GeoStrata

# Geotechnical Investigation

# Maverik, Inc

**Southeast Corner of Fountain Boulevard and Union Boulevard**
**Colorado Springs, CO**

**April 27, 2020**

**Prepared For:**

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## TABLE OF CONTENTS

APPENDICES

# 1.0 EXECUTIVE SUMMARY

This report presents the results of a geotechnical investigation conducted for the proposed construction of a new Maverik store to be located at the southeast corner of Fountain Boulevard and Union Boulevard in Colorado Springs, Colorado. The purposes of this investigation were to assess the nature and engineering properties of the subsurface soils at the proposed site and to provide geotechnical recommendations for general site grading and the design and construction of foundations, slabs-on-grade, and pavement sections.

Based on the subsurface conditions encountered at the site, it is our opinion that the subject site is suitable for the proposed construction provided that the recommendations contained in this report are complied with.

The subsurface soil conditions were explored at the subject property by advancing four boreholes to depths of 21½ feet below the existing site grade. Based on our observations and geologic literature review, we encountered approximately 21½ feet of tan-grey, medium dense to dense, moist, sands consisting of Silty SAND (SM), Poorly Graded SAND with silt (SP-SM). This sand unit is mapped as eolian sand that is deposited by wind and preserved on surfaced downwind of the mainstrem river valleys. These deposits persisted to the full depth of our investigation in each of the boreholes. The eolian sand observed in our field investigation were not cemented and had between 9 to 16 percent fines content.

Groundwater was not encountered in any of the explorations completed as part of our investigation. Seasonal fluctuations in precipitation, surface runoff from adjacent properties, or other on or offsite sources may increase moisture conditions.

The foundations for the proposed structures may consist of conventional strip and/or spread footings. Foundations for the proposed structures should be established on a native undisturbed soil. Foundation elements should not be founded on undocumented fill soils, and if these soils are encountered, they should be over-excavated until suitable, native soils are exposed. Structural fill should meet material recommendations and be placed and compacted as recommended in Section 6.2.5. Conventional strip and spread footings founded as described above may be proportioned for a maximum net allowable bearing capacity of **2,000 pounds per square foot (psf).**

Specific considerations and recommendations concerning lateral earth pressures, pavement considerations, and soil corrosion are provided within the body of this report.

**IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL-ENGIEERING REPORT:**

**Do not rely on the executive summary. The executive summary omits a number of details, any one of which could be crucial. Read and refer to the report in full. Do not rely on this report if this report was prepared for a different client, different project, different purpose, different site, and/or before important events occurred at the site or adjacent to it. All recommendations in this report are confirmation dependent. A two-page document prepared by GBA explains these items with greater detail is found in Appendix D.**

# 2.0 INTRODUCTION

## 2.1 PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical investigation conducted for the proposed construction of a new Maverik store to be located on the southeast corner of Fountain Boulevard and Union Boulevard in Colorado Springs, Colorado. The purposes of this investigation were to assess the nature and engineering properties of the subsurface soils at the proposed site and to provide geotechnical recommendations for general site grading and the design and construction of foundations, slabs-on-grade, and pavement sections.

The scope of work completed for this study included a site reconnaissance, subsurface exploration, soil sampling, laboratory testing, engineering analyses, and preparation of this report. Our services were performed in accordance with our proposal dated March 24, 2020. The recommendations contained in this report are subject to the limitations presented in the "Limitations" section of this report.

## 2.2 PROJECT DESCRIPTION

The project site is located on the southeast corner of the intersection of Fountain Boulevard and Union Boulevard in Colorado Springs, Colorado (see Plate A-1, *Site Vicinity Map*). Information concerning the project was provided by the Client as well as in a preliminary site plan titled “Fit Study Analysis 01” dated August 29, 2019, we understand that the proposed development will consist of a 4,425 sq-ft Maverik store with associated fueling island, fuel tanks, and pavements. Our investigation for the property will be used to provide geotechnical design parameters for the construction of the proposed building, fueling island canopy, and the associated pavements and landscaping areas.

