GROUND ENGINEERING

Preliminary Subsurface Exploration Program
Woodmen and Meridian Development
Colorado Springs, Colorado

Prepared for:
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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 Woodmen and Meridian development that will include approximately 35.7 acres of retail/commercial development to be located near the intersection of Woodmen Road and Meridian Road in Colorado Springs, Colorado. Our study was conducted in general accordance with Agreement between Evergreen Devco, Inc. and GROUND dated July 29, 2015 and GROUND's Proposal Number 1507-1490 dated July 20, 2015.

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

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

When final site grading, utility/roadway alignments, and applicable structure types, location(s), and dimensions are known, improvement-specific, final geotechnical subsurface exploration programs must be performed in order to confirm the preliminary design parameters provided as well as to provide additional, detailed geotechnical design information. This preliminary report, presented herein, should not be used for design purposes. Pavement design for private internal drives area provided in this report and should be reevaluated upon completion of rough final surface grades.

PROPOSED CONSTRUCTION

We understand that proposed construction may ultimately include a retail/commercial development. Based on the conceptual site plan dated July 1, 2015, we understand that proposed construction may consist of retail/commercial facilities, internal drives, and associated parking areas in Lots 1 through 4. The remainder of the project site is proposed for a detention pond and future development area.

The installation of numerous underground utilities will also be included in the project scope. Grading plans were not available at the time of this report preparation. Based on site topography, material cuts/fills up to approximately 10 feet or more may be planned for the site. Once updated grading information is available, we should be notified to review and re-evaluate the preliminary parameters provided herein, as necessary.

SITE CONDITIONS

The project area was generally undeveloped with the exception of a residential structure and small barn/shed structures associated with the northeast corner of the development. The project site's surface generally consists of short to tall grasses and weeds. A drainage ditch traverses the project site and contained some standing water at the time of our exploration program. Additionally, standing water was observed within Tract A and C which prohibited drill rig access in this area. Drainage/culvert structures were associated with Woodmen Drive and the drainage ditch. The project site was bordered by a single-family residential development to the west, Meridian Road and a residential development to the east, Woodmen Drive and a commercial facility to the south, and vacant land to the north.

Topographically, the project site was gently sloping at about 2 percent descending toward the southeast. The maximum elevation difference across the site was approximately 30 feet.

GEOLOGIC SETTING

Regional Geology

The geological history of Colorado Springs began with the formation of the Pikes Peak Granite in the Precambrian age. The ocean levels steadily transgressed until the area was a warm shallow beach environment in the Mississippian age. During formation of the Ancestral Rockies around the time period of 300 million years ago, Colorado Springs was above the ocean surface. During the following 240 million years or so, the area was periodically submerged and reemerged, becoming deserts, beaches, and relatively deep coastal floors. This trend was changed during the middle Cretaceous period when the average sea level was unusually high. This allowed the Pierre Shale to slowly be deposited. Around 65 million years ago, the Laramide Orogeny occurred rearranging the orientation of all prior rock formations. While uplifting of the Rocky Mountains was underway, streams began depositing sediments derived from the Rocky Mountains within the Colorado Springs area. This formed a sequence of sedimentary rock formations including the Dawson Formation encountered beneath the site. In the general project area, these sedimentary rocks dip eastward at low angles (less than 10 degrees, typically). Over time, fluctuating water levels, tectonic activity, and erosion have shaped the landscape of the Colorado Springs area.

Site Geology

Published maps, e.g. Scott, Taylor, Epis, and Wobus (1976[1]), depict the site as underlain by the Holocene to late Pleistocene Eolian Sand (**Qes**). These materials consist of fine to coarse grained silty sand deposited by wind and preserved on surfaces to the east of the I-25 corridor. These materials are underlain by the Upper Cretaceous and Paleocene Dawson Formation (**Tkd**). The Dawson Formation consists of light-gray to greenish-gray arkosic sandstone and olive-green to brownish gray, pebbly, andesitic sandstone interbedded with dark-gray to grayish-green claystones and siltstones.

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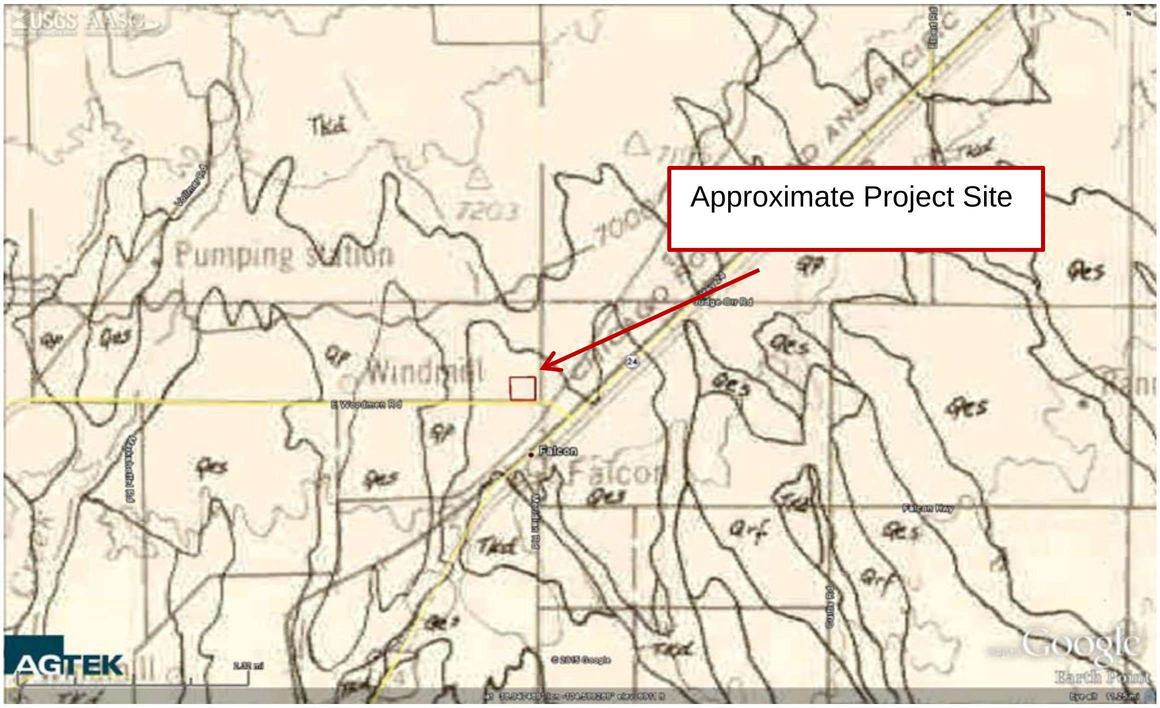
[1] Scott, Glenn, R., Taylor, Richard B., Epis, Rudy C., and Wobus, Reinhard A. 1976, Geologic Map of the Pueblo Quadrangle, S. Central Colorado, Miscellaneous field studies Map MF-775.



GEOLOGIC HAZARDS

Expansive Soils As discussed in the *Subsurface Conditions* section of this report, the shallow earth materials underlying the site include sand, clay, sandstone and claystone/siltstone bedrock. Swelling clayey soils are often present in the general area and change volume in response to changes in moisture content that can occur seasonally, or in response to changes in land use, including development. Expansion potentials vary with moisture contents, density and details of the clay chemistry and mineralogy. The swell potential in any particular area can vary markedly both laterally and vertically due to the complex interbedding of the site soil and bedrock materials. Moisture changes also occur erratically, resulting in conditions that cannot always be predicted.

Swell-consolidation testing indicated a potential for heave (See Table 1) in the tested on-site materials. Laboratory testing on selected samples indicated a swell of approximately 0.5 percent when wetted against various surcharge pressures.

Although there is always risk involved where structures are placed on these types of soils, with appropriate consideration given to geotechnical factors and appropriate design that is properly implemented during construction, the proposed development is feasible with regard to expansive earth materials. It is important that the soil conditions be reviewed on an individual basis once final structure/improvement layouts are known as expansive materials are likely present in these areas.

Collapsible Soils Certain surficial deposits in the Colorado Springs area, typically eolian (wind-blown) materials including loess, are known to be susceptible to local hydro-consolidation or “collapse.” Hydro-consolidation consists of a significant volume loss due to re-structuring of the constituent grains of the soil to a more compact arrangement upon wetting under a surcharge load.

Site surficial soils are interpreted to be eolian materials. Based on our laboratory testing program, consolidations of approximately 0.1 percent were measured under various surcharge pressures. Greater consolidations may be possible in the site soils. Additionally, variable standard penetration resistance values were determined from our field exploration. Design-level geotechnical evaluations of individual building sites, roadway alignment, etc. should include an assessment of the possible presence of collapsible materials in the foundation soils, so that appropriate, remedial design and construction can be implemented, if necessary.

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

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

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

The nearest potentially active faults to the site are the Rampart Range Fault and the Ute Pass Fault, which is mapped approximately 15 miles and 21 miles, respectively, to the west. The Rampart Range Fault trends north-south along the Front Range, north of

Colorado Springs. It is a range-front fault that experienced reverse movement during the Laramide, but normal movement during the late Cenozoic. The Ute Pass fault zone is defined by a series of five generally northwest-trending faults west of Colorado Springs. It is highly weathered and the result of a huge piece of exposed Pikes Peak Batholith. being thrust upward and outward over the sedimentary strata to the east. Detailed seismic studies on the faults concluded that there was no evidence to indicate the faults have moved since between 600 and 30 to 50 thousand years ago (Rogers, et. al., 1998[2]). Because these fault systems have not reported movement within the Holocene time period, the risk of the faults giving rise to damaging, earthquake-induced ground motions at the site is considered to be relatively low.

The largest recorded earthquake (estimated magnitude 6.2 to 6.6) in Colorado occurred in November 1882. While the specific location of this earthquake is very uncertain, it is postulated to have occurred in the Front Range near Rocky Mountain National Park. The most recent significant seismic movements associated with the Rock Mountain Arsenal Fault (Commerce City, Colorado) occurred in the 1960s, generating earthquakes up to magnitude 5.5. Since the early 1960s, numerous earthquakes with magnitudes up to approximately 5, with the majority possessing magnitudes of 2 to 4, have been experienced within the State. Recently, earthquakes ranging in magnitude from 3.7 (Craig, Colorado) to 3.9 (Eads, Colorado and Trinidad, Colorado) occurred during the time period of July, 2009 through August, 2009. Earthquakes with similar magnitudes, and potentially greater, are anticipated to continue by the USGS, throughout the State.

In accordance with the 2012 International Building Code®, it is GROUND's opinion that Seismic Design Category B may be applicable for seismic foundation design, based on an Occupancy Category of I, II, or III. For Occupancy Category IV, a Seismic Design Category C would be applicable. The Project Structural Engineer should ultimately determine the Seismic Design Category at the time of the foundation design. Compared with other regions of the Western United States, recorded earthquake frequency in the project vicinity is relatively low.

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[2] Rogers, W.P., Kirkham, R.M., and Widmann, B.L., compilers, 1998, Quaternary fault and fold database of the United States: U.S. Geological Survey website, http://earthquakes.usgs.gov/hazards/qfaults.

Based on the soil conditions encountered in the test holes drilled on the site, our review of applicable geologic maps, as well as our experience within the Project site vicinity, GROUND estimates that a Site Class D according to the 2012 IBC classification (Table 1613.3.2) could be considered for seismic foundation design. This parameter was estimated utilizing the above-referenced table as well as extrapolation of data beyond the deepest depth explored. In the event the Client desires to potentially utilize Site Class C for design, according to the 2012 IBC, actual seismic shear wave velocity testing will be required. A proposal for this work can be provided upon request.

Slope Stability and Erosion Colton and others (1975[3]), as well as larger scale geologic maps providing coverage of the site that were reviewed for this study, did not depict landslide deposits on or adjacent to the subject site.

As noted in the *Site Conditions* section of this report, the site is relatively flat to gently sloping in some areas. During our preliminary reconnaissance of the site area, no evidence was noted of mass-wasting processes associated with steep slopes, such as landslides, slumps or unusual soil creep. Therefore, the likelihood of project developments being affected by existing large scale unanticipated slope instabilities is considered low.

Formational sandstone bedrock and overburden soils are vulnerable to erosion. Erosion potential is greater along drainage areas suseptible to high velocity storm water flows and across sparsely vegetated terrain. Mitigation measures should be used to limit erosional damage to the soils as a result of increased surface water runoff. Installation of drainage culverts, riprap, or other measures to alleviate concentrated surface runoff should be considered during design and construction of the project.

Preliminarily, un-retained, permanent slope cuts should be less than 10 feet in height and maintain a maximum 3:1 (horizontal : vertical) slope angle or less with proper erosion control measures implemented. Proper surface drainage controls to reduce the potential for erosional slope damage need to be implemented in the grading design to control runoff, which may be increased due to proposed pavement surfaces, structures

[3] Colton, R.B., J.A. Holligan, and L.W. Anderson, 1975, *Preliminary Map of Landslide Deposits, Denver 1 Degree x 2 Degree Quadrangle*, Colorado, U.S. Geological Survey, Miscellaneous Field Studies Map, MF-705

and landscape irrigation. Re-vegetation or other means of protection should be used on graded slopes.

Flooding A drainage traverses the center of the project site. The area associated with the drainage is located within the 100-year floodplain area (Zone A) per FEMA[4]. The remainder of the project site is designated as Zone X and not depicted as lying in a flood prone area. Therefore, the portion of the site associated with the drainage area appears to be vulnerable to flooding. The remainder of the site may be vulnerable to flooding during episodes of heavy rainfall and associated temporary ponding of run-off in areas of relatively slow surface drainage.

Wetlands Potential According to the U.S. Fish and Wildlife Service[5], the project site is not designated as a wetland area. During site development all regulations concerning wetland protection, as well as any other areas designated as wetlands by the Federal Wetlands Protection Act should be adhered to. Explicit designation of wetlands was not included as part of the scope of this study.

Mining Activity and Subsidence Review of U.S. Geological Survey topographic maps covering the site (e.g., U.S.G.S. 1965, revised 1994) and Jones, and Turney and Murray-Williams (1983) and other available, published maps depicting areas of coal extraction, did not indicate past mining activities on or adjacent to the subject parcel. Additionally, no indications of mining activities were apparent on the site during the site reconnaissance. Therefore, there appears to be little potential for surface subsidence associated with consolidation of former mine workings at depth.

Published geologic maps do not indicate formations underlying the site at shallow depths that include evaporite (salt, gypsum, etc.) deposits, limestones or other materials vulnerable to subsurface dissolution. Therefore, the likelihood of subsidence or other mining-related hazards appears to be low.

In review of the City of Colorado Springs requirements for a geologic hazard study and published information reviewed for the site, the site appears to be feasible for

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[4] FEMA, September, 28 1982, Weld County, CO
[5] U.S. Fish and Wildlife Service, National Wetlands Inventory, May 20, 2010

development with respect to potential geologic hazards and general geotechnical design concerns.

SUBSURFACE EXPLORATION

Field reconnaissance and subsurface exploration for the project was conducted on July 30 and 31, 2015. A total of thirteen (13) test holes were drilled with truck and buggy-mounted drill rigs advancing continuous flight augers to evaluate subsurface conditions, including depths to groundwater and bedrock, and to retrieve samples for laboratory testing and analyses. Of these, nine (9) test holes were drilled at locations indicated on the Client-provided site plan for preliminary purposes and four (4) test holes were drilled within the proposed drive lanes. The test holes were drilled to depths ranging from approximately 5 to 40 feet below existing grade. Two of the test holes (Test Holes 4 and 7) were completed as temporary groundwater observation points. A representative of GROUND directed the subsurface exploration, logged the test holes in the field, and prepared the soil and bedrock samples for transport to our laboratory.

Two of the test holes (Test Holes 1 and 3) were offset to the east due to the wet conditions within Tracts A and C. It should be noted that the standing water conditions within these tracts prohibited access with truck and buggy-mounted rigs.

Samples of the subsurface materials were retrieved from the test holes with a 2-inch inside diameter 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.

The approximate locations of the test holes are shown in Figure 1. The interpolated elevation of bedrock and groundwater maps is provided on Figures 1A and 1B. Logs of the exploratory test holes and temporary groundwater observation points are presented in Figures 2 through 6. Explanatory notes and a legend are provided in Figure 7. The test holes were staked in the field by a representative of GROUND utilizing a hand-held GPS device and an overlay of the site plan in Google imagery. The elevations of the test holes were approximated utilizing the Client-provided ALTA survey.

LABORATORY TESTING

Samples retrieved from our test holes were examined and visually classified in the laboratory by the project engineer. Laboratory testing included standard property tests such as natural moisture contents, dry unit weights, grain size analyses, and liquid and plastic limits. Unconfined compressive strength, swell-consolidation testing, direct shear testing, water-soluble sulfates, pH, and corrosivity testing were performed on select samples as well. Proctor and resilient modulus testing were performed on collected bulk samples. Laboratory tests were performed in general accordance with applicable ASTM and AASHTO protocols. Results of the laboratory testing program are summarized on Tables 1 and 2.

SUBSURFACE CONDITIONS

Our interpretation of the subsurface conditions is based on the results of the field exploration and laboratory testing, and our experience with the general geology of the area. Generally, native, overburden soils consisting of sands and clays were encountered beneath the surface underlain by sandstone and claystone/siltstone bedrock encountered at depths ranging from approximately 8 to 14 feet (elevations of approximately 6,869 to 6,890 feet) below existing grades. The test holes extended to depths ranging between approximately 5 to 40 feet below surface grades. Topsoil[6]-like material was also observed on the surface in various areas across the site.

The following sections provide generalized descriptions of the materials encountered. Additional detail is presented in Figure 7.

Sands and Clays were interbedded, fine to coarse grained, low to medium plasticity, very loose to medium dense/soft to very stiff , moist to wet, and brown in color.

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[6] '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.

Sandstone Bedrock was generally clayey to silty, fine- to coarse-grained with gravel, non- to low plastic, moderately hard to very hard, weakly cemented, moist to wet, and pale brown to grayish brown in color. Highly cemented sandstone may be encountered in localized areas across the site. This material caved at various depths in the test holes during our exploration program.

Sandstone and Siltstone/Claystone Bedrock were interbedded, fine to medium grained, non-plastic to medium plastic, hard to very hard, moist to wet, occasionally iron stained, and olive to gray to brown in color.

Groundwater was encountered in the test holes at depths between approximately 2 and 20 feet at the time of drilling. After 5 or 7 days, groundwater was measured in Test Holes 4 and 7 at depths ranging from approximately 7 and 13 feet, respectively. Groundwater levels will fluctuate, and most likely raise, in response to annual and longer-term cycles of precipitation, irrigation, snowmelt, surface drainage and land use, and the development and drainage of transient, perched water conditions.

Based on our observations of the project site, poor drainage measures were observed within Tract A and C. Prior to construction, positive surface drainage measures should be established to direct surface-generated water away from the project area. The Contractor and the Project Team should consider these complex conditions prior to commencing construction.

In areas where surface and subsurface drainage have not been properly designed or maintained, groundwater and/or wet conditions will be encountered. Therefore, shallow groundwater conditions, seepage through temporary and permanent cuts, and related soft and wet subgrade conditions should be anticipated by the Project Team and Contractors. Proper drainage measures should be employed during and after construction.

PRELIMINARY FOUNDATION/FLOOR SYSTEMS

Geotechnical Considerations for Design

Based on the results of our field and laboratory testing program, a potential for heave/consolidation is present in the site earth materials. Additionally, relatively low penetration resistance values were observed in the overburden materials. GROUND estimates that shallow foundations and floor systems placed directly on the on-site materials could experience 2 inches or more of movement (including differential and total movements). For the least potential for post-construction movement, the use of a deep foundation system consisting of drilled piers with a structural floor system should be utilized.

As an alternate foundation/floor system, shallow foundation systems consisting of spread footings with a slab-on-grade floor system placed on properly moisture-density treated materials may be considered. Due to the variable soil conditions, over-excavation and replacement of the on-site materials will likely be necessary to reduce the potential for movement to an owner acceptable level. We anticipate depths ranging from approximately 3 to 6 feet may be necessary beneath footings and depths ranging from approximately 6 to 9 feet may be necessary beneath slabs (beneath the underslab gravel layer). Actual fill prism depths should be further evaluated during final geotechnical investigation and upon review of final grading plans.

Additionally, shallow groundwater was encountered in our exploration program at depths as shallow as 2 feet below existing grade. Therefore, shallow groundwater levels and their seasonal variations must be considered in the establishment of lowest finished floor grades and/or design of permanent dewatering systems. In order to help accommodate the construction of a conventional shallow foundation system in some areas within the development, grades may have to be raised, on the order of 2 to 4 feet or more as practical (mainly to elevate the floor system), with temporary and/or permanent dewatering measures implemented.

As stated, an existing residential facility exists within the northeast corner of Lot 1. Therefore, proposed structures may be located within and beyond the extents of the existing residential structures. Below grade levels may be present in the existing residential structure. Demolition and proper excavation backfill (uniform fill prism) of the existing structures should be performed prior to new construction.

Below is a general discussion of potential foundation/floor systems within the project site. They are provided to assist in general overall project cost estimates but may not contain enough information for specific cost analysis. All discussions/parameters provided herein are subject to revisions and modifications after site-specific studies are performed. **Additionally, specific tenant requirements for corporate facilities should be provided during the final geotechnical studies for individual structures.**

Anticipated Foundation Systems

The design parameters provided below are preliminary. They are provided to assist in overall project cost estimates. All parameters provided herein are subject to revisions and modifications after site-specific studies are performed.

Drilled Pier Foundation System: For the least risk of post-construction movement, a deep foundation system should be used to support the proposed structures. This includes any attached building appurtenances. Commonly, along the Front Range area, deep foundations consisting of drilled piers advanced into the underlying formational bedrock are used to reduce potential structural movements as a result of heave (expansive materials) to an owner-acceptable level. As stated previously, building specific conditions will need to be identified, verified, and evaluated to provide final parameters.

Anticipated piers may be designed for allowable end bearing pressures of 30,000 to 50,000 psf and a skin friction of 3,000 to 5,000 psf for the portion of the pier penetrating competent bedrock. Piers will require an estimated minimum length of 30 feet or more, and minimum penetrations into competent bedrock of 10 feet or greater. Based on the depths to bedrock encountered at the site (approximately 8 to 14 feet below existing grades), pier lengths of 30 feet or more below proposed final grades may be needed. The actual pier lengths, however, should be based on the design loads, etc., as determined by the structural engineer following site-specific geotechnical explorations. As previously mentioned, shallow groundwater present at the site would likely require remediation (i.e. dewatering, drilled pier casing, etc.) to facilitate construction.

Spread Footing Foundation System: In general, structures underlain by properly moisture-density treated materials could be founded on shallow foundation systems designed for an allowable soil bearing pressure of 2,000 psf. Spread footings should have a minimum footing dimension of 14 or more inches. Actual footing dimensions, however, should be determined by the Structural Engineer, based on the design loads. Final geotechnical exploration in order to confirm foundations must be performed prior to final design.

As stated, in certain areas within the site, grades will likely have to be raised, on the order of 2 to 4 feet or more as practical (mainly to elevate the floor system), additional subgrade improvements performed, with temporary and/or permanent dewatering measures implemented.

Anticipated Floor Systems

The design parameters provided below are preliminary. They are provided in order to assist in overall project cost estimates. All design parameters provided herein are subject to revisions and modifications after a site-specific study(s) is performed.

Structural Floor System: Structural floors should be supported on grade beams and straight-shaft drilled piers. Requirements for the number and position of additional piers to support the floors will depend upon the span, design load, and structural design, and should be developed by the structural engineer. Structural floors should be constructed to span above a well-ventilated crawl space permitting utility lines to be installed above the soils and bedrock. The crawl space should be adequate to allow access and maintenance to utility piping. Piping connections through the floor should allow for differential movement between the piping and the floor system.

Slab-on-Grade Floor System: The on-site materials, exclusive of topsoil, vegetation, and any deleterious materials, are suitable to support lightly to moderately loaded slab-on-grade construction, provided they are properly moisture-density treated to a depth determined following a final geotechnical evaluation.

WATER-SOLUBLE SUFATES

The concentrations of water-soluble sulfates measured in selected samples retrieved from the test holes was less than 0.01 percent by weight. (See Table 2) Such concentrations of water-soluble sulfates represent a negligible degree of 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 (2011). The Colorado Department of Transportation utilizes a corresponding scale with 4 classes of severity of sulfate exposure (Class 0 to Class 3) as described in the published table below.

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

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

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

SOIL CORROSIVITY

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

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

pH Where pH is less than 4.0, soil serves as an electrolyte; the pH range of about 6.5 to 7.5 indicates soil conditions that are optimum for sulfate reduction. In the pH range above 8.5, soils are generally high in dissolved salts, yielding a low soil resistivity (AWWA, 2010). Testing indicated pH values of approximately 8.6 and 8.7.

Reduction-Oxidation testing indicated negative potentials: -90 and -102 millivolts. Such low potentials typically create a more corrosive environment.

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

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

Measurements of electrical resistivity indicated values from approximately 7,565 and 12,865 ohm-centimeters in samples of the site earth materials. The following table presents the relationship between soil resistivity and a qualitative corrosivity rating (ASM, 2003) [7].

Corrosivity Ratings Based on Soil Resistivity

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

Corrosivity Assessment The American Water Works Association (AWWA, 2010[8]) 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.

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[7] ASM International, 2003, *Corrosion: Fundamentals, Testing and Protection,* ASM Handbook, Volume 13A.
[8] American Water Works Association ANSI/AWWA C105/A21.5-05 Standard

Table A.1 Soil-test Evaluation

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

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

We anticipate that drainage at the site after construction will be effective. Nevertheless, based on the preliminary values obtained for this study, the overburden soils and bedrock appear to comprise a moderately corrosive environment for ferrous metals (11.5 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

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

General Considerations

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

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

Remnant foundation elements and any debris/man-made fill should be entirely removed and the resultant excavation properly backfilled beneath future structures and critical pavement/improvement areas.

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 pro-active measures to control surface waters during construction, to direct them away from excavations and into appropriate drainage structures. Wetting of foundation soils during construction can have adverse effects on the performance of a proposed facility.

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

Existing Fill Soils

Man-made fill was not obviously encountered during the exploration but may exist throughout the site. Actual contents and composition of the man-made fill materials are not known; therefore, some of the excavated man-made fill materials may not be suitable for replacement as backfill. The Geotechnical Engineer should be retained during site excavations to observe the excavated fill materials and provide guidance for its suitability for reuse.

Use of Existing Native Soils and Bedrock

The local native soils and excavated bedrock materials 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. Coarse cobbles and boulders, however, should not be incorporated into project fills.

Where excavated **sandstone bedrock** materials are placed as fill, the contractor should anticipate significantly more than typical efforts to moisture condition and compact the fill properly. 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. **Claystone/siltstone bedrock** fragments should be reduced so as to achieve a soil-like mass. Adequate watering, and compaction equipment that aids in breaking down the material (e.g., a Caterpillar 825 compactor-roller), likely will be needed. Excavated bedrock may require additional moisture conditioning and processing in an open area outside prior to placement as backfill.

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 with the above requirement. 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 relatively impervious soils that have less than 50 percent passing the No. 200 Sieve and should have a plasticity index of less than 15.** Representative samples of the materials proposed for import should be tested and approved by the Geotechnical Engineer 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	≤ 35
Plasticity Index	≤ 6

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

Bulkage and Shrinkage

The in-place densities of the on-site materials are variable and could be grouped into two main categories: i) formational bedrock materials and ii) overburden soils.

i) The sandstone and claystone/siltstone formational bedrock materials will likely be placed at lower densities than they exist in their native state, resulting in a net "volume gain" or bulking. This value is variable and may range from slight shrinkage to 8 percent bulking or more. (Our experience is such that an average of approximately 5 percent may be more likely).

ii) The existing overburden site soils likely will be placed at higher densities than they exist in their native state resulting in a net "volume loss" or shrink. This value is variable and may range from 0 to 5 percent shrinking or more. (Again, our experience is such that an average of approximately 5 percent may be more likely).

Such values necessarily are highly dependent upon the average depth of earthworking, the average degree of compaction achieved, construction methodology, and the variation in soil materials.

Fill Platform Preparation

Prior to filling, the top 8 to 12 inches of in-place materials on which fill soils will be placed should be scarified, moisture conditioned and properly compacted in accordance with the parameters below to provide a uniform base for fill placement.

If 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.

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

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

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

No fill materials should be placed, worked, rolled while they are frozen and/or thawing.

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.

Settlements

Settlements will occur in filled ground, typically on the order of 1 to 2 percent of the fill depth. If fill placement is performed properly and is tightly controlled, in GROUND's experience the majority (on the order of 60 to 80 percent) of that settlement will typically take place during earthwork construction, provided the contractor achieves the compaction levels provided 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.

Existing Drainages

If the existing drainage will be filled to accommodate future development, a drain should be placed at the bottom along the axis of the pre-existing drainage to discharge water that may continue to flow along the former surface drainage route. Prior to placement of the drain, all loose, soft or low density soils along the lower portions of the gully or swale should be excavated. Although depths of loose, soft or low density soils will vary, based on our test holes, we anticipate that about 1 to 3 feet of material, on average, will require removal.

Where fill is to be placed within the drainage, the slopes should be benched. The benches shall be cut approximately 10 feet horizontally into the existing slope to create a stepped bench condition. The vertical step should not exceed 4 feet between benches. To achieve adequate compaction near the outer faces of fill slopes, it may be beneficial to over-build the slopes and trim them back. Fill materials should be placed in accordance with the fill placement specification above.

The project Civil Engineer should evaluate the future potential for any drainage to convey water after being in-filled as this could influence long-term, post-construction settlements and associated movements.

Stress Release in Over-Consolidated Soils

The removal of large quantities of soils or bedrock (over 5 feet) may result in stress release of the underlying, over-consolidated materials. Stress release usually results in some degree of expansion of the soil strata. It is difficult to quantify the actual amount of expansion that may occur, however, it is possible for the expansion associated with stress release to impact the performance of the structure(s) founded in these areas. It may be advantageous to perform deep cuts as soon as possible to allow as much of the anticipated stress release to occur prior to construction of structures as possible.

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 (H) to 1 (V). In the event slopes greater than 10 feet in height are planned, a slope stability analysis should be performed. 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.

Wet Subgrade Preparation

The following subgrade preparation parameters and considerations should be utilized where soft, wet, and unstable subgrade conditions are encountered:

a. In areas where apparently stable conditions are found, the subgrade should be proof-rolled.

b. Pockets of weak or pumping soils should be excavated and replaced with pre-approved coarse granular fill (pit run) or road base. The depth of over-excavation will be on the order of 1 to 3 feet or more to provide a stable surface. The use of recycled concrete aggregate may be a cost effective alternative in this application.

c. In cases where placement of coarse aggregate fill does not result in stable conditions, it will be necessary to place a woven geotextile, Mirafi® HP370 or equivalent fabric placed below the coarse aggregate fill.

d. The surface of the subgrade should be leveled prior to geosynthetic reinforcement placement. Very weak or pumping soils should be excavated and replaced with granular fill or road base for best performance. The geosynthetic reinforcement should be placed directly on the prepared subgrade. Placement should be performed according to manufacturer's recommendations.

e. The geosynthetic rolls should be overlapped in accordance with manufacturer's recommendations.

f. Geosythetic reinforcement will be disturbed under the wheel loads of heavy construction vehicles, especially track type vehicles, therefore no vehicle traffic should be allowed over the geosynthetic reinforcement until 8 or more inches of soil has been placed over.

Use of Squeegee

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

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

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

Wherever possible, excavations should be backfilled with approved, on-site soils placed as properly compacted fill. Where this is not feasible, use of "Controlled Low Strength

Material" (CLSM), i.e., a lean, sand-cement slurry ("flowable fill") or a similar material for backfilling should be considered.

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

Detention Ponds

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

EXCAVATION CONSIDERATIONS

The test holes for the subsurface exploration performed to date by GROUND at the site were advanced to the depths indicated on the test hole logs by means of conventional truck- and buggy-mounted, continuous flight auger equipment. Practical drill rig refusal was not encountered at the time of subsurface exploration, however, very hard and resistant bedrock was encountered.

We anticipate that excavation into the bedrock will be slow even with conventional, heavy duty, excavating equipment, and will entail greater than typical wear on the equipment used.

Some excavation difficulties are anticipated, however. These may include the following:

- The presence of very hard formational bedrock within the Dawson Formation. The majority of the formational bedrock was non- to weakly cemented; however, layers and lenses of moderately to strongly cemented sandstones and conglomerates may be encountered. Where encountered in excavations, these materials will entail increased excavation difficulties, even for heavy-duty earthmoving equipment. In isolated cases, specialized equipment or light blasting may be cost effective to facilitate excavation in these deposits, particularly in trenches. Crushing or other size-reducing methods may be necessary to sufficiently reduce/process these materials adequately for use in site grading operations.

- The presence of claystone/siltstone formational bedrock. Significant processing and moisture conditioning of claystone/siltstone formational bedrock may be needed prior to incorporation in project fills.

Groundwater Conditions

Groundwater was encountered in the test holes at depths between approximately 2 and 20 feet at the time of drilling. After 5 or 7 days, groundwater was measured in Test Holes 4 and 7 at depths ranging from approximately 7 and 13 feet (elevations of approximately 6,867 to 6,893 feet), respectively. The Contractor should anticipate encountering water near these approximate elevations and be prepared to work in the presence of groundwater. Shallower excavations locally may also expose wet soils or seepage. Where seepage or groundwater is encountered in shallow project excavations, a Geotechnical Engineer should be retained to evaluate the conditions and provide additional recommendations, as appropriate.

Should seepage or flowing groundwater be encountered in project excavations, the slopes should be flattened or shored as necessary to maintain stability or a geotechnical engineer should be retained to evaluate the conditions and provide additional discussion or parameters, as appropriate. The risk of slope instability will be significantly increased in areas of seepage along excavation slopes.

The contractor should take pro-active measures to control surface waters during construction and maintain good surface drainage conditions to direct waters away from excavations and into appropriate drainage structures. The contractor should develop a dewatering plan prior to construction. 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 Cut Slopes

Temporary, unshored excavation slopes up to 10 feet in height should be cut no steeper than 1½:1 (horizontal:vertical) or flatter in the on-site soils and bedrock in the absence of groundwater or seepage. Some surficial sloughing may occur on slope faces cut at this angle. Steeper slopes in the formational bedrock materials and elsewhere may be possible depending on the conditions excavated. Loose, dry sand, or soft, wet, or seeping materials will require flatter slopes.

Should site constraints prohibit the use of the temporary slope angles, then temporary shoring should be used. Actual shoring systems should be designed for the contractor by a registered engineer.

Good surface drainage should be provided around temporary excavation slopes to direct surface runoff away from the slope faces. A properly designed swale should be provided at the top of the excavations. In no case should water be allowed to pond at the site.

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

Excavations in which personnel will be working must comply with all OSHA Standards and Regulations. The contractor's "responsible person" should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. GROUND has provided the information above solely as a service to the client, and is not assuming responsibility for construction site safety or the contractor's activities.

Locations of plowed and/or stockpiled snow during and after construction should be reviewed by a civil engineer. Snow should not be stockpiled above permanent cut and fill areas or above and below retaining structures.

UTILITY PIPE INSTALLATION AND BACKFILLING

Pipe Support: The bearing capacity of the site soils appeared adequate, in general, for support of the proposed water line. The pipe + water are less dense than the soils which will be displaced for installation. Therefore, GROUND anticipates no significant pipe settlements in these materials where properly bedded.

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

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

Controlled Low Strength Material: Because of these limitations, the 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 to 200 psi so that re-excavation is not unusually difficult.

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

Compacted Soil Backfilling: Where compacted soil backfilling is employed, using the site soils or similar materials as backfill, the risk of backfill settlements entailed in the selection of this higher risk alternative must be anticipated and accepted by the Client/Owner.

We anticipate that the on-site soils excavated from trenches will be suitable, in general, for use as common trench backfill within the above-described limitations. Backfill soils should be free of vegetation, organic debris and other deleterious materials. Fragments of rock, cobbles, and inert construction debris (e.g., concrete or asphalt) coarser than 3 inches in maximum dimension should not be incorporated into trench backfills.

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

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

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

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

SURFACE DRAINAGE

The site soils are relatively stable with regard to moisture content – volume relationships at their existing moisture contents. Other than the anticipated, post-placement settlement of fills, post-construction soil movement will result primarily from the introduction of water into the soil underlying the proposed structure, hardscaping, and pavements. Additionally, shallow groundwater was encountered in our exploration program at depths as shallow as 2 feet below existing grade. Therefore, wetting of the site soils likely will result from infiltrating surface waters (precipitation, irrigation, etc.), elevated groundwater, and water flowing along constructed pathways such as bedding in utility pipe trenches. As stated, in order to help accommodate the construction of a conventional shallow foundation system in some areas within the development, grades may have to be raised, on the order of 2 to 4 feet or more as practical (mainly to elevate the floor system), with temporary and/or permanent dewatering measures implemented.

The following drainage measures should be incorporated as part of project design and during construction. The facility should be observed periodically to evaluate the surface drainage and identify areas where drainage is ineffective. Routine maintenance of site drainage should be undertaken throughout the design life of the project. If these measures are not implemented and maintained effectively, the movement estimates provided in this report could be exceeded.

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

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

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

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

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

4) In GROUND's experience, it is common during construction that in areas of partially completed paving or hardscaping, bare soil behind curbs and gutters, and utility trenches, water is allowed to pond after rain or snow-melt events. Wetting of the subgrade can result in loss of subgrade support and increased settlements / increased heave. By the time final grading has been completed, significant volumes of water can already have entered the subgrade, leading to subsequent distress and failures. The contractor should maintain effective site drainage throughout construction so that water is directed into appropriate drainage structures.

5) On some sites, slopes may descend toward buildings locally. Such slopes can be created during grading even on comparatively flat sites. In such cases, even where the slopes as described above are implemented effectively, water may flow toward and beneath a structure or other site improvements with resultant additional, post-construction movements. Where the final site configuration includes graded or retained slopes descending toward the improvements, surface drainage swales and/or interceptor drains should be installed between the improvements and the slope.

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

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

7) Based on our experience with similar facilities, the project may include landscaping/watering near site improvements. Irrigation water – both that applied to landscaped areas and over-spray – is a significant cause of distress to improvements. To reduce the potential for such distress, vegetation requiring watering should be located 10 or more feet from building perimeters, flatwork, or other improvements. Irrigation sprinkler heads should be deployed so that applied water is not introduced near or into foundation/subgrade soils. Landscape irrigation should be limited to the minimum quantities necessary to sustain healthy plant growth.

8) Use of drip irrigation systems can be beneficial for reducing over-spray beyond planters. Drip irrigation can also be beneficial for reducing the amounts of water introduced to foundation/subgrade soils, but only if the total volumes of applied water are controlled with regard to limiting that introduction. Controlling rates of moisture increase beneath the foundations, floors, and other improvements should take higher priority than minimizing landscape plant losses.

   Where plantings are desired within 10 feet of a building, it is GROUND's opinion that the plants be placed in water-tight planters, constructed either in-ground or above-grade, to reduce moisture infiltration in the surrounding subgrade soils. Planters should be provided with positive drainage and landscape underdrains. As an alternative involving a limited increase in risk, the use of water-tight planters may be replaced by local shallow underdrains beneath the planter beds. Colorado Geological Survey – Special Publication 43 provides additional guidelines for landscaping and reducing the amount of water that infiltrates into the ground.

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

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

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

    Cobbles or other materials that tend to act as baffles and restrict surface flow should not be used to cover the ground surface near the foundations.

11) North facility areas where drainage seeps into subsurface soils may be susceptible to frost heave, which can damage site improvements.

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

PAVEMENT SECTIONS

A pavement section is a layered system designed to distribute concentrated traffic loads to the subgrade. Performance of the pavement structure is directly related to the physical properties of the subgrade soils and traffic loadings. The standard care of practice in pavement design describes the flexible pavement section as a “20-year” design pavement: however, most flexible pavements will not remain in satisfactory condition without routine maintenance and rehabilitation procedures performed throughout the life of the pavement. Pavement designs for the private pavements were developed in general accordance with the design guidelines and procedures of the American Association of State Highway and Transportation Officials (AASHTO).

Subgrade Materials

Based on the results of our field exploration and laboratory testing, the potential pavement subgrade materials classify as A-1-b to A-4 soils in accordance with the American Association of State Highway and Transportation Officials (AASHTO) classification system.

Resilient modulus (M_R) testing (AASHTO T-307) was performed on representative composite “worst case” samples of the subgrade materials encountered at the site. Typically, the R-value, unconfined compressive strength, California Bearing Ratio (CBR), or other index properties of subgrade materials have been obtained and the resilient modulus obtained only by correlation. However, due to the variability in the correlations, subjecting representative samples of the subgrade to the actual resilient modulus testing is the most accurate way to determine soil support characteristics for use in pavement design.

A dynamic load test, the resilient modulus measures the elastic rebound stiffness of flexible pavement materials, base courses, and subgrades under repeated loading. The loading cycles were applied under various confining and deviatoric stresses as specified in AASTHO T-294. The material was compacted to 95 percent of maximum dry density at optimum moisture content, and at 2 percent and 4 percent above the optimum, based on AASHTO T-99, the “standard Proctor”.

According to our testing results, a resilient modulus value of 5,850 psi was determined for the on-site materials. It is important to note that significant decreases in soil support have been observed as the moisture content increases above the optimum. Pavements that are not properly drained may experience a loss of the soil support and subsequent reduction in pavement life.

Anticipated Traffic

GROUND attempted to retrieve traffic data; however, this information 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 5 and 10 were assumed for the general parking areas and high traffic areas, respectively. The EDLA values of 5 and 10 were converted to equivalent 18-kip single axle load (ESAL) values of 36,500 and 73,000, respectively for a 20-year design life. If anticipated traffic loadings differ significantly from these assumed values, GROUND should be notified to re-evaluate the pavement sections below

Pavement Design

The soil resilient modulus and the ESAL values were used to determine the required design structural number for the project pavements. The required structural number was then used to develop the pavement sections. Pavement designs were based on the DARWin™ computer program that solves the 1993 AASHTO pavement design equations. A Reliability Level of 80 percent and a terminal serviceability of 2 were utilized for design of the pavement sections. A structural coefficient of 0.40 was used for hot bituminous asphalt and 0.12 was used for aggregate base course. The minimum pavement sections for a 20-year design are tabulated below.

Minimum Pavement Sections

Location	***Flexible Section*** *(inches Asphalt)*	***Composite Section*** *(inches Asphalt / inches Aggregate Base)*	***Rigid Section*** *(inches Concrete)*
General Parking Areas	5	3½ / 6	6
Truck Traffic Areas	6	4 / 6	6

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

Asphalt pavement should consist of a bituminous plant mix composed of a mixture of aggregate and bituminous material. Asphalt mixture(s) should meet the requirements of a job-mix formula established by a qualified Engineer.

Concrete pavements should consist of a plant mix composed of a mixture of aggregate, Portland cement and appropriate admixtures meeting the requirements of a job-mix formula established by a qualified engineer. Normally, concrete with a 28-day compressive strength of 4,000 psi should develop this modulus of rupture value. The concrete should be air-entrained with approximately 6 percent air and should have a minimum cement content of 6 sacks per cubic yard. Maximum allowable slump should be 4 inches.

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

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

Subgrade Preparation

As stated, in order to reduce the potential for post-construction movement, over-excavation and replacement of the site earth materials to depths ranging from approximately 6 to 9 feet may be necessary. However, we understand that these depths may not be cost effective for most projects. Provided the owner understands the risks identified above and accepts the potential for post-construction movement as discussed in this report, the subgrade under pavement or other (non-building) site improvements could be scarified to a depth of **12 or more inches**. This depth will result in movements and subsequent distress to site improvements. These movements will likely be more severe if surface drainage in not effective and maintained.

The Contractor should be prepared either to dry the subgrade materials or moisten them, as needed, prior to compaction. It may be difficult for the contractor to achieve and maintain compaction in some on-site soils encountered without careful control of water contents. Likewise, some site soils likely will “pump” or deflect during compaction if moisture levels are not carefully controlled. The Contractor should be prepared to process and compact such soils to establish a stable platform for paving, including use of chemical stabilization, if necessary.

Immediately prior to paving, the subgrade should be proof rolled with a heavily loaded, pneumatic tired vehicle. Areas that show excessive deflection during proof rolling should be excavated and replaced and/or stabilized. Areas allowed to pond prior to paving will require significant re-working prior to proof-rolling. Passing a proof roll is an additional requirement, beyond placement and compaction of the subgrade soils in accordance with this report. Some soils that are compacted in accordance with the parameters herein may not be stable under a proof roll, particularly at moisture contents in the upper portion of the acceptable range.

Additional Observations

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

Landscape irrigation in planters adjacent to pavements and in “island” planters within paved areas should be carefully controlled or differential heave and/or rutting of the nearby pavements will result. Drip irrigation systems are suggested for such planters to reduce over-spray and water infiltration beyond the planters. Enclosing the soil in the planters with plastic liners and providing them with positive drainage also will reduce differential moisture increases in the surrounding subgrade soils. In our experience, infiltration from planters adjacent to pavements is a principal source of moisture increase beneath those pavements. This wetting of the subgrade soils from infiltrating irrigation commonly leads to loss of subgrade support for the pavement with resultant accelerating distress, loss of pavement life and increased maintenance costs. This is particularly the case in the later stages of project construction after landscaping has been emplaced but heavy construction traffic has not ended. Heavy vehicle traffic over wetted subgrade commonly results in rutting and pushing of flexible pavements, and cracking of rigid pavements. In relatively flat areas where design drainage gradients necessarily are small, subgrade settlement can obstruct proper drainage and yield increased infiltration, exaggerated distress, etc. (These considerations apply to project flatwork, as well.)

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

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

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

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

ADDITIONAL EXPLORATION REQUIREMENTS

The above data and information are based on a limited preliminary subsurface exploration only. Additional geotechnical studies <u>must</u> be performed to further evaluate the site for building-specific foundation and floor system and final site grading.

CLOSURE

Geotechnical Review

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

In addition, building-specific geotechnical exploration(s) must be completed for the project prior to final design and construction. The preliminary geotechnical information presented in this report are contingent upon observation and testing of project earthworks by representatives of GROUND. If another geotechnical consultant is selected to provide materials testing, then that consultant must assume all responsibility for the geotechnical aspects of the project by concurring in writing with the parameters in this report, or by providing alternative parameters.

Materials Testing

The client should consider retaining a Geotechnical Engineer to perform materials testing during construction. The performance of such testing or lack thereof, in no way alleviates the burden of the contractor or subcontractor from constructing in a manner that conforms to applicable project documents and industry standards. The contractor or pertinent subcontractor is ultimately responsible for managing the quality of their work; furthermore, testing by the geotechnical engineer does not preclude the contractor from obtaining or providing whatever services they deem necessary to complete the project in accordance with applicable documents.

Limitations

This report has been prepared for Evergreen Development as it pertains to the Woodmen and Meridian 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 the final geotechnical subsurface exploration will be performed prior to construction. Changes in project plan development or schedule should be brought to the attention of the Geotechnical Engineer, in order that the preliminary geotechnical information may be re-evaluated and, as necessary, modified.

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

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

The materials present on-site are stable at their natural moisture content, but may change volume or lose bearing capacity or stability with changes in moisture content. Performance of the proposed structure(s) and pavement will depend on implementation of the preliminary information in this report, final geotechnical exploration, 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.

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

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

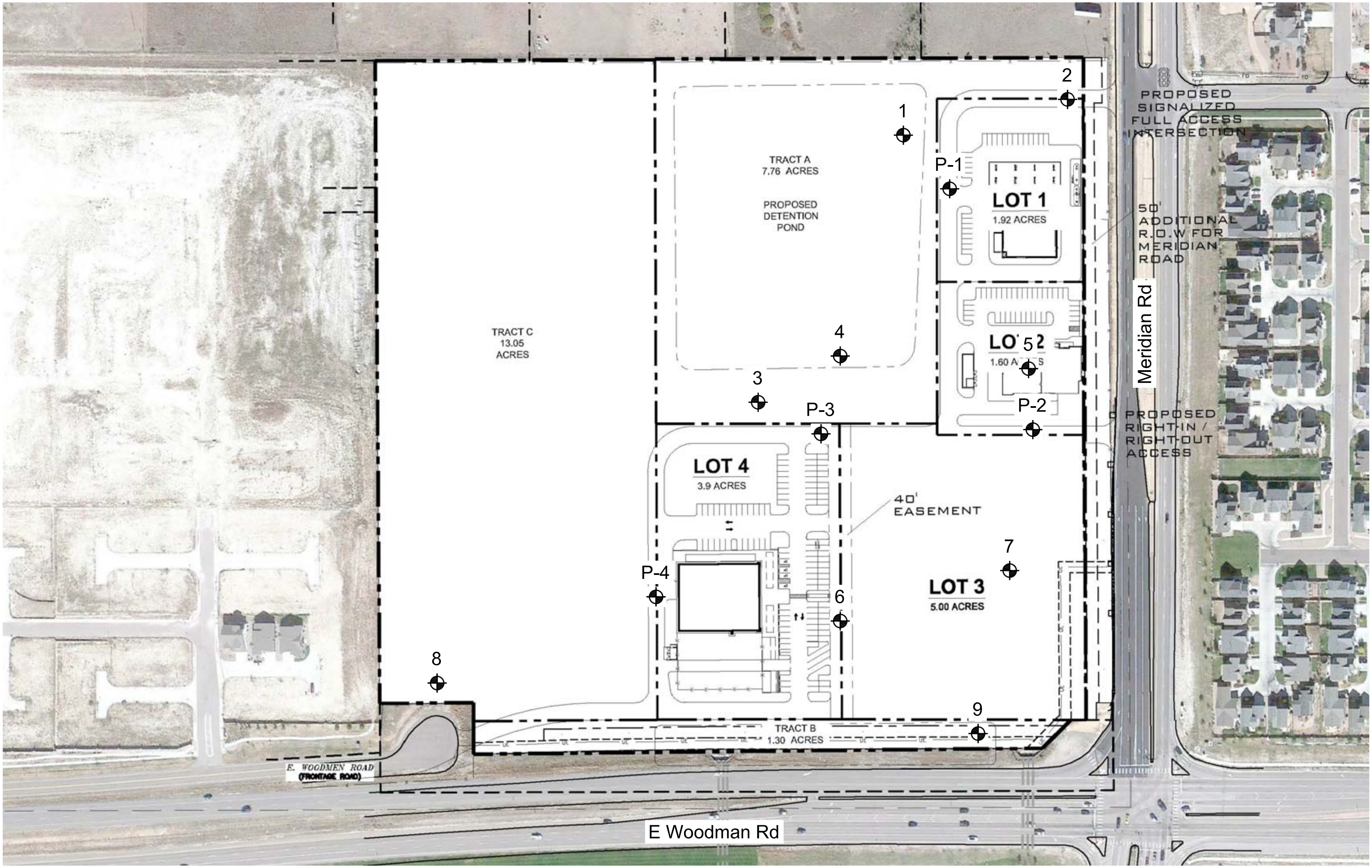
GROUND appreciates the opportunity to complete this portion of the project and welcomes the opportunity to provide the Owner with a cost proposal for final geotechnical studies and construction observation and materials testing prior to construction commencement.

Sincerely,

GROUND Engineering Consultants, Inc.

Amy Crandall, P.E.

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



SITE PLAN PROVIDED BY CLIENT

1
Indicates test hole number and approximate location.

(Not to Scale)

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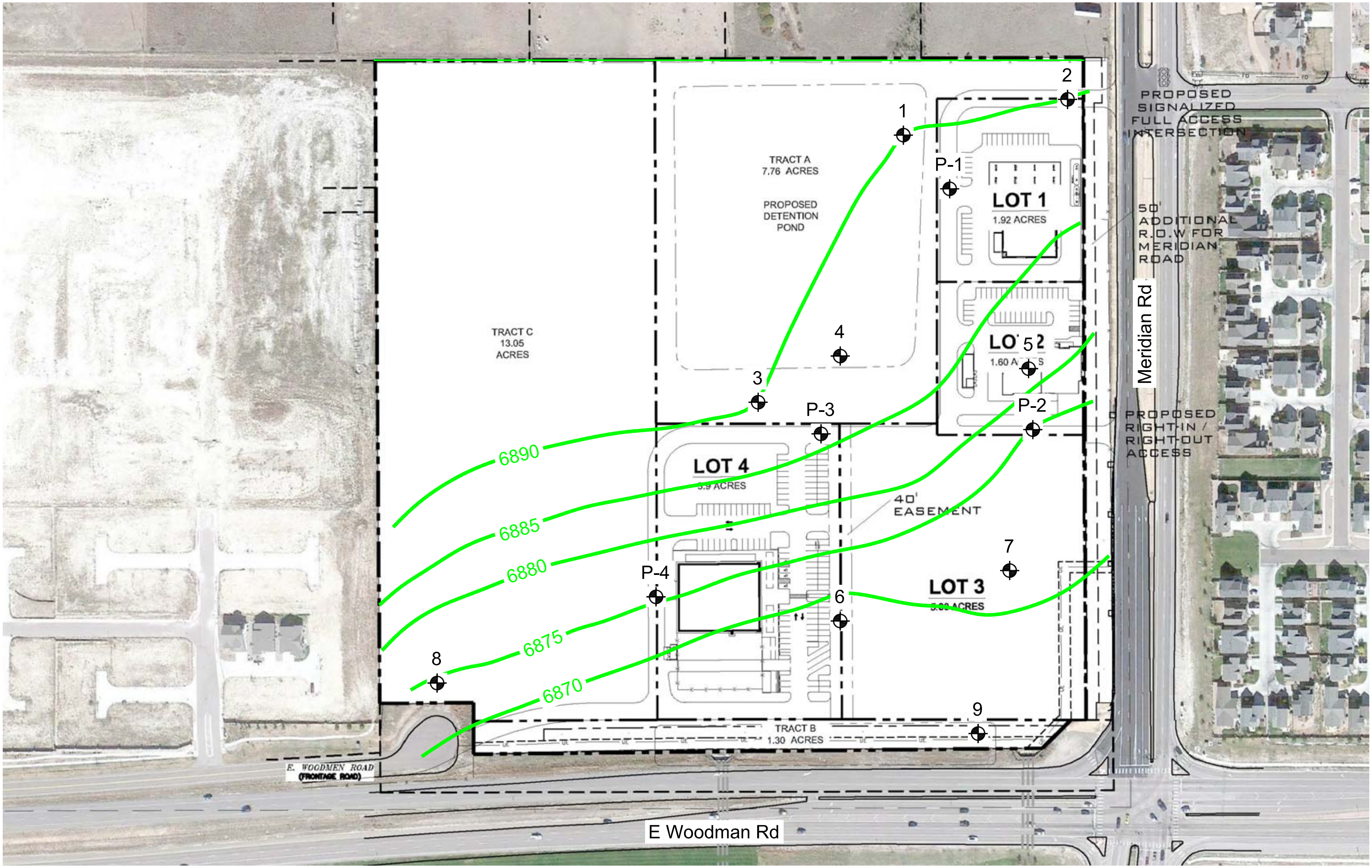
LOCATION OF TEST HOLES

JOB NO.: 15-3622	FIGURE: 1
CADFILE NAME: 3622SITE.DWG	

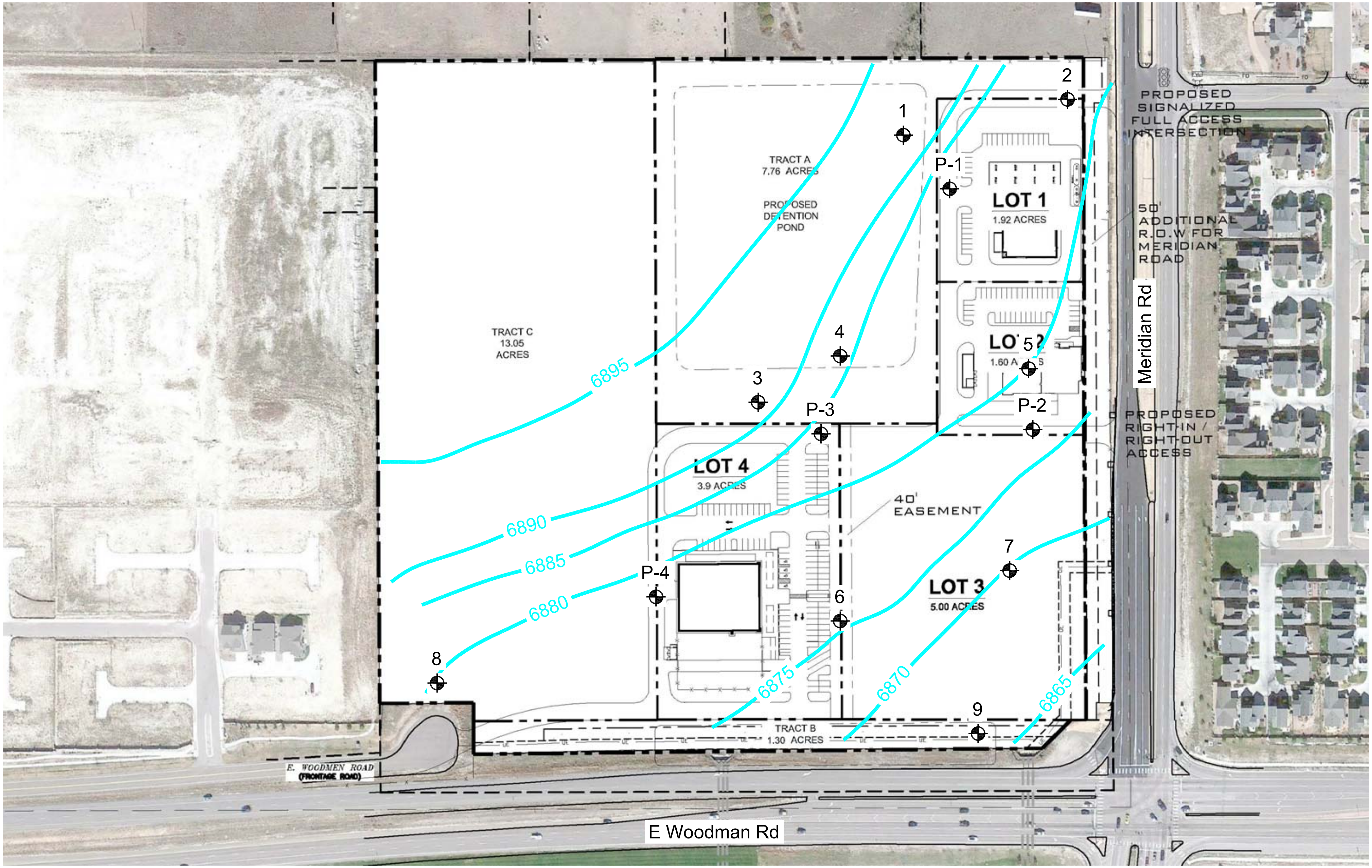


SITE PLAN PROVIDED BY CLIENT

1 Indicates test hole number and approximate location.

(Not to Scale)

GROUND ENGINEERING CONSULTANTS	
INTERPOLATED BEDROCK ELEVATIONS	
JOB NO.: 15-3622	FIGURE: 1A
CADFILE NAME: 3622SITE.DWG	



SITE PLAN PROVIDED BY CLIENT

1 ⊕ Indicates test hole number and approximate location.

(Not to Scale)

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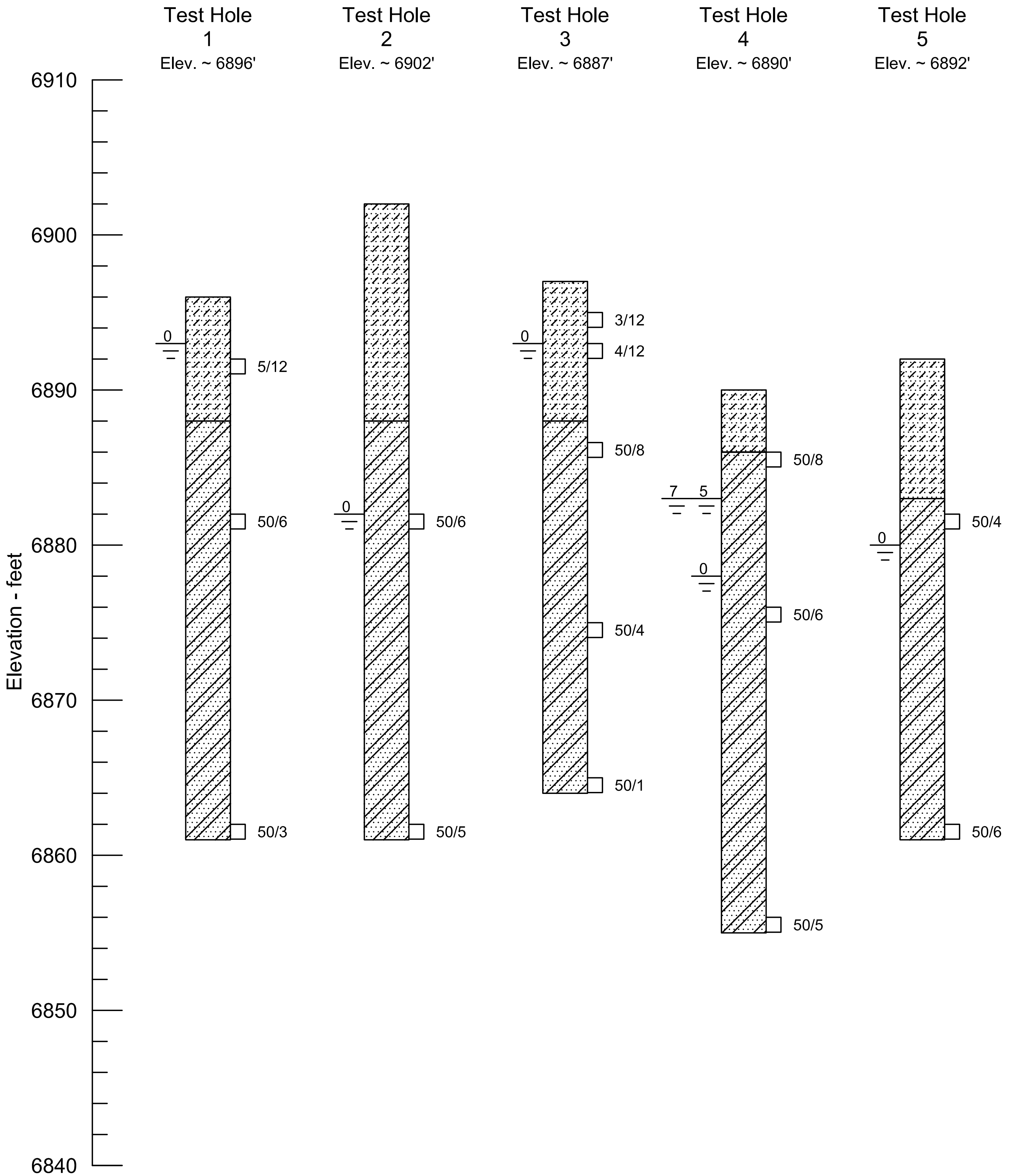
INTERPOLATED GROUNDWATER ELEVATIONS

JOB NO.: 15-3622	FIGURE: 1B
CADFILE NAME: 3622SITE.DWG	



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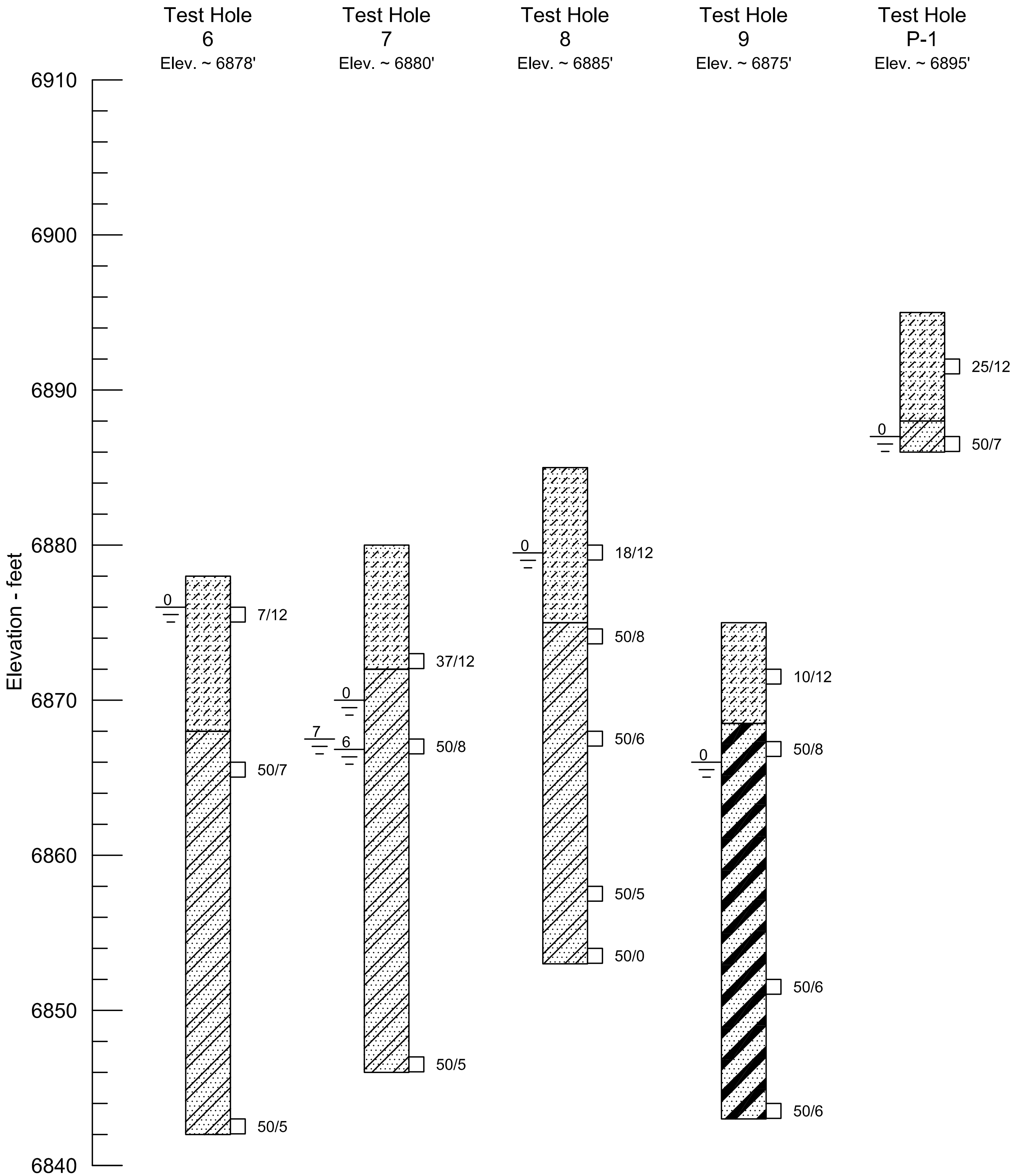
LOGS OF TEST HOLES

JOB NO.: 15-3622	FIGURE: 2
CADFILE NAME: 3622LOG01.DWG	



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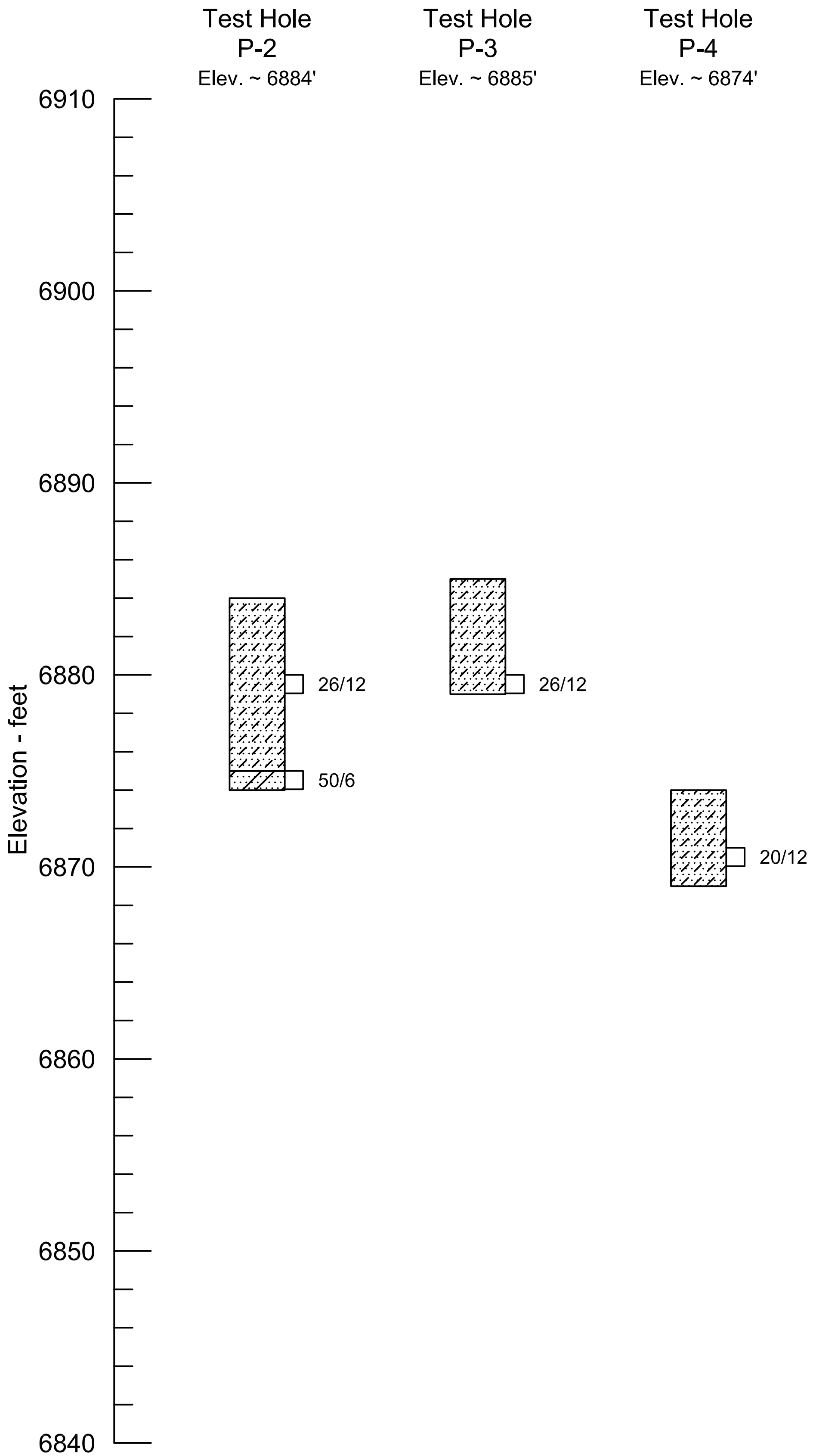
LOGS OF TEST HOLES

JOB NO.: 15-3622	FIGURE: 3
CADFILE NAME: 3622LOG02.DWG	



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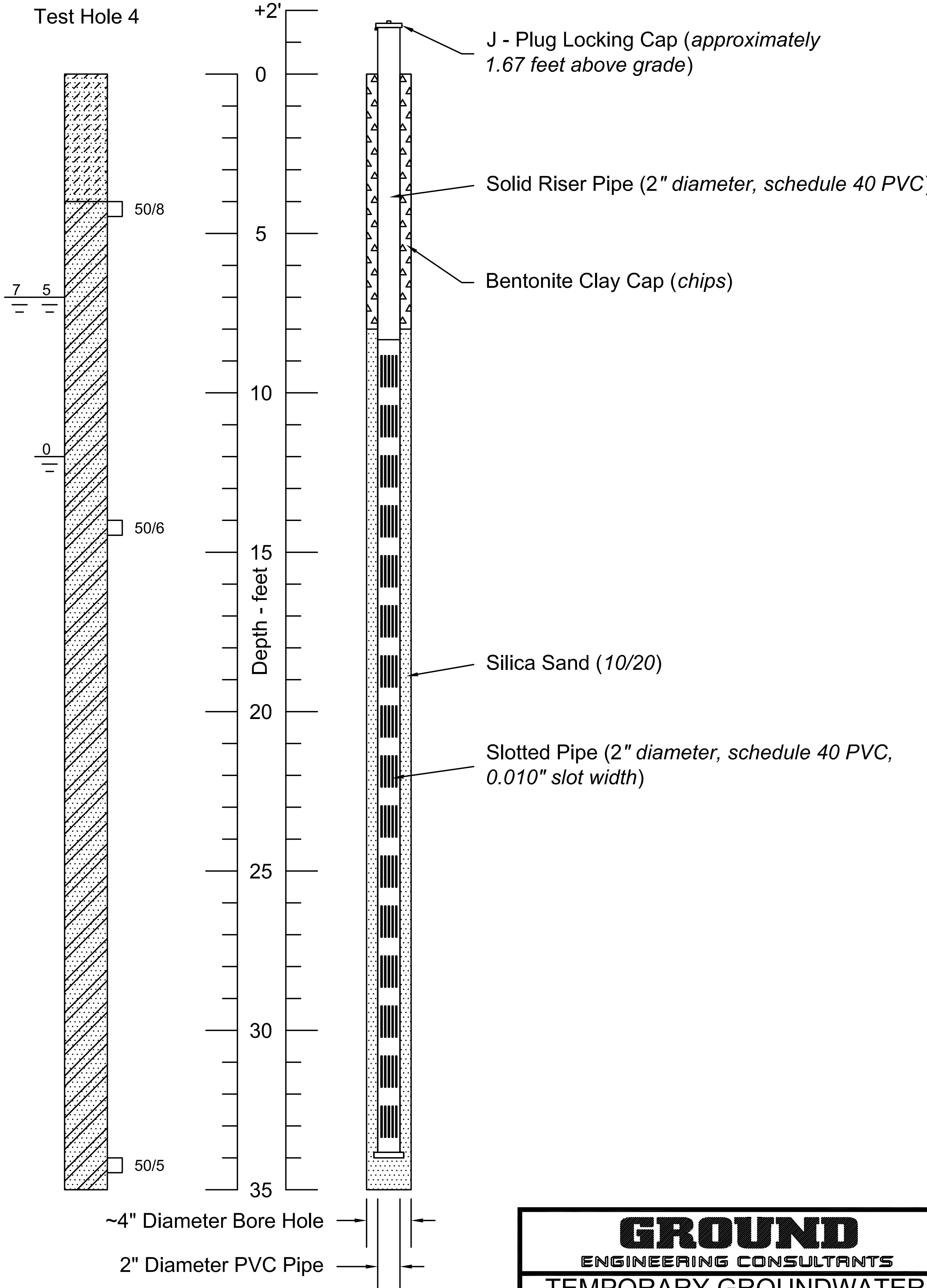
LOGS OF TEST HOLES

JOB NO.: 15-3622	FIGURE: 4
CADFILE NAME: 3622LOG03.DWG	



* Groundwater was encountered during drilling or construction of installation.

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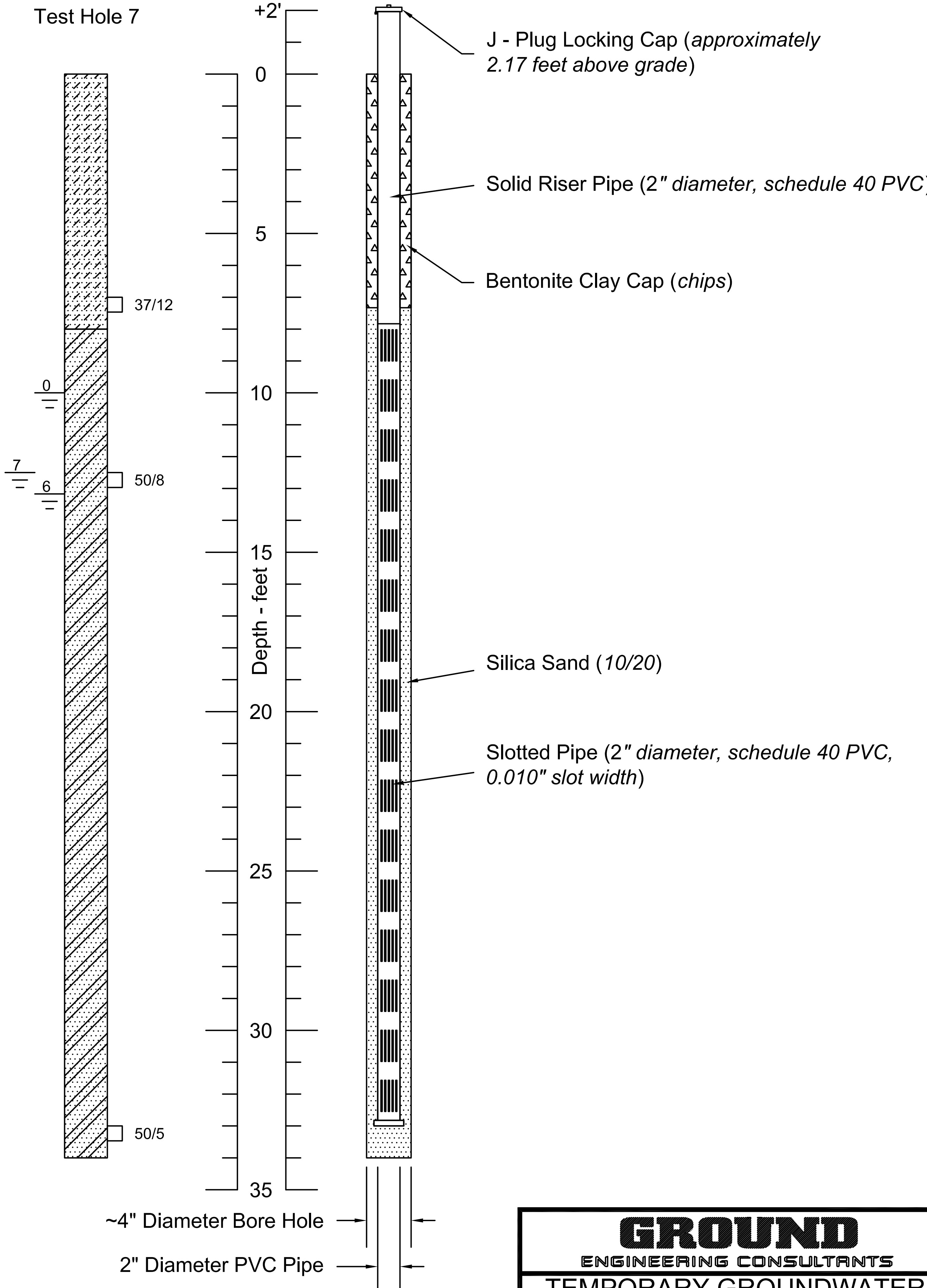
TEMPORARY GROUNDWATER OBSERVATION POINT

JOB NO.: 15-3622	FIGURE: 5
CADFILE NAME: 3622TGOP01.DWG	



* Groundwater was encountered during drilling or construction of installation.

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TEMPORARY GROUNDWATER OBSERVATION POINT

JOB NO.: 15-3622	FIGURE: 6
CADFILE NAME: 3622TGOP02.DWG	

LEGEND:

Sands and Clays: Interbedded, fine to coarse grained, low to medium plasticity, very loose to medium dense/soft to very stiff , moist to wet, and brown in color.

Sandstone Bedrock: Clayey to silty, fine- to coarse-grained with gravel, non- to low plastic, moderately hard to very hard, weakly cemented, dry to moist, and pale brown to grayish brown in color. Highly cemented sandstone may be encountered in localized areas across the site.

Sandstone and Siltstone/Claystone Bedrock: Interbedded, fine to medium grained, non-plastic to medium plastic, hard to very hard, dry to moist, occasionally iron stained, and gray to brown in color.

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

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

0

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

NOTES:

1) Test holes were drilled on 07/30, 07/31 and 08/04/2015 with 4-inch diameter continuous flight augers.
2) Locations of the test holes were measured approximately by pacing from features shown on the site plan provided.
3) Elevations of the test holes were extrapolated from client provided documents and the logs of the test holes are hung to elevation.
4) The test hole locations and elevations should be considered accurate only to the degree implied by the method used.
5) The lines between materials shown on the test hole logs represent the approximate boundaries between material types and the transitions may be gradual.
6) Groundwater 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.

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LEGEND AND NOTES

JOB NO.: 15-3622	FIGURE: 7
CADFILE NAME: 3622LEG.DWG	

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TABLE 1
SUMMARY OF LABORATORY TEST RESULTS

Sample Location		Natural	Natural	Gradation		Percent	Atterberg Limits		Percent	Unconfined	USCS	AASHTO	
Test Hole No.	Depth *(feet)*	Moisture Content *(%)*	Dry Density *(pcf)*	Gravel *(%)*	Sand *(%)*	Passing No. 200 Sieve	Liquid Limit	Plasticity Index	Swell (Surcharge Pressure)	Compressive Strength *(psf)*	Classification	Classification (GI)	Soil or Bedrock Type
1	34	11.9	117.5	-	-	30	33	5	-	14,480	SM	A-2-4(0)	SANDSTONE Bedrock
2	20	19.0	94.1	-	-	31	28	8	-	-	SC	A-2-4(0)	SANDSTONE Bedrock
3	4	12.7	98.0	3	89	8	NV	NP	-	-	SP-SM	A-1-b(0)	SAND with Silt
4	34	15.9	110.8	2	91	7	18	0	-	-	SP-SM	A-1-b(0)	SANDSTONE Bedrock
5	30	16.0	79.7	2	85	13	NV	NP	-	-	SM	A-2-4(0)	SANDSTONE Bedrock
6	2	13.1	SD	4	92	4	NV	NP	-	-	SP-SM	A-1-b(0)	SAND with Silt
6	12	14.5	111.4	-		1	29	7	-0.1% (1,500 psf)	-	SP	A-2-4(0)	SANDSTONE Bedrock
7	7	11.7	118.1	-	-	18	28	7	-	-	SC-SM	A-2-4(0)	Silty, Clayey SAND
8	17	17.9	108.3	-	-	7	NV	NP	-	-	SP-SM	A-2-4(0)	SANDSTONE Bedrock
9	3	5.4	99.2	4	28	68	33	10	0.5% (375 psf)	-	sCL	A-4(5)	Sandy CLAY
P1	3	9.5	121.0	0	83	17	25	7	-	-	SC-SM	A-2-4(0)	Silty, Clayey SAND
P2	4	9.2	124.5	2	86	12	31	10	-	-	SP-SC	A-2-4(0)	SAND with Clay
P3	5	9.7	120.3	5	85	10	30	7	-	-	SP-SC	A-1-b(0)	SAND with Clay
P1-P4	1-5	6.7*	129.1*	-	-	-	-	-	-	-	SP	A-2-4(0)	Sand

SD=Sample disturbed, NV=Non-viscous, NP=Non-plastic

*Negative swell indicates consolodation

Job No. 15-3622

GROUND
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TABLE 2
SUMMARY OF SOIL CORROSION TEST RESULTS

Sample Location		Water Soluble Sulfates	pH	Redox Potential	Sulfides Content	Resistivity	USCS Classification	Soil or Bedrock Type
Test Hole No.	**Depth** *(feet)*	*(%)*		*(mV)*		*(ohm-cm)*		
7	7	<0.01	8.6	-90	Positive	7,565	SC-SM	Silty, Clayey SAND
P-1	3	<0.01	8.7	-102	Positive	12,865	SC-SM	Silty, Clayey SAND

Job No. 15-3622