of the pavements.

EXTERIOR FLATWORK

We anticipate that the exteriors of proposed buildings and other portions of the site will be provided with concrete flatwork. Like other site improvements, flatwork will experience post-construction movements as soil moisture contents increase after construction and distress likely will result. Due to the variability of the materials encountered at the site, the potential for damaging movement for exterior flatwork and other hardscaping vary greatly. The following measures will help to reduce damages to these improvements, but will not prevent all movements. <u>To achieve performance similar to a slab-on-grade floor, similar depths of remedial earthwork will be necessary</u>.

1) The soils beneath project sidewalks, paved entryways and patios, masonry planters and short, decorative walls, and other flatwork should be excavated and/or scarified to a depth of **2 feet or more**, moisture-conditioned and properly re-compacted.

   In addition, remedial earthwork should be advance to sufficient depth to remove all existing, un-documented fill soils from beneath flatwork and replace them with properly compacted fill. If the owner opts not to remove all of the existing fill soils, additional flatwork maintenance should be anticipated.

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

2) Prior to placement of flatwork, a proof roll should be performed to identify areas that exhibit instability and deflection. The deleterious soils in these areas should be removed and replaced with properly compacted fill. The contractor should take care to achieve and maintain compaction behind curbs to reduce differential sidewalk settlements. Passing a proof roll is an additional requirement to placing and compacting the subgrade fill soils within the specified ranges of moisture content in the *Project Earthwork* section of this report. Subgrade stabilization may be cost-effective in this regard.

3) Flatwork should be provided with control joints extending to an effective depth and spaced no more than **10 feet** apart, both ways. Narrow flatwork, such as sidewalks, likely will require more closely spaced joints.

4) In no case should exterior flatwork extend to under any portion of the building where there is less than **2 inches** of vertical clearance between the flatwork and any element of the building. Exterior flatwork in contact with brick, rock facades, or any other element of the building can cause damage to the structure if the flatwork experiences movements.

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

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

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

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

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

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

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

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

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

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

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

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

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

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 information in this report.

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

Materials Testing Rubicon Development 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 Rubicon Development as it pertains to The Mapleton development as described herein. It may not contain sufficient information for other parties or other purposes. The owner or any prospective buyer relying upon this report must be made aware of and must agree to the terms, conditions, and liability limitations outlined in the proposal.

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

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

If during construction, surface, soil, bedrock, or groundwater conditions appear to be at variance with those described herein, a geotechnical engineer should be advised 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.

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

ALL DEVELOPMENT CONTAINS INHERENT RISKS. It is important that ALL aspects of this report, as well as the estimated performance (and limitations with any such estimations) of proposed project improvements are understood by Rubicon Development and properly conveyed to any future owner(s). Utilizing these parameters for planning, design, and/or construction constitutes understanding and acceptance of conclusions or information provided herein, potential risks, associated improvement performance, as well as the limitations inherent within such estimates.

If any information referred to herein is not well understood, it is imperative for the Client, Owner (if different), or anyone using this report to contact the author or a company principal immediately. 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 Boulder County, Colorado, area at the date of preparation. GROUND makes no warranties, either expressed or implied, as to the professional data, opinions or conclusions contained herein. This document, together with the concepts and conclusions presented herein, as an instrument of service, is intended only for the specific purpose and client for which it was prepared. Re-use of, or improper reliance on this document without written authorization and adaption by GROUND Engineering Consultants, Inc., shall be without liability to GROUND Engineering Consultants, Inc.

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

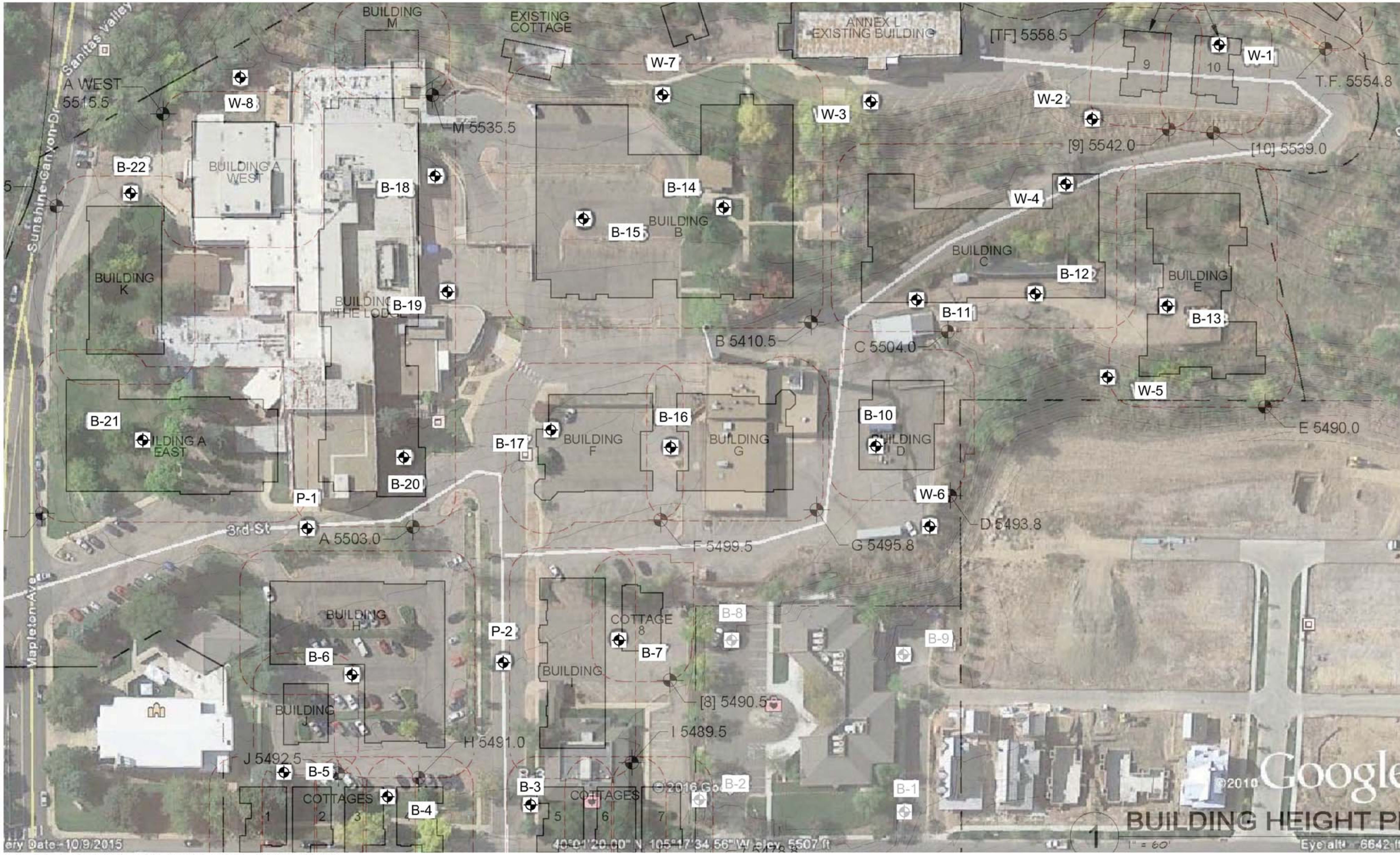
Sincerely,

GROUND Engineering Consultants, Inc.

COLORADO LICENSED
BRIAN H. RECK
48265
PROFESSIONAL ENGINEER

9/21/2016

Kelsey Van Bemmel, P.E. Reviewed by Brian H. Reck, P.G., C.E.G., P.E.



GOOGLE EARTH AERIAL IMAGE (DATE UNKNOWN)
SITE PLAN PROVIDED BY CLIENT

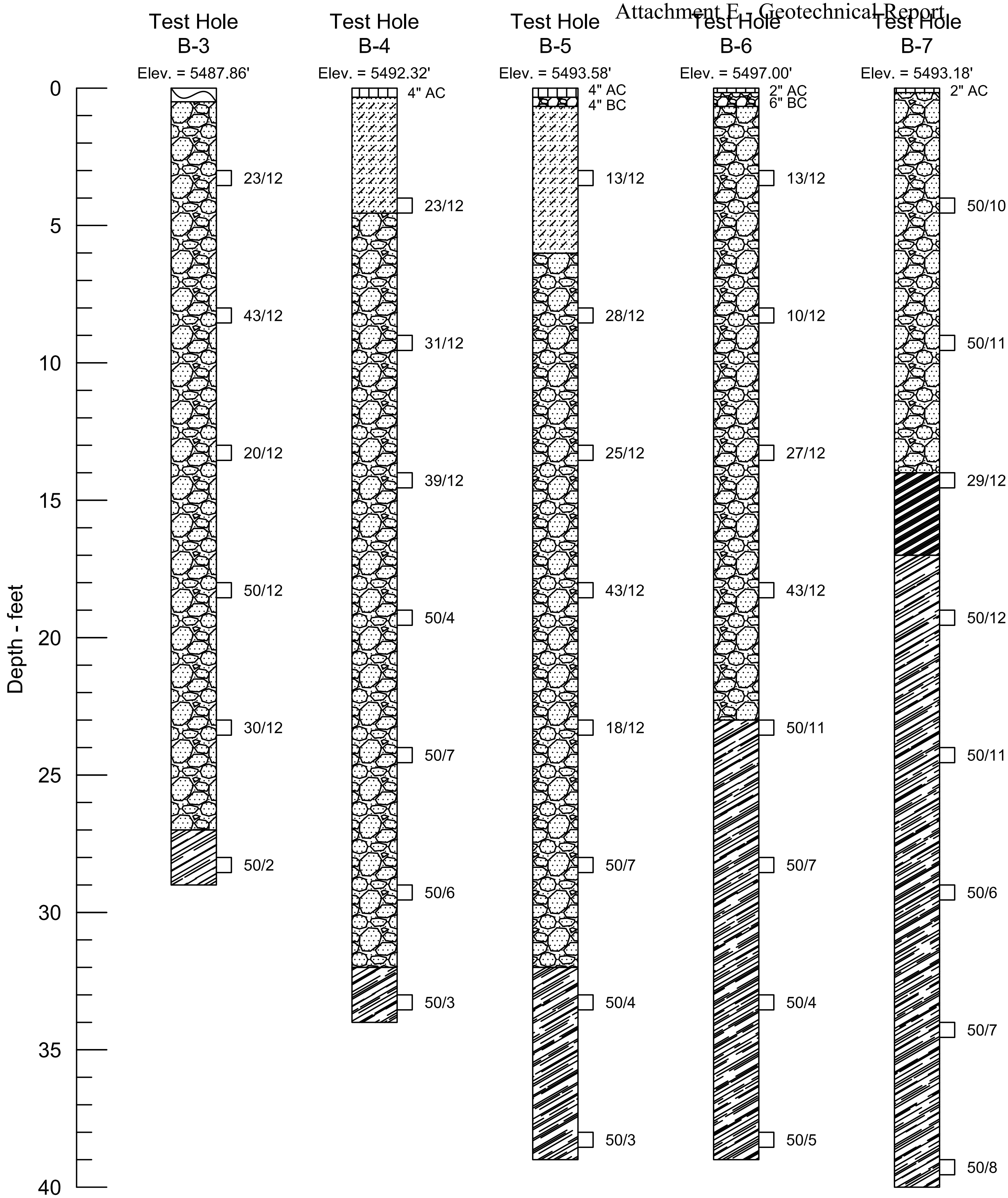
1 Indicates test hole number and approximate location.

1 Indicates test hole number and approximate location. That were not drilled.

(Not to Scale)

GROUND
ENGINEERING CONSULTANTS

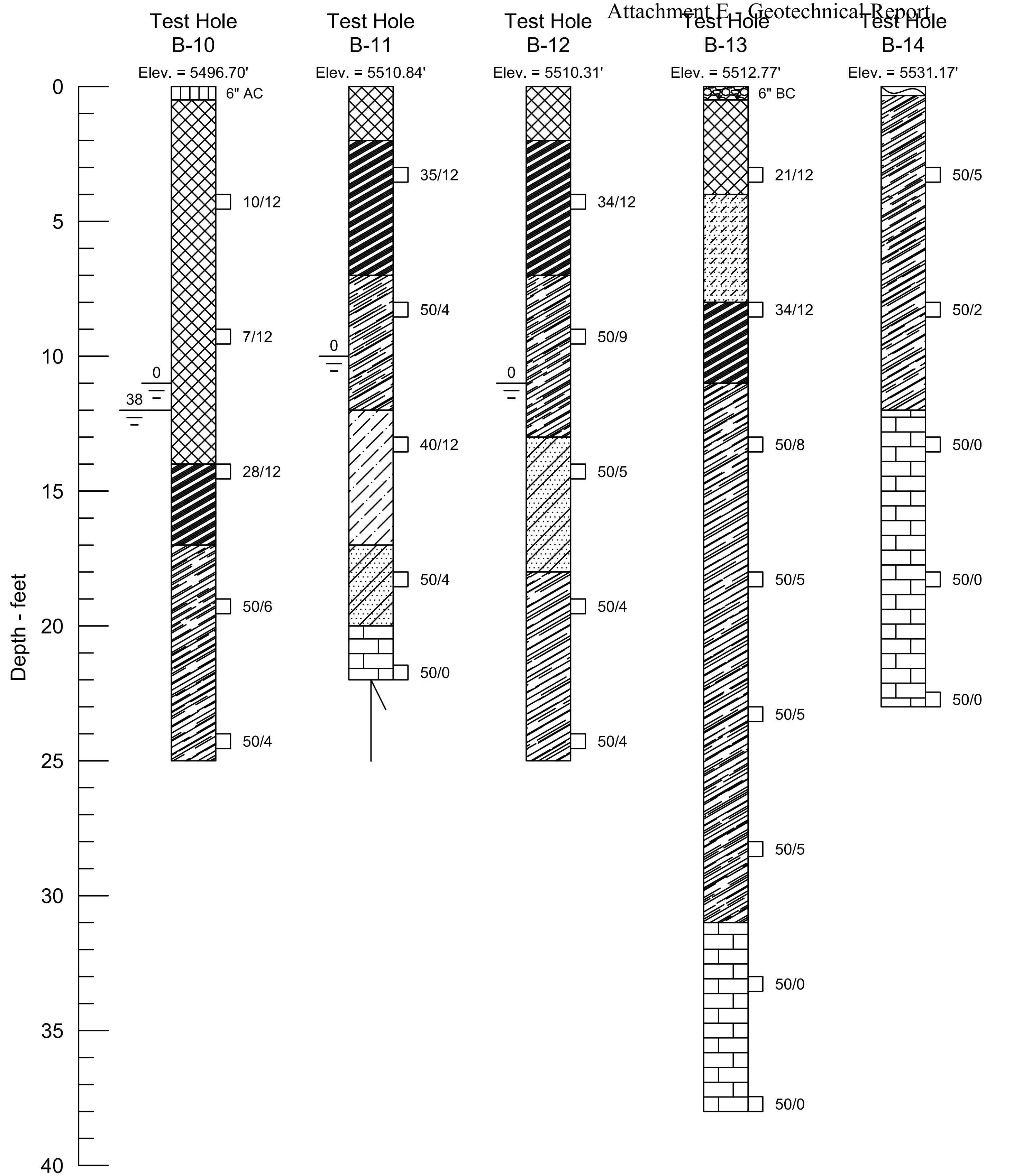
LOCATION OF TEST HOLES	
JOB NO.: 16-0011	FIGURE: 1
CADFILE NAME: 0011SITE.DWG	



GROUND ENGINEERING CONSULTANTS

LOGS OF TEST HOLES

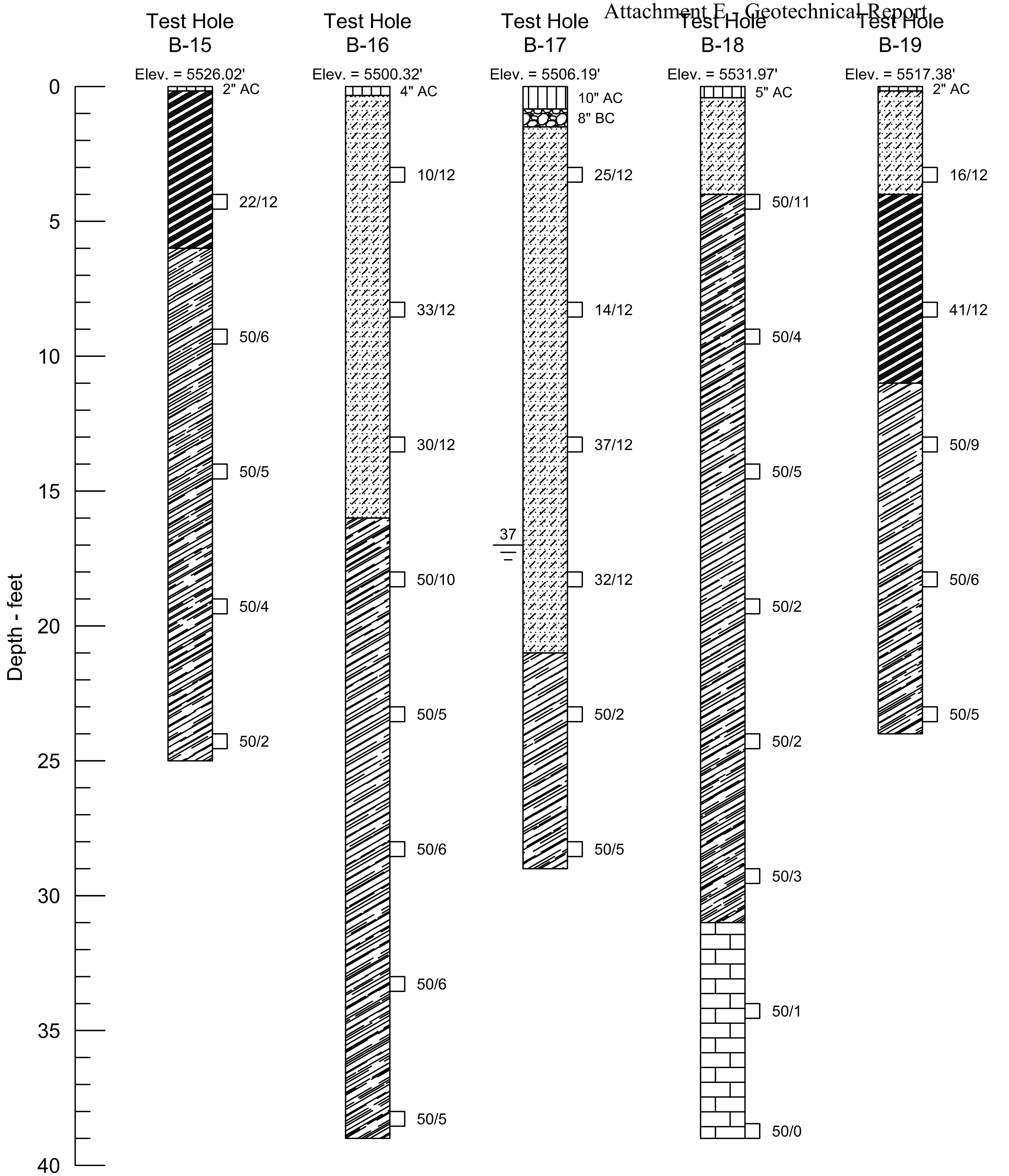
JOB NO.: 16-0011	FIGURE: 2
CADFILE NAME: 0011LOG01.DWG	



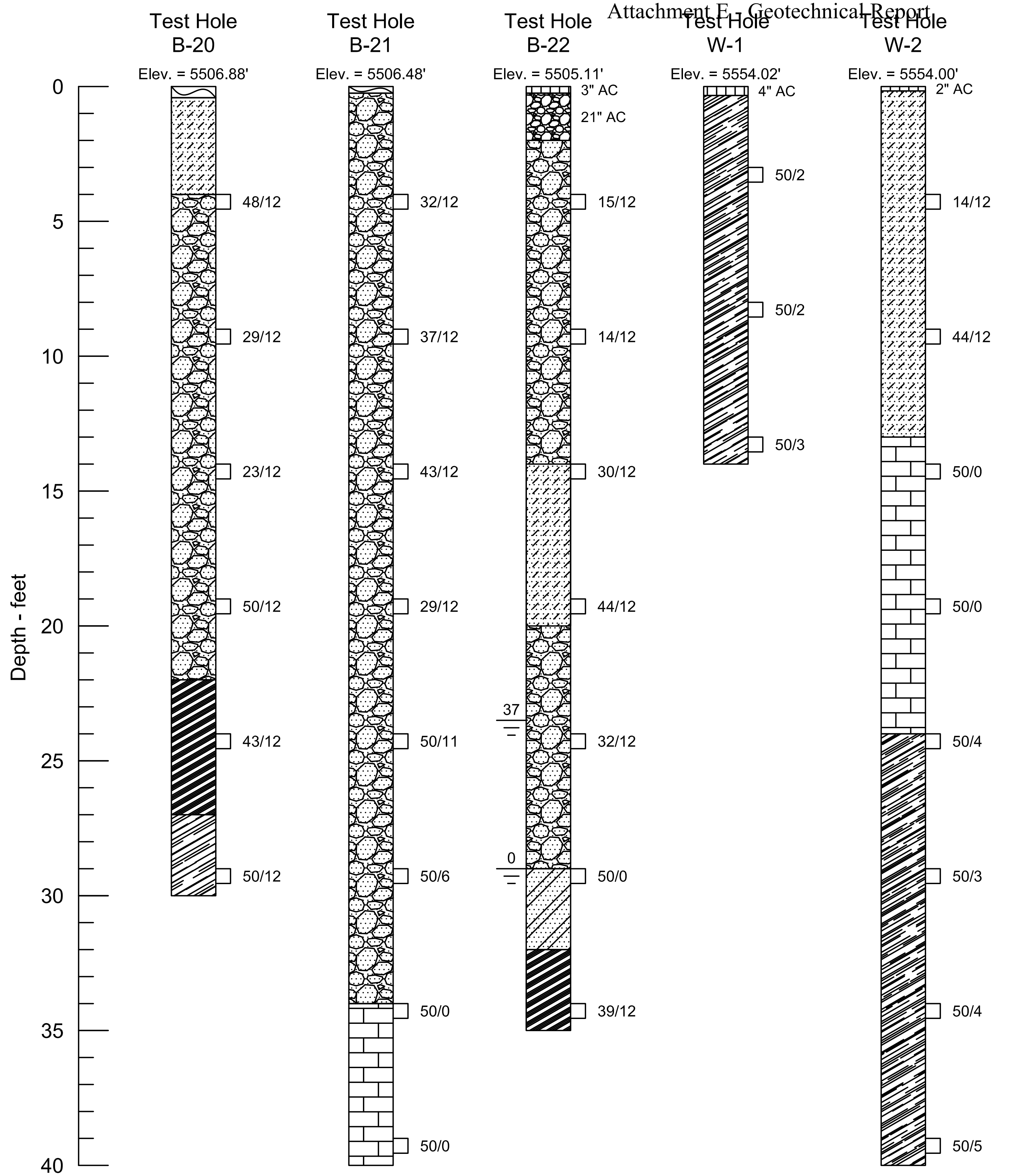
GROUND ENGINEERING CONSULTANTS

LOGS OF TEST HOLES

JOB NO.: 16-0011	FIGURE: 3
CADFILE NAME: 0011LOG02.DWG	



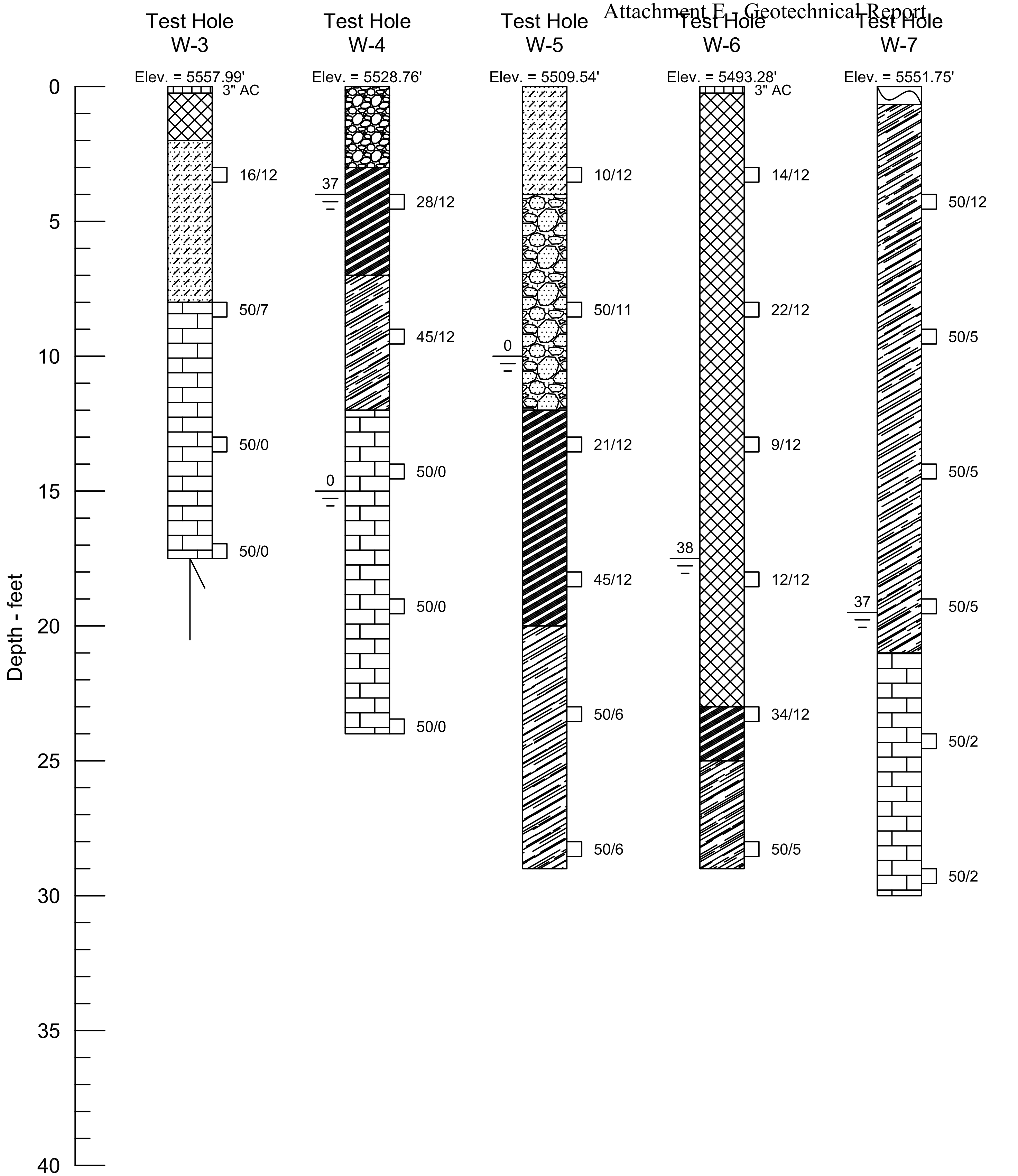
GROUND ENGINEERING CONSULTANTS	
LOGS OF TEST HOLES	
JOB NO.: 16-0011	FIGURE: 4
CADFILE NAME: 0011LOG03.DWG	



GROUND ENGINEERING CONSULTANTS

LOGS OF TEST HOLES

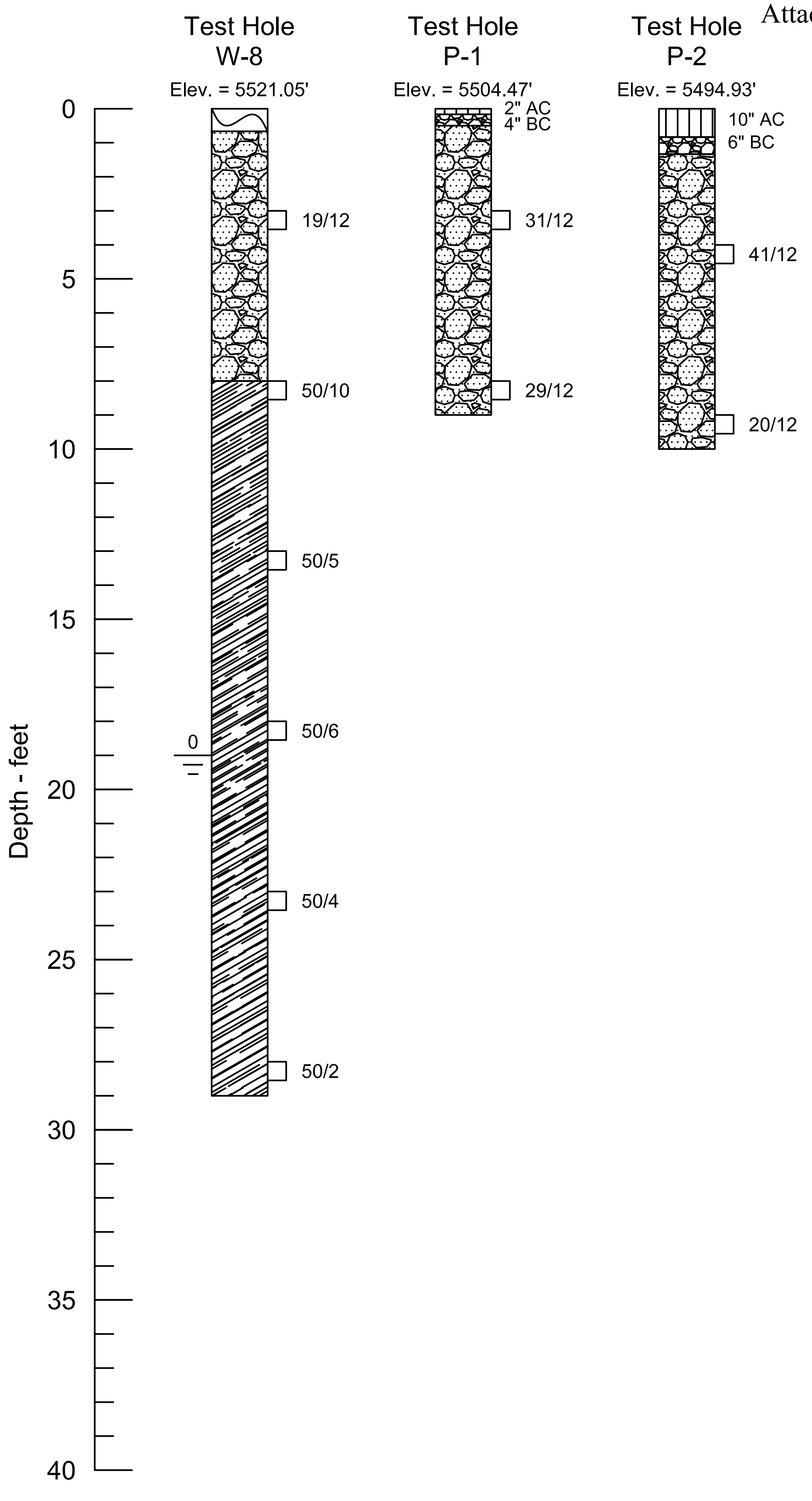
JOB NO.: 16-0011	FIGURE: 5
CADFILE NAME: 0011LOG04.DWG	



GROUND
ENGINEERING CONSULTANTS

LOGS OF TEST HOLES

JOB NO.: 16-0011	FIGURE: 6
CADFILE NAME: 0011LOG05.DWG	



GROUND ENGINEERING CONSULTANTS

LOGS OF TEST HOLES

JOB NO.: 16-0011	FIGURE: 7
CADFILE NAME: 0011LOG06.DWG	

LEGEND:

Topsoil

Asphalt

Base Course

Fill: Fine to coarse sands and gravels with clay and scattered construction debris. They were non- to medium plastic, loose to medium dense, slightly moist to wet, and brown to black in color.

Sandy Clays: Dry to moist, low to medium plastic, stiff to very stiff, and light brown to red-brown in color, with local caliche and iron oxide staining. The sand fractions were fine to medium.

Sands and Gravels: Fine to coarse with local clay, low to medium plastic, medium dense to very dense, dry to wet, and brown to red- brown in color. Caliche was noted locally, as was iron oxide staining.

Weathered Clay Shale: Medium to highly plastic, weathered, slightly moist, and pale brown to gray in color. Iron oxide staining was common.

Clay Shale: Medium to highly plastic, hard to very hard, dry to slightly moist, and pale brown to gray to black in color. Iron oxide staining was noted locally. Sandstone beds and lenses were present locally, as well.

Weathered Sandstone: Fine grained, clayey, low to medium plastic, moderately hard, moist, and gray in color. Iron oxide staining was noted commonly.

Sandstone: Fine grained, clayey, low to medium plastic, very hard, moist, and gray in color. Iron oxide staining was noted commonly.

Limestone: Finely crystalline, thinly to moderately bedded, very hard, slightly moist, and white to pale yellow to pale green in color. Shale beds were present locally.

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

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

Practical Rig Refusal

0

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

GROUND ENGINEERING CONSULTANTS

LEGEND AND NOTES

JOB NO.: 16-0011	FIGURE: 8
CADFILE NAME: 0011LEG01.DWG	

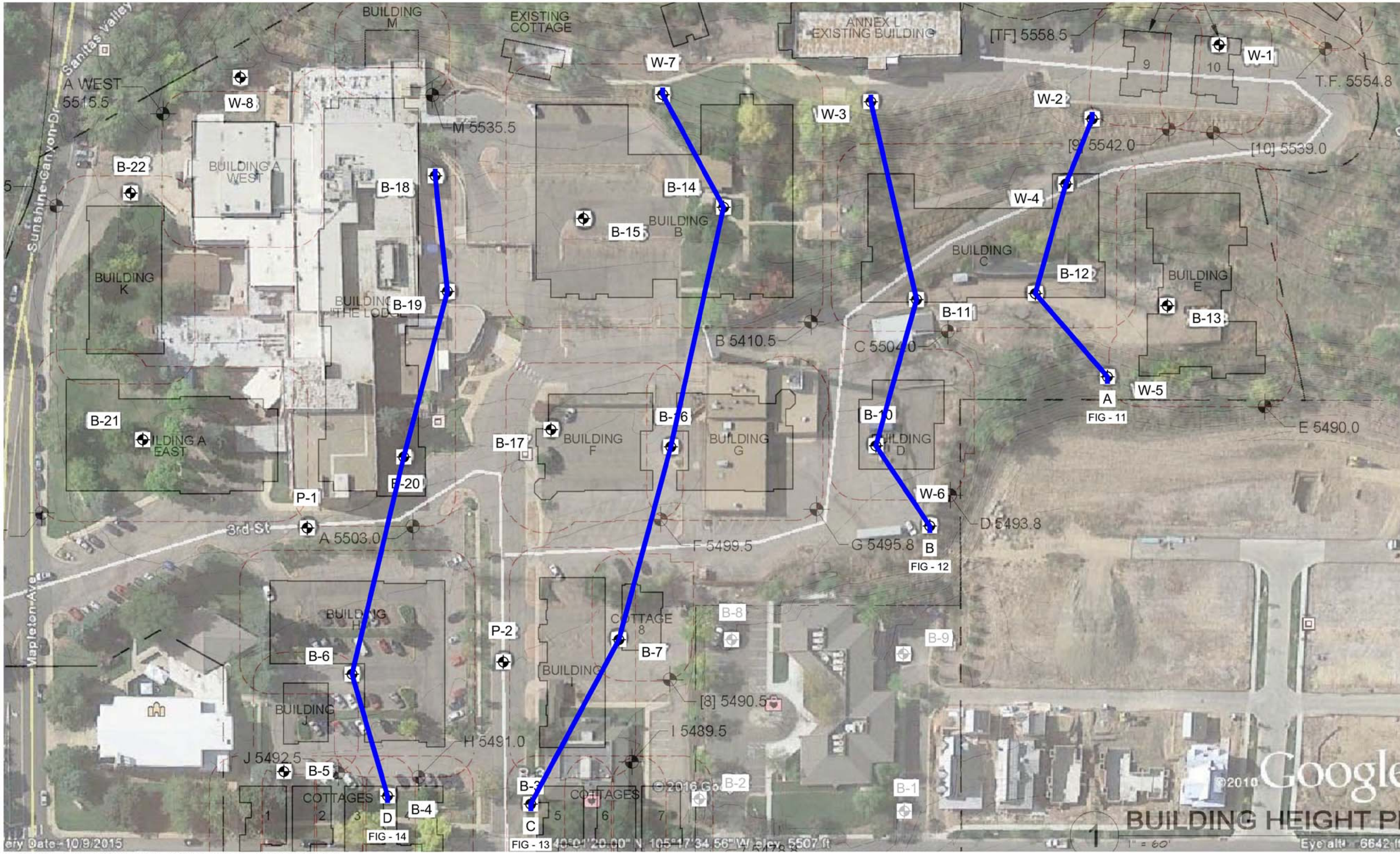
NOTES:

1) Test holes were drilled on 04/11, 04/12, 04/13, 04/14/2016 with 4-inch diameter continuous flight augers.
2) Locations of the test holes were measured approximately by pacing from features shown on the site plan provided.
3) Elevations of the test holes were surveyed by a representative of the client and the logs of the test holes are drawn to depth and hung to elevation.
4) The test hole locations and elevations should be considered accurate only to the degree implied by the method used.
5) The lines between materials shown on the test hole logs represent the approximate boundaries between material types and the transitions may be gradual.
6) Groundwater level readings shown on the logs were made at the time and under the conditions indicated. Fluctuations in the water level may occur with time.
7) The material descriptions on this legend are for general classification purposes only. See the full text of this report for descriptions of the site materials and related information.
8) All test holes were immediately backfilled upon completion of drilling, unless otherwise specified in this report.

GROUND ENGINEERING CONSULTANTS

LEGEND AND NOTES

JOB NO.: 16-0011	FIGURE: 9
CADFILE NAME: 0011LEG02.DWG	



GOOGLE EARTH AERIAL IMAGE (DATE UNKNOWN)
SITE PLAN PROVIDED BY CLIENT

Indicates approximate path of section lines.

1 Indicates test hole number and approximate location.

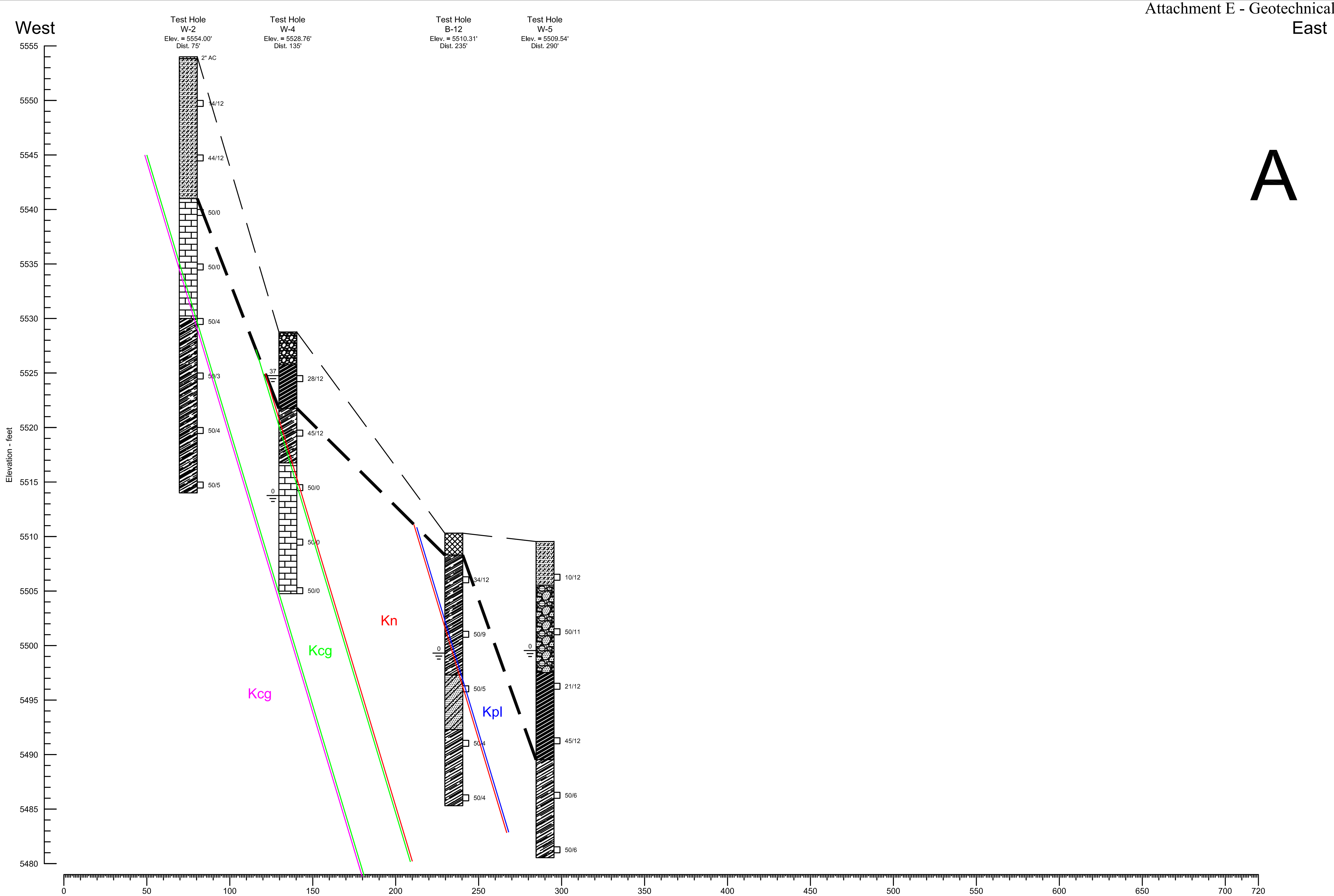
1 Indicates test hole number and approximate location. That were not drilled.

(Not to Scale)

GROUND ENGINEERING CONSULTANTS

SECTION LINES

JOB NO.: 16-0011	FIGURE: 10
CADFILE NAME: 0011SITE.DWG	



Kpl - lower member, Pierre Shale

Kn - Niobrara Shale

Kcg - Greenhorn Limestone

Kcg - Carlisle Shale

Schematic ground surface.

Indicates approximate interpolated top of bedrock.

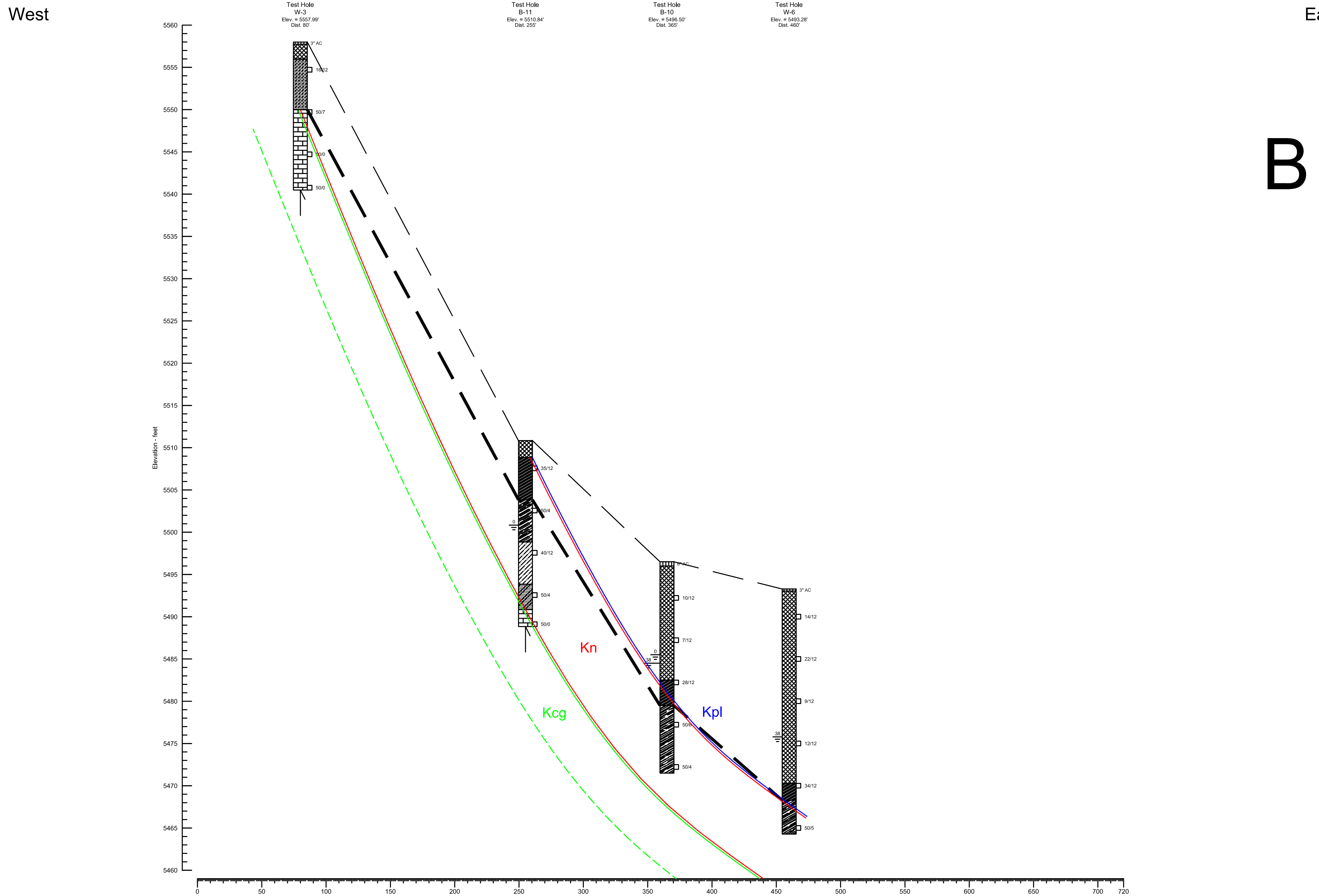
Note:

Contacts between geologic units are approximate and interpolated between and extrapolated beyond the test hole locations.

GROUND ENGINEERING CONSULTANTS

INTERPRETED GEOLOGIC SECTION

JOB NO.: 16-0011	FIGURE: 11
CADFILE NAME: 0011FENCE.DWG	



Kpl - lower member, Pierre Shale

Kn - Niobrara Shale

Kcg - Greenhorn Limestone

Kcg - Carlisle Shale

Schematic ground surface.

Indicates approximate interpolated top of bedrock.

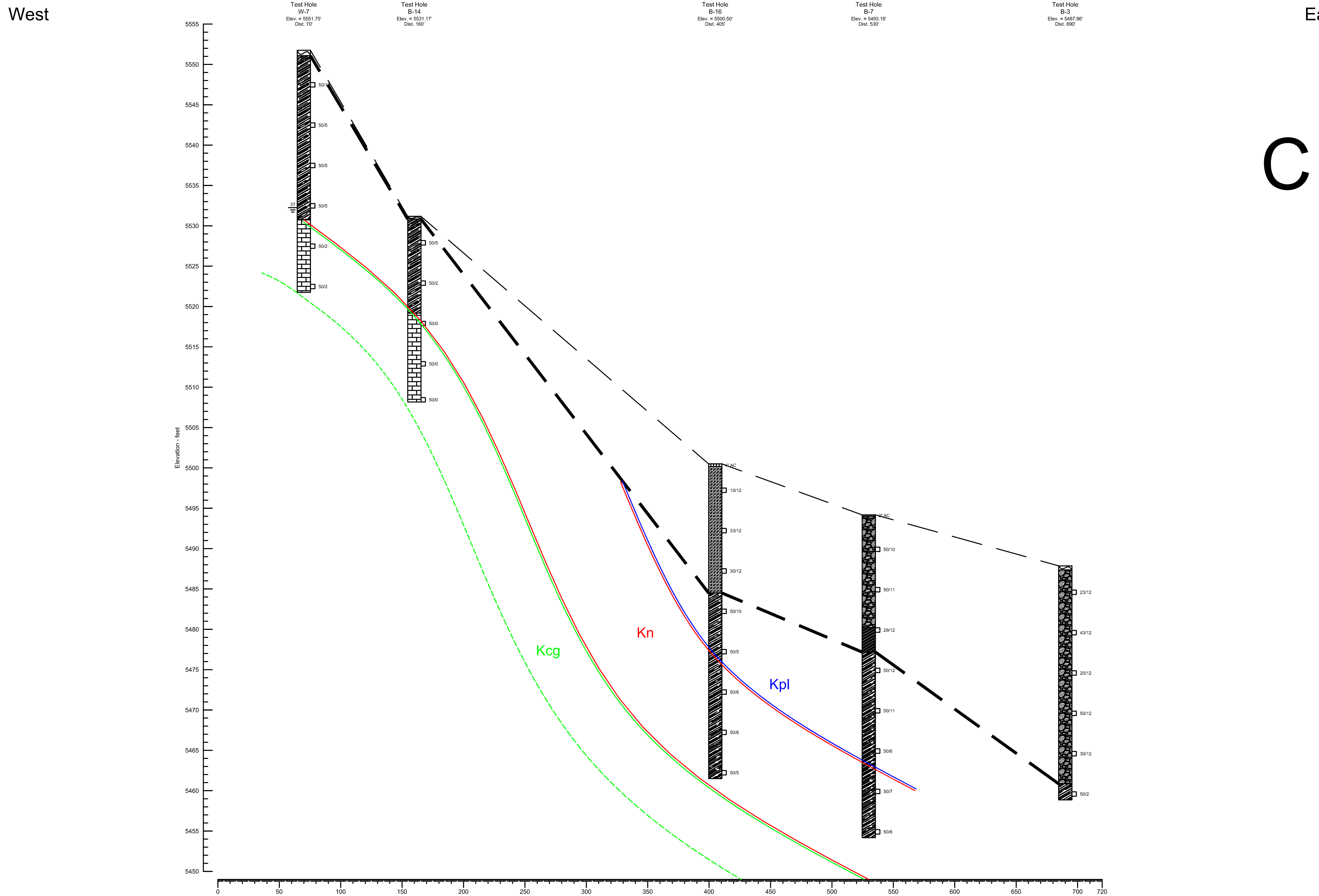
Note:

Contacts between geologic units are approximate and interpolated between and extrapolated beyond the test hole locations.

GROUND ENGINEERING CONSULTANTS

INTERPRETED GEOLOGIC SECTION

JOB NO.: 16-0011	FIGURE: 12
CADFILE NAME: 0011FENCE.DWG	



Kpl - lower member, Pierre Shale

Kn - Niobrara Shale

Kcg - Greenhorn Limestone

Kcg - Carlisle Shale

Schematic ground surface.

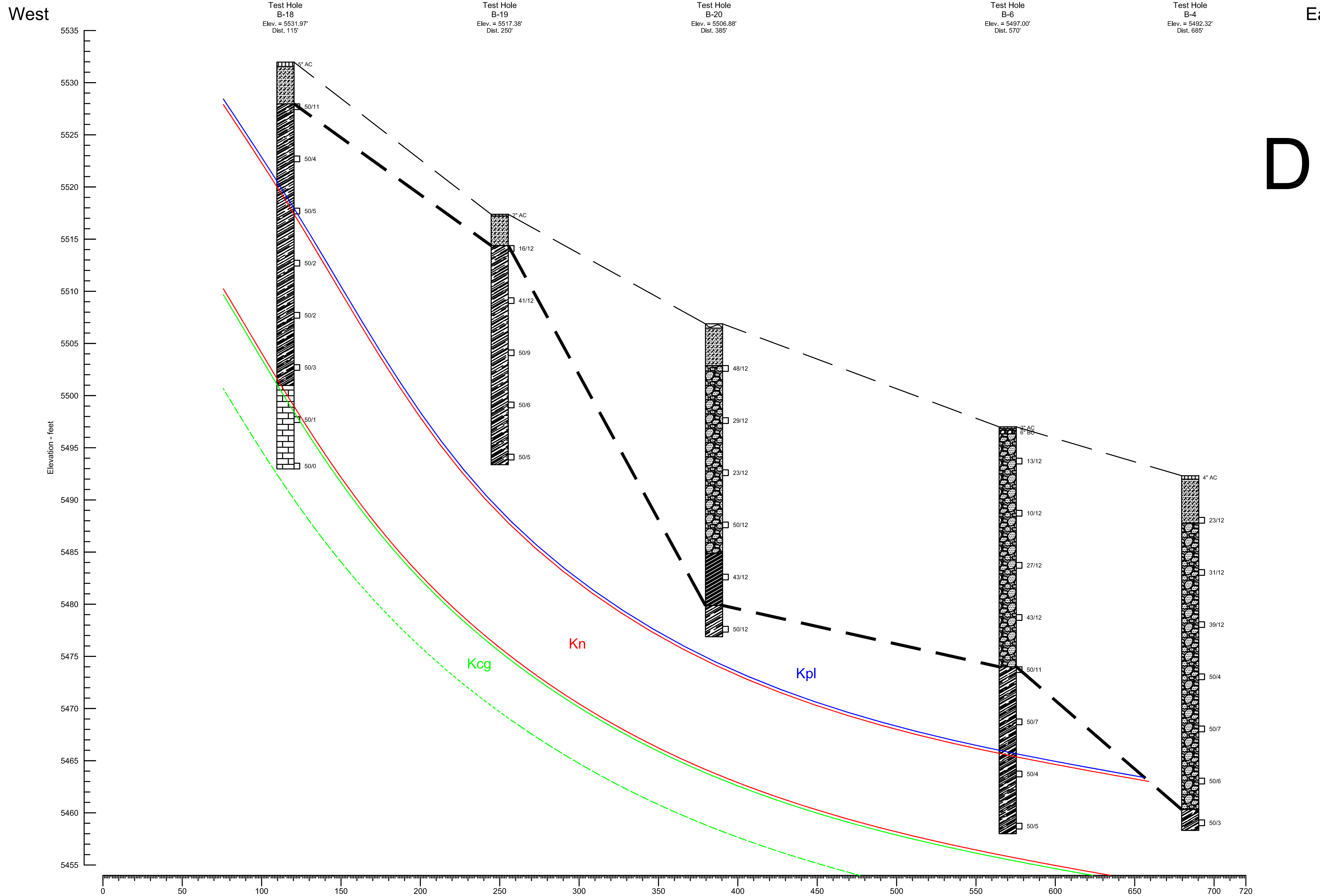
Indicates approximate interpolated top of bedrock.

Note:
Contacts between geologic units are approximate and interpolated between and extrapolated beyond the test hole locations.

GROUND ENGINEERING CONSULTANTS

INTERPRETED GEOLOGIC SECTION

JOB NO.: 16-0011	FIGURE: 13
CADFILE NAME: 0011FENCE.DWG	



Kpl - lower member, Pierre Shale

Kn - Niobrara Shale

Kcg - Greenhorn Limestone

Kcg - Carlisle Shale

Schematic ground surface.

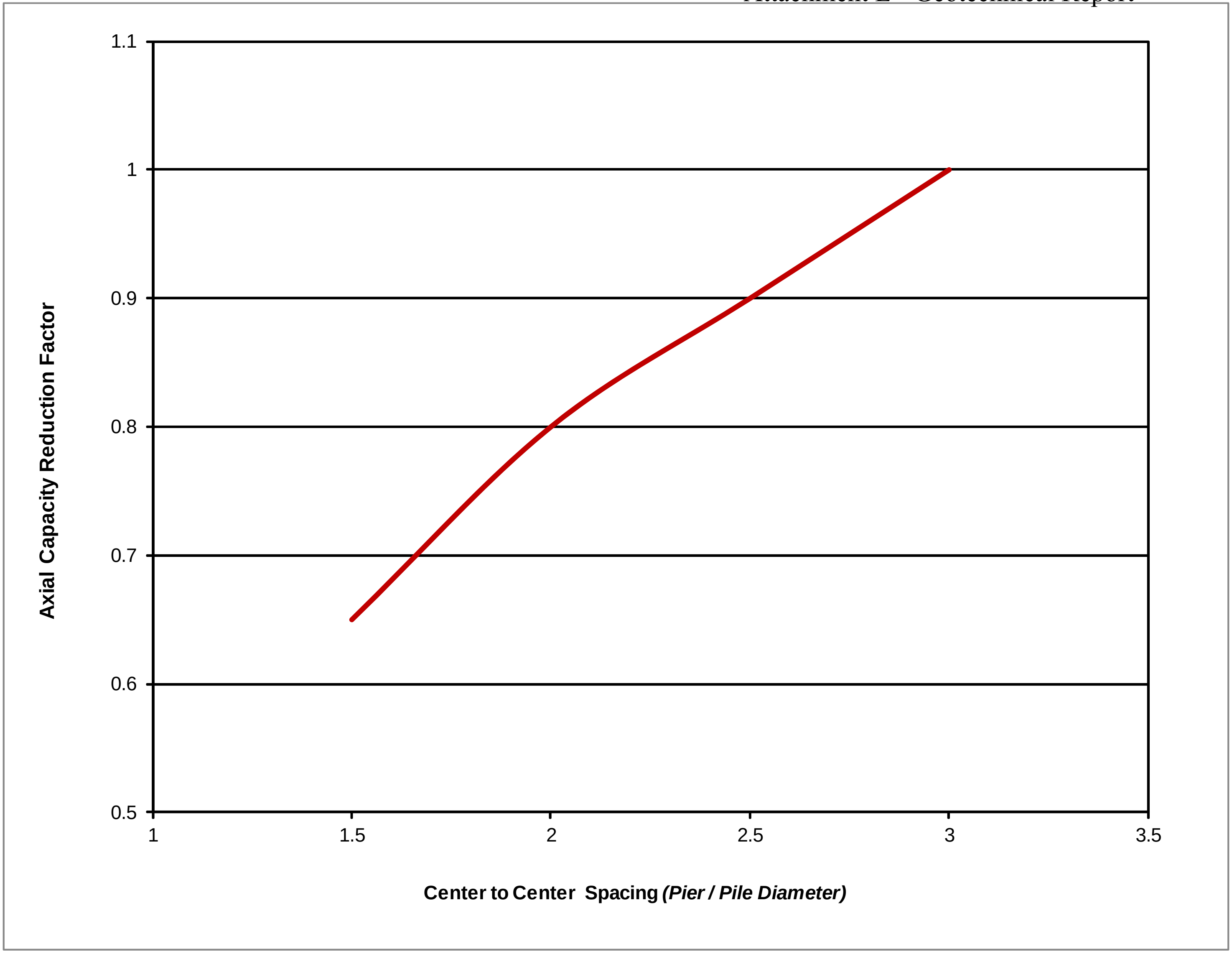
Indicates approximate interpolated top of bedrock.

Note:
Contacts between geologic units are approximate and interpolated between and extrapolated beyond the test hole locations.

GROUND
ENGINEERING CONSULTANTS

INTERPRETED GEOLOGIC SECTION

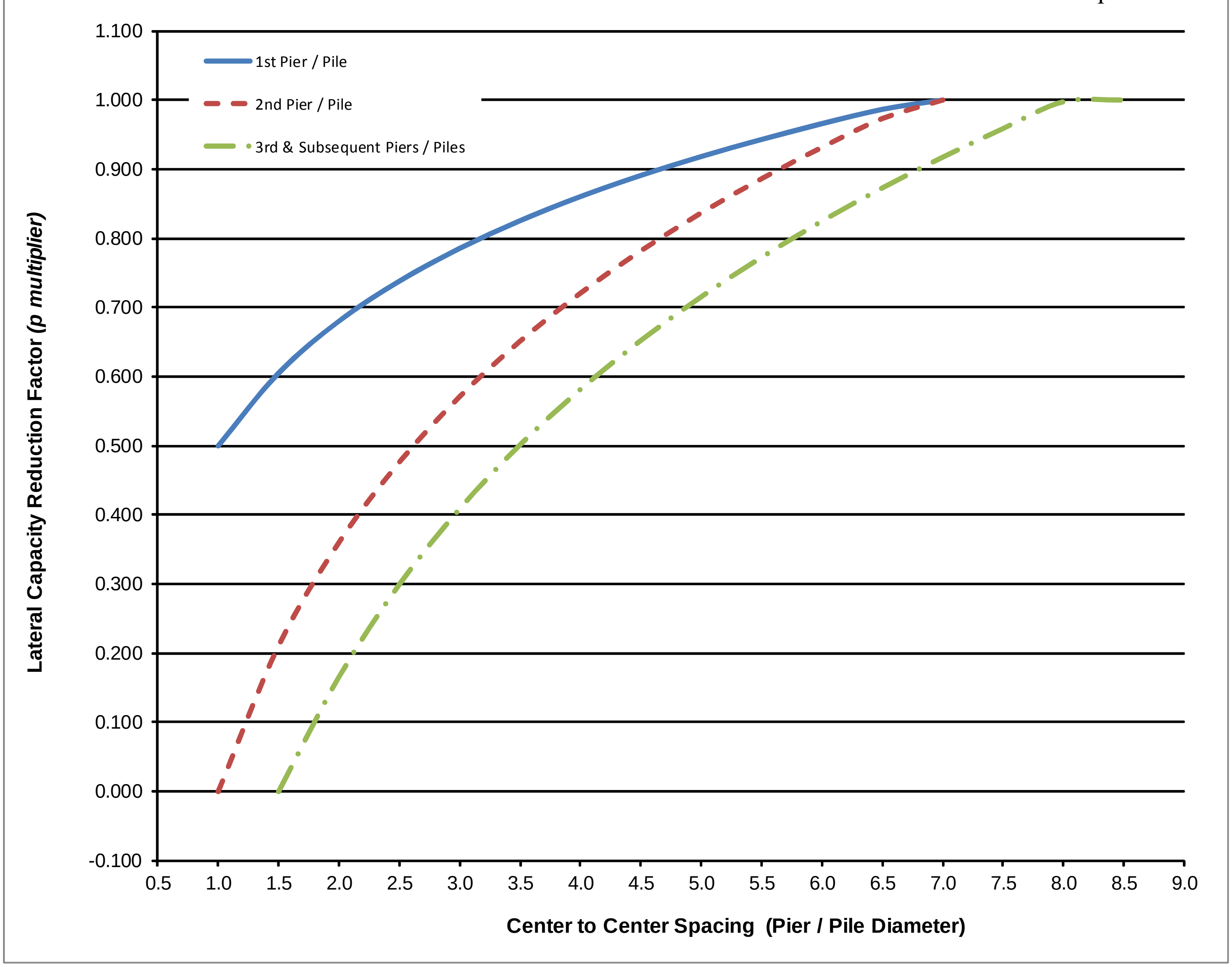
JOB NO.: 16-0011	FIGURE: 14
CADFILE NAME: 0011FENCE.DWG	

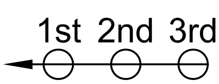


GROUND ENGINEERING CONSULTANTS

AXIAL CAPACITY
REDUCTION FACTORS
FOR CLOSELY SPACED PIERS / PILES

JOB NO.: 16-0011	FIGURE: 15
CADFILE NAME: 0011APR.DWG	

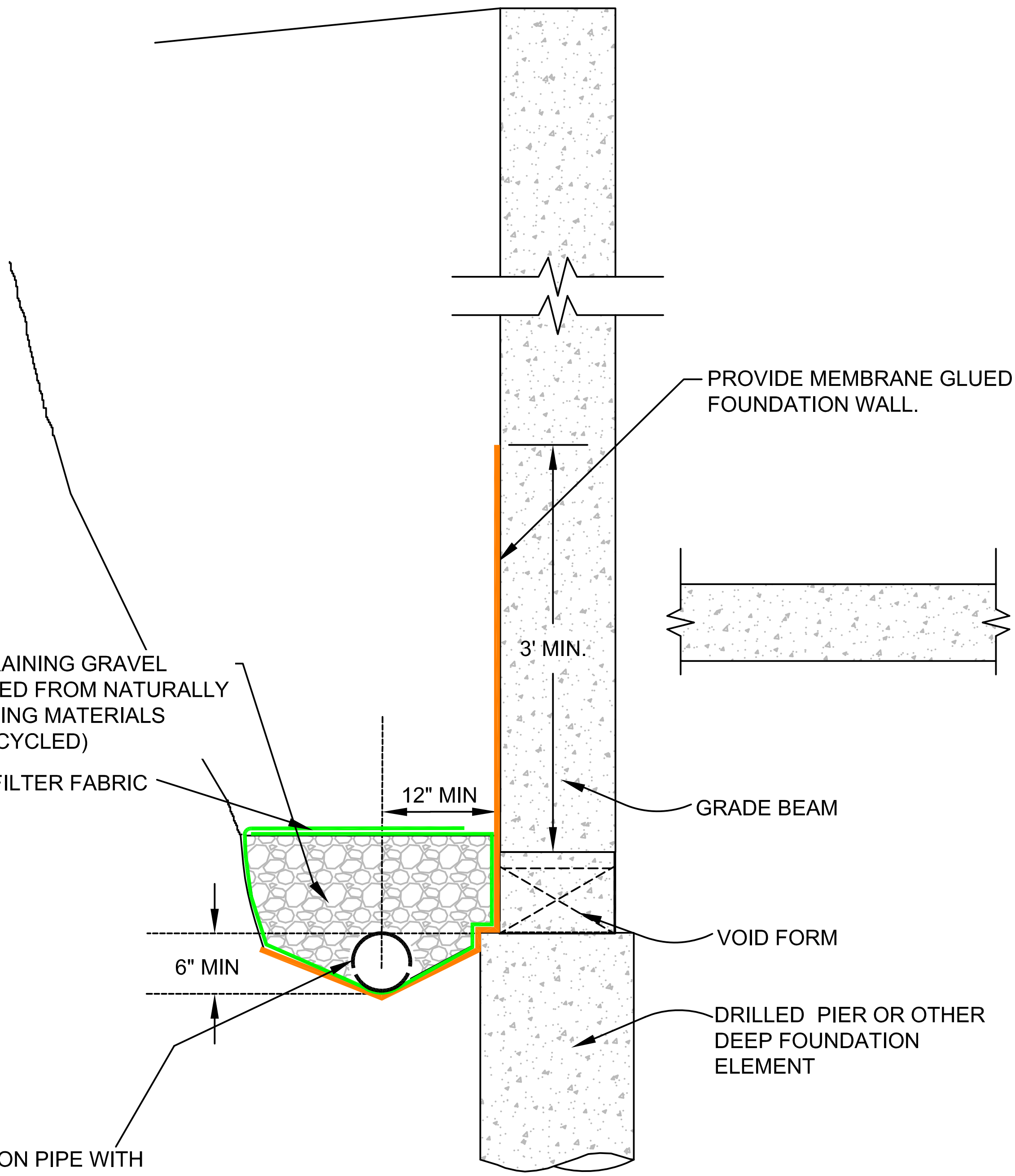




The “1st“ or “lead” pier / pile is the element that leads movement in the direction that the lateral load will cause the piers to deflect, as shown .

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.: 16-0011	FIGURE: 16
CADFILE NAME: 0011LLPR.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.

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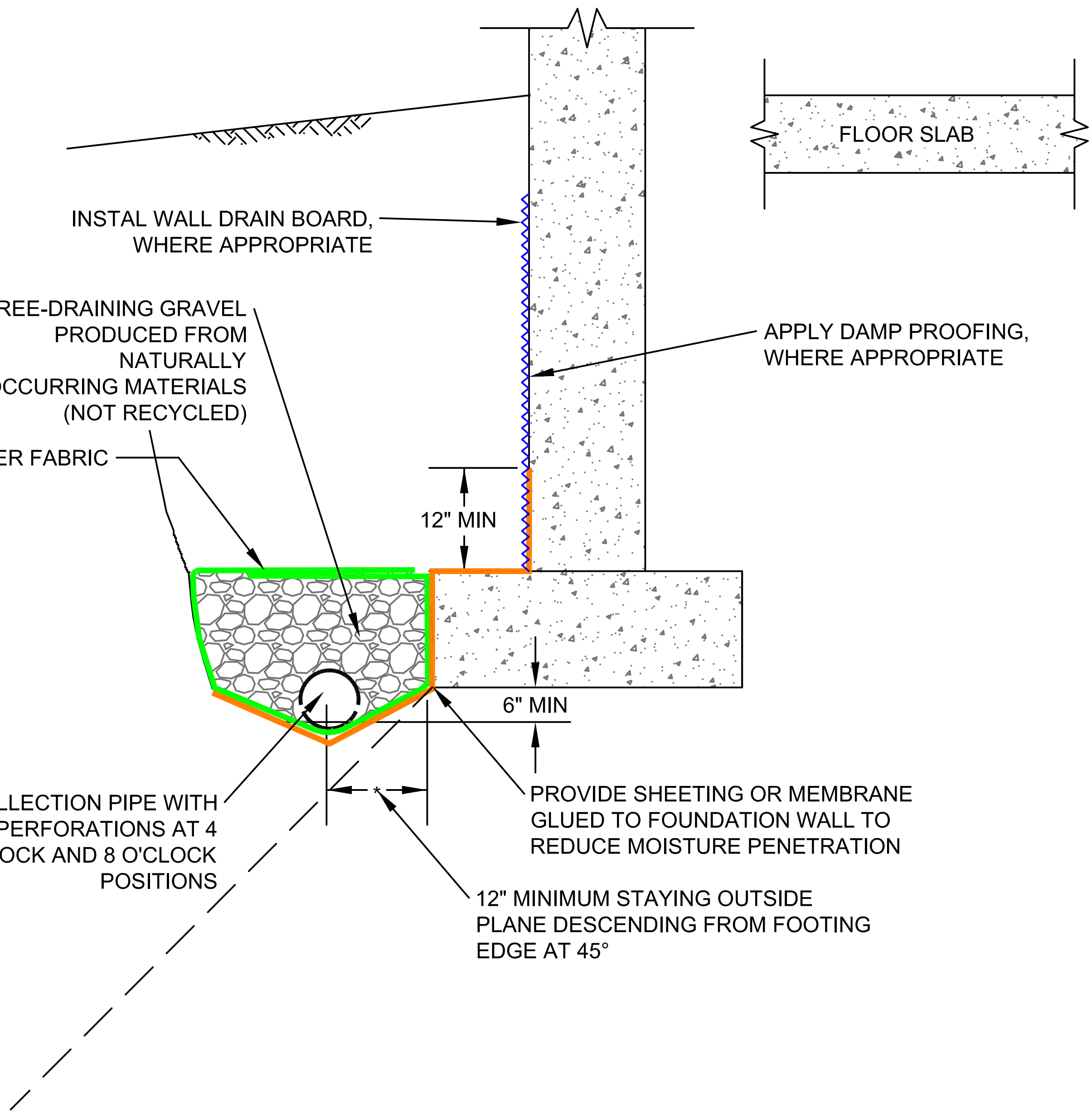
TYPICAL UNDERDRAIN DETAIL

JOB NO: 16-0011	FIGURE: 17
CADFILE NAME: 0011DRAIN01.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.
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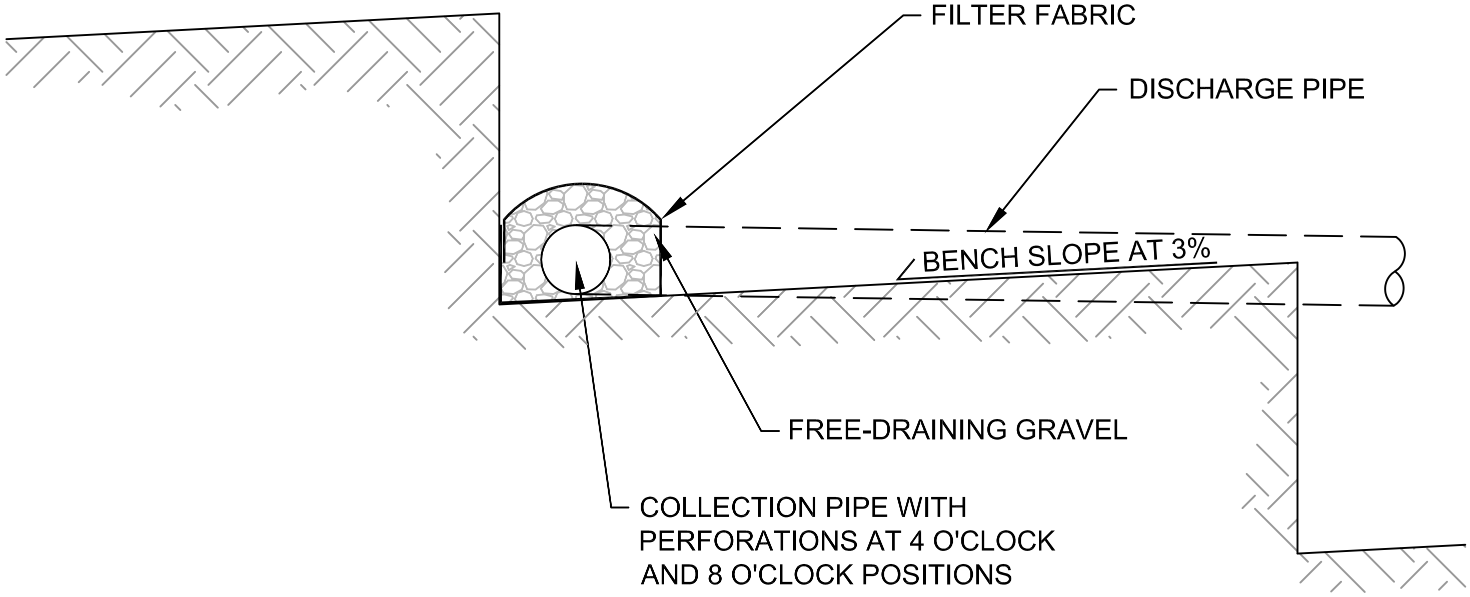
TYPICAL UNDERDRAIN DETAIL

JOB NO.: 16-0011	FIGURE: 18
CADFILE NAME: 0011DRAIN02.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.

(Not to Scale)

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TYPICAL BACK DRAIN DETAIL

JOB NO.: 16-0011	FIGURE: 19
CADFILE NAME: 0011DRAIN03.DWG	

GROUND
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TABLE 1
SUMMARY OF LABORATORY TEST RESULTS

Sample Location		Natural	Natural	Passing	Atterberg Limits		Percent Swell/Consolidation at Surcharge Pressure		Unconfined	USCS	Soil or Bedrock Type
Test Hole No.	Depth *(feet)*	Moisture Content *(%)*	Dry Density *(pcf)*	No. 200 Sieve *(%)*	Liquid Limit	Plasticity Index	%	Surcharge (psf)	Compressive Strength *(psf)*	Classification	
B-3	3	14.1	107.0	27	36	12	.			SC	Clayey SAND with Gravel
B-5	3	12.9	110.0	58	34	15				CL	Sandy CLAY
B-6	13	8.7	110.5	27	20	5				SM	Silty SAND
B-6	28	14.9	113.5	74	49	26			10,051	CL	CLAY SHALE Bedrock
B-7	14	18.7	105.2	75	37	18	+0.4	500		CL	Weathered CLAY SHALE
B-7	19	19.2	108.3	73	44	23	+0.4	1000		CL	CLAY SHALE Bedrock
B-7	24	14.0	117.2	87	40	19	-0.8	1500		CL	CLAY SHALE Bedrock
B-10	14	18.2	111.0	95	45	22	0.0	1000		CL	CLAY SHALE Bedrock
B-11	13	13.5	116.3	39	35	12	-0.8	500		SC	Weathered SANDSTONE
B-11	18	11.9	118.0	30	29	8	-1.1	1000		SC	SANDSTONE Bedrock
B-12	14	12.4	117.4	44	36	14	-2.1	1500		SC	SANDSTONE Bedrock
B-13	8	20.3	105.1	86	45	20				CL	Sandy CLAY
B-13	13	12.8	116.6	62	39	17				CL	CLAY SHALE Bedrock
B-13	18	13.7	116.5	82	38	13			15,899	CL	CLAY SHALE Bedrock
B-15	14	11.3	114.4	89	40	18	+5.1	500		CL	CLAY SHALE Bedrock
B-15	19	9.3	118.6	85	47	25	+5.1	1000		CL	CLAY SHALE Bedrock
B-16	13	17.6	111.8	74	42	21				CL	Sandy CLAY
B-16	18	14.9	114.9	88	43	19	-0.9	1000		CL	CLAY SHALE Bedrock
B-16	23	10.5	121.0	86	46	25	+1.6	1500		CL	CLAY SHALE Bedrock

GROUND
ENGINEERING CONSULTANTS

TABLE 1
SUMMARY OF LABORATORY TEST RESULTS

Sample Location		Natural	Natural	Passing	Atterberg Limits		Percent Swell/Consolidation at Surcharge Pressure		Unconfined	USCS	
Test Hole No.	Depth *(feet)*	Moisture Content *(%)*	Dry Density *(pcf)*	No. 200 Sieve *(%)*	Liquid Limit	Plasticity Index	%	Surcharge (psf)	Compressive Strength *(psf)*	Classifi-cation	Soil or Bedrock Type
B-17	13	19.3	109.2	86	46	23	+0.5	500		CL	Sandy CLAY
B-18	14	8.4	114.3	91	38	21			15,007	CL	CLAY SHALE Bedrock
B-18	29	11.5	118.9	74	33	13	-1.5	1500		CL	CLAY SHALE Bedrock
B-19	8	18.3	110.2	92	47	22	+0.6	1000		CL	CLAY SHALE Bedrock
B-19	13	14.3	117.8	86	39	15	+0.6	1500		CL	CLAY SHALE Bedrock
B-19	19	11.0	120.6	74	39	18	-0.7	2000		CL	CLAY SHALE Bedrock
B-20	14	13.0	111.5	31	29	13				SC	Clayey SAND
B-22	9	7.9	-	18	27	11				SC	Clayey SAND
W-2	9	22.7	98.0	56	39	15				SC	Sandy CLAY
W-3	3	21.2	102.5	86	39	17				CL	Sandy CLAY
W-5	3	-	91.8	40	38	15				SC	Clayey SAND
W-6	8	18.0	107.3	38	42	19				SC/GC	Fill: Clayey SAND with Gravel
W-7	9	14.7	100.3	88	45	18				CL	CLAY SHALE Bedrock
W-8	8	14.0	116.5	86	39	15				CL	CLAY SHALE Bedrock
P-1	3	11.1	123.2	30	37	16				SC/GC	Clayey SAND with Gravel

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TABLE 2
SUMMARY OF LABORATORY TEST RESULTS, CONTINUED

Sample Location		Water-Soluble Sulfates *(%)*	Resisitivity *(ohm-cm)*	pH	Redox Potential *(mV)*	Sulfide Reactivity	Soil or Bedrock Type
Test Hole No.	Depth *(feet)*						
B-3	3	< 0.01	9,385	9.6	-213	Trace	Clayey SAND with Gravel
B-6	13	0.06	2,628	8.0	-118	Trace	Silty SAND
B-13	13	0.02	8,132	8.2	-124	Positive	CLAY SHALE Bedrock
B-16	13	0.02	1,886	7.7	-98	Trace	Sandy CLAY
B-20	14	0.01	3,735	8.4	-136	Trace	Clayey SAND
P-1	3	0.06	2,322	7.6	-92	Trace	Clayey SAND with Gravel

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