# 3.0 METHOD OF STUDY

## 3.1 SUBSURFACE INVESTIGATION

As part of this investigation, subsurface soil conditions were explored by advancing four exploratory boreholes to depths of 21½ feet below the site grade as it existed at the time of our investigation. The approximate locations of the explorations are shown on the *Exploration Location Map*, Plate A-2 in Appendix A. Exploration points were selected to provide a representative cross section of the subsurface soil conditions in the anticipated vicinity of the proposed structures. Subsurface soil conditions as encountered in the explorations were logged at the time of our investigation by a representative of the geotechnical engineer and are presented on the enclosed Borehole Logs, Plates B-1 to B-4 in Appendix B. A Key to USCS Soil Symbols and Terminology is presented on Plate B-5.

The boreholes were advanced using a truck mounted drill rig with hollow stem augers. Bulk samples were collected through the use of a standard 2-inch diameter split-spoon sampler. In addition, grab samples of the cuttings were obtained. All samples were transported to our laboratory for testing to evaluate engineering properties of the various earth materials observed. The soils were classified according to the *Unified Soil Classification System* (USCS) by the field personnel. Classifications for the individual soil units are shown on the attached Borehole Logs.

## 3.2 LABORATORY TESTING

Geotechnical laboratory tests were conducted on selected bulk soil samples obtained during our field investigation. The laboratory testing program was designed to evaluate the engineering characteristics of onsite earth materials. Laboratory tests conducted during this investigation include:

- Grain Size Distribution Analysis (ASTM D422) [1]
- Atterberg Limits (ASTM D4318)
- Moisture Density Relationship Test (ASTM D698)
- California Bearing Ratio (AASHTO T 193)
- Sulfate Content
- Soil Electrical Resistivity and pH

The results of laboratory tests are presented on the Borehole Logs in Appendix B (Plates B-1 to B-4), the Laboratory Summary Table and the test result plates presented in Appendix C (Plates C-1 through C-4).

## 3.3 ENGINEERING ANALYSIS

Engineering analyses were performed using soil data obtained from the laboratory test results and empirical correlations from material density, depositional characteristics and classification. Appropriate factors of safety were applied to the results consistent with industry standards and the accepted standard of care.

# 4.0 GENERALIZED SITE CONDITIONS

## 4.1 SURFACE CONDITIONS

At the time of our subsurface investigation, the subject site existed as a vacant lot covered in moderate amounts of native brush, small trees, and grasses, with sidewalk along Union Boulevard and approximately 90 feet of sidewalk to a bus stop. Other than utility boxes no evidence of previous structures was observed during our field investigation. Site topography is relatively flat. The subject property is bordered by Fountain Boulevard on the north, by Union Boulevard on the west, and undeveloped property to the east and south.

## 4.2 SUBSURFACE CONDITIONS

As mentioned previously, the subsurface soil conditions were explored at the subject property by advancing four boreholes to depths of 21½ feet below the existing site grade. Subsurface soil conditions were logged during our field investigation and are included on the Borehole Logs in Appendix B (Plates B-1 to B-4). The soil and moisture conditions encountered during our investigation are discussed below.

### 4.2.1 Soils

Based on our observations and geologic literature review, we encountered approximately 21½ feet of granular soils composed of tan-grey, medium dense to dense, moist, Silty SAND (SM) and Poorly Graded SAND with silt (SP-SM). This sand unit is mapped as eolian sand that is deposited by wind and preserved on surfaced downwind of the mainstrem river valleys. These deposits persisted to the full depth of our investigation in each of the boreholes. The eolian sand observed in our field investigation were not cemented and had between 9 to 16 percent fines content.

### 4.2.2 Groundwater Conditions

Groundwater was not encountered in any of the explorations completed as part of our investigation. The moisture content of samples obtained ranged from 2.8 to 4.7 percent. Seasonal fluctuations in precipitation, surface runoff from adjacent properties, or other on or offsite sources may increase moisture conditions.

### 4.2.3 Collapsible Soils

Collapse (often referred to as “hydro-collapse”) is a phenomena whereby undisturbed soils exhibit volumetric strain and consolidation upon wetting under increased loading conditions. Collapsible soils can cause differential settling of structures and roadways. Collapsible soils do not necessarily preclude development and can be mitigated by over-excavating porous, potentially collapsible soils and replacing with engineered fill and by controlling surface drainage and runoff. For some structures that are particularly sensitive to differential settlement, or in areas where collapsible soils are identified at great depth, a deep foundation system should be considered.

Soils that have a potential to collapse under increased loading and moisture conditions are typically characterized by a pinhole structure and relatively low unit weights. Cemented eolian sands typically have a potential to collapse. As stated above the eolian soils were not observed to be cemented, and it is anticipated that collapsible soils will not present a risk to the foundation elements within the proposed development if the recommendations presented in this report are incorporated into the design and construction of the However, if cemented sands are observed in any on-site excavation, GeoStrata should be contacted to provide recommendations for construction.

# 5.0 GEOLOGIC CONDITIONS

## 5.1 GEOLOGIC SETTING

The subject property is situated within the Colorado Springs Quadrangle located in El Paso County, Colorado. Colorado Springs lies along the flank of the northern Front Range within the Colorado Piedmont Physiographic Province, about 6 miles east of the mountain front within the Denver Basin. The Denver Basin is an asymmetric bowl-shaped structural depression on the east side of the Front Range. Sedimentary material shed from the rising Front Range uplift during the Laramide mountain building event filled the basin as it developed. Precambrian crystalline basement, which is at the surface in the Front Range, drops to 14,000 to 15,000 feet below the surface at its greatest depth near Castle Rock (Hemborg, 1996). To the north the Denver Basin is separated from the Cheyenne Basin by the Greeley Arch; to the south, it is separated from the Raton Basin by the Apishapa Arch. Quaternary deposits include extensive alluvium associated with modern stream systems, gravel deposits from older stream systems long abandoned, and wind deposits of sand and finer-grained loess. Evidence has not been documented that any areas in the higher parts of the Rampart Range were glaciated. However, alluvial deposits record episodes of deposition followed by erosion that correspond to periods of glaciation followed by de-glaciation elsewhere in the region (Scott, 1963a). Wind-deposited sand and loess are interpreted to reflect climatic conditions during periods of glaciation (Madole and others, 2005).

## 5.2 FAULTNG AND SEISMICITY

Research based on Colorado's earthquake history suggests that an earthquake of 6.3 or larger has a one percent probability of occurring each year somewhere in Colorado. According to the U.S. Geological Survey, the probability that a magnitude 5 or greater earthquake will occur in the next 50 years in El Paso County is 3 percent or less. The probability of such an event occurring in the next 150 years is 6 percent or less. Small earthquakes that cause no or little damage are more likely. Overall, the probability of a damaging earthquake somewhere in the county is considered occasional, 1- to 10-percent chance of occurrence in any given year, or a recurrence interval of 11 to 100 years.

Seismic hazard maps depicting probabilistic ground motions and spectral response have been developed for the United States by the U.S. Geological Survey as part of NEHRP/NSHMP

(Frankel et al, 1996). These maps have been incorporated into both *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA, 1997) and the *International Building Code* (IBC) (International Code Council, 2018). Spectral responses for the Risk-Targeted Maximum Considered Earthquake ($MCE_R$) are shown in the table below. These values generally correspond to a one percent probability of structure collapse in 50 years for a "firm rock" site. To account for site effects, site coefficients which vary with the magnitude of spectral acceleration are used. Based on our field exploration to 21½ feet, it is our opinion that this location is best described as a Site Class D (very dense soil and soft rock). The spectral accelerations are calculated based on the site's approximate latitude and longitude of 38.817° and -104.791° respectively and the Seismic Design Maps web-based application at https://seismicmaps.org/.

| **Description** | **Value** |
|---|---|
| Site Class | D (Default) |
| $S_s$ - $MCE_R$ ground motion (period – 0.2s) | 0.201 |
| $S_1$ - $MCE_R$ ground motion (period – 1.0s) | 0.058 |
| $F_a$ - Site amplification factor at 1.0s | 1.6 |
| $F_v$ - Site amplification factor at 1.0s | 2.4 |
| PGA - $MCE_G$ peak ground acceleration | 0.111 |
| $PGA_M$ – Site modified peak ground acceleration | 0.143 |

It should be noted that our investigation did not include a site-specific ground motion hazard analysis and a Site Class C has been used to determine the seismic parameters presented above based on SPT blowcount and seismic shear wave velocity correlations (Wair et al, 2012) to the maximum depths explored of 21½ feet. A ground motions hazard analysis has not been performed as part of this geotechnical investigation and is not required for the subject site according to ASCE 7-16 because $S_1$ is less than 0.6.

## 5.3 LIQUEFACTION

Certain areas within the intermountain region possess a potential for liquefaction during seismic events. Liquefaction is a phenomenon whereby loose, saturated, granular soil deposits lose a significant portion of their shear strength due to excess pore water pressure buildup resulting from dynamic loading, such as that caused by an earthquake. Among other effects, liquefaction can result in densification of such deposits causing settlements of overlying layers after an earthquake as excess pore water pressures are dissipated. The primary factors affecting liquefaction potential of a soil deposit are: (1) level and duration of seismic ground motions; (2) soil type and consistency; and (3) depth to groundwater.

Based on the lack of groundwater at the subject property as well as the relatively low anticipated seismic forces, we evaluate the liquefaction potential for this site to be low. It is possible that soil units susceptible to liquefaction may be present at depths greater than those explored as part of this investigation. If the Client wishes to have a greater understanding of the liquefaction potential at the subject site, then a liquefaction analysis can be performed.

# 6.0 ENGINEERING ANALYSIS AND RECOMMENDATIONS

## 6.1 GENERAL CONCLUSIONS

Supporting data upon which the following recommendations are based have been presented in the previous sections of this report. The recommendations presented herein are governed by the physical properties of the earth materials encountered and tested as part of our subsurface exploration and the anticipated design data discussed in the **PROJECT DESCRIPTION** section. If subsurface conditions other than those described herein are encountered in conjunction with construction, and/or if design and layout changes are initiated, GeoStrata must be informed so that our recommendations can be reviewed and revised as changes or conditions may require.

Based on the subsurface conditions encountered at the site, it is our opinion that the subject site is suitable for the proposed development provided that the recommendations contained in this report are incorporated into the design and construction of the project.

## 6.2 EARTHWORK

Prior to the placement of foundations, general site grading is recommended to provide proper support for foundations, exterior concrete flatwork, and concrete slabs-on-grade. Site grading is also recommended to provide proper drainage and moisture control on the subject property and to aid in preventing differential settlement of foundations as a result of variations in subgrade moisture conditions.

### 6.2.1 General Site Preparation and Grading

Within areas to be graded (below proposed structures, fill sections, concrete flatwork, or pavement sections), any existing vegetation, topsoil, undocumented fill, debris, or otherwise unsuitable soils should be removed. Any soft, loose, or disturbed soils should also be removed. If over-excavation is required, the excavation should extend a minimum of one foot laterally for every foot of depth of over-excavation. Excavations should extend laterally at least two feet beyond flatwork, pavements, and slabs-on-grade. Following the removal of vegetation, topsoil, undocumented fill, unsuitable soils, and loose or disturbed soils, as described above, site grading may be conducted to bring the site to design elevations.

Based on our observations in boreholes advanced for the site investigation, there is approximately 6-inches of topsoil overlying the proposed development. These deposits should be removed prior to placement of structural fill, structures, concrete flatwork, and pavements. Although not identified in our borings, any undocumented fill soils encountered during site grading should likewise be removed prior to the placement of structural fill, structures, concrete flatwork, and pavements.

A GeoStrata representative should observe the site preparation and grading operations to assess that the recommendations presented in this report are complied with.

#### 6.2.2 Soft Soil Stabilization

If soils become saturated, soft or pumping soils may be exposed in excavations at the site. Once exposed, all subgrade surfaces beneath proposed footings should be proof rolled with a piece of heavy wheeled-construction equipment. Although not anticipated, if soft or pumping soils are encountered, these soils should be stabilized prior to construction of footings. Stabilization of the subgrade soils can be accomplished using a clean, coarse angular material worked into the soft subgrade. We recommend the material be greater than 2-inch diameter, but less than 6 inches. A locally available pit-run gravel may be suitable but should contain a high percentage of particles larger than 2 inches and have less than 7 percent fines (material passing the No. 200 sieve). A pit-run gravel may not be as effective as a coarse, angular material in stabilizing the soft soils and may require more material and greater effort. The stabilization material should be worked (pushed) into the soft subgrade soils until a firm relatively unyielding surface is established. Once a firm, relatively unyielding surface is achieved, the area may be brought to final design grade using structural fill.

In large areas of soft subgrade soils, stabilization of the subgrade may not be practical using the method outlined above. In these areas it may be more economical to place a non-woven geotextile fabric against the soft soils covered by a geogrid and 12 inches of granular structural fill meeting requirements of Section 6.2.4 below. The geogrid should consist of Tensar TX130S or prior approved equivalent. The filter fabric should consist of Tencate Mirafi 140N or equivalent as approved by the Geotechnical Engineer.

### 6.2.3 Excavation Stability

Based on Occupational Safety and Health Administration (OSHA) guidelines for excavation safety, trenches with vertical walls up to 5 feet in depth may be occupied, however, the presence of fill soils, loose soils, or wet soils may require that the walls be flattened to maintain safe working conditions. When the trench is deeper than 5 feet, we recommend a trench-shield or shoring be used as a protective system to workers in the trench. Based on our soil observations, laboratory testing, and OSHA guidelines, native soils at the site classify as Type C soils. Deeper excavations, if required, should be constructed with side slopes no steeper than one- and one-half horizontal to one vertical (1½H:1V). In excavations deeper than 5 feet in depth the side slopes should be further flattened to maintain slope stability. Alternatively, shoring or trench boxes may be used to improve safe work conditions in trenches and deeper tank excavations. The contractor is ultimately responsible for trench and site safety. Pertinent OSHA requirements should be met to provide a safe work environment. If site specific conditions arise that require engineering analysis in accordance with OSHA regulations, GeoStrata can respond and provide recommendations as needed.

We recommend that a GeoStrata representative be on-site during all excavations to assess the exposed foundation soils. We also recommend that the Geotechnical Engineer be allowed to review the grading plans when they are prepared in order to evaluate their compatibility with these recommendations.

### 6.2.4 Fuel Tank Bedding

If the fuel tanks for the proposed construction have rounded bottoms, bedding material should be placed below the tank spring line to provide proper support for the tanks. Bedding material should consist of sand or gravel meeting tank manufacturer and/or project specifications. If the bedding material is not proctorable according to ASTM D1557, the bedding material should be compacted below and around the haunches of the tank to a minimum of 75% of the relative maximum density as determined by ASTM D4253. If the bedding material meets requirements for ASTM D1557, bedding material should be placed as structural fill and meet placement and compaction requirements given below in Section 6.2.5.

### 6.2.5 Structural Fill and Compaction

All fill placed for the support of structures, concrete flatwork, or pavements should consist of structural fill. Native, onsite granular and fine-grained sand soils may be utilized as structural fill, although the Client should be aware that these soils may be difficult to moisture condition and compact during certain times of the year. As an alternative, structural fill may consist of an imported soil. Imported structural fill may consist of a relatively well graded granular soil with a maximum of 50 percent passing the No. 4 mesh sieve and a minimum fines content (minus No. 200 mesh sieve) of 25 percent. All structural fill soils should be approved by the Geotechnical Engineer prior to placement. Clay and silt particles in imported structural fill should have a liquid limit less than 35 and a plasticity index less than 15 based on the Atterberg Limit's test (ASTM D-4318). The contractor should anticipate testing all soils used as structural fill frequently to assess the maximum dry density, fines content, and moisture content, etc. Soils not meeting the aforementioned criteria may be suitable for use as structural fill. These soils should be evaluated on a case-by-case basis and should be approved by the Geotechnical Engineer prior to use.

All structural fill should be placed in maximum 6-inch loose lifts if compacted by small hand-operated compaction equipment, maximum 12-inch loose lifts if compacted by light-duty rollers, and heavy duty compaction equipment that is capable of efficiently compacting the entire thickness of the lift. We recommend that all structural fill be compacted on a horizontal plane, unless otherwise approved by the geotechnical engineer. Structural fill should be compacted to at least 95% of the maximum dry density as determined by ASTM D-1557 where total fill thickness is less than 5 feet. Where total structural fill thickness is 5 feet or more, structural fill should be compacted to at least 98% of the maximum dry density (ASTM D-1557). The moisture content should be at or slightly above the optimum moisture content at the time of placement and compaction. Also, prior to placing any fill, the excavations should be observed by the geotechnical engineer to observe that any unsuitable materials or loose soils have been removed. In addition, proper grading should precede placement of fill, as described in the **General Site Preparation and Grading** subsection of this report (Section 6.2.1).

The gradation, placement, moisture, and compaction recommendations contained in this section meet our minimum requirements but may not meet the requirements of other governing agencies such as city, county, or state entities. If their requirements exceed our recommendations, their specifications should override those presented in this report.

## 6.3 FOUNDATIONS

The foundations for the proposed structures may consist of conventional strip and/or spread footings. Strip and spread footings should be a minimum of 20 and 36 inches wide, respectively, and exterior shallow footings should be embedded at least 30 inches below final grade for frost protection and confinement. Interior shallow footings not susceptible to frost conditions should be embedded at least 18 inches for confinement.

### 6.3.1 Installation and Bearing Material

Foundations for the proposed structures should be established on a native undisturbed sand soils. Foundation elements should not be founded on undocumented fill soils, and if these soils are encountered, they should be over-excavated until suitable, native soils are exposed. Structural fill should meet material recommendations and be placed and compacted as recommended in Section 6.2.5.

### 6.3.2 Bearing Pressure

Conventional strip and spread footings founded as described above may be proportioned for a maximum net allowable bearing capacity of **2,000 pounds per square foot (psf).** The recommended net allowable bearing pressure refers to the total dead load and can be increased by 1/3 to include the sum of all loads including wind and seismic.

### 6.3.3 Settlement

Settlements of properly designed and constructed conventional footings, founded as described above, are anticipated to be less than 1 inch. Differential settlements should be on the order of half the total settlement over 30 feet.

### 6.3.4 Frost Depth

According to the Pikes Peak Regional Building Department, 30-inches of foundation cover is required for frost protection. This includes walk-out areas and may require fill to be placed around buildings. In order to achieve adequate bearing capacity, all footings should be embedded at least 18 inches for confinement. If foundations are constructed through the winter months, all soils on which footings will bear shall be protected from freezing.

### 6.3.5 Construction Observation

A geotechnical engineer shall periodically monitor excavations prior to installation of footings. Inspection of soil before placement of structural fill or concrete is required to detect any field conditions not encountered in the investigation which would alter the recommendations of this report. All structural fill material shall be tested under the direction of a geotechnical engineer for material and compaction requirements.

## 6.4 EARTH PRESSURES AND LATERAL RESISTANCE

Lateral forces imposed upon conventional foundations due to wind or seismic forces may be resisted by the development of passive earth pressures and friction between the base of the footing and the supporting subgrade. A coefficient to friction of 0.40 should be used for natives soils against concrete.

Ultimate lateral earth pressures from backfill consisting of native soils acting against buried walls and structures may be computed from the lateral pressure coefficients or equivalent fluid densities presented in the following table:

| Condition | Lateral Pressure Coefficient | Equivalent Fluid Density (pounds per cubic foot) |
|---|---|---|
| Active* | 0.31 | 37 |
| At-rest** | 0.47 | 56 |
| Passive* | 3.25 | 391 |
| Seismic Active*** | 0.03 | 4 |
| Seismic Passive*** | -0.10 | -12 |

*     Based on Coulomb's equation
**   Based on Jaky
*** Based on Mononobe-Okabe Equation

These coefficients and densities assume level, granular backfill with no buildup of hydrostatic pressures. The lateral earth pressures presented are for native soils only, if granular imported soils are used as backfill, GeoStrata should be contacted to provide lateral earth pressures for these conditions. The force of the water should be added to the presented values if hydrostatic pressures are anticipated. If sloping backfill is present, we recommend the geotechnical engineer be consulted to provide more accurate lateral pressure parameters once the design geometry is established.

Walls and structures allowed to rotate slightly should use the active condition. If the element is constrained against rotation, the at-rest condition should be used. These values should be used with an appropriate factor of safety against overturning and sliding. A value of 1.5 is typically used. Additionally, if passive resistance is calculated in conjunction with frictional resistance, the passive resistance should be reduced by ½.

For seismic analyses, the *active* and *passive* earth pressure coefficient provided in the table is based on the Mononobe-Okabe pseudo-static approach and only accounts for the dynamic horizontal thrust produced by ground motion. Hence, the resulting dynamic thrust pressure *should be added* to the static pressure to determine the total pressure on the wall. The pressure distribution of the dynamic horizontal thrust may be closely approximated as an inverted triangle with stress decreasing with depth and the resultant acting at a distance approximately 0.6 times the loaded height of the structure, measured upward from the bottom of the structure.

The coefficients shown assume a vertical wall face. Hydrostatic and surcharge loadings, if any, should be added. Over-compaction behind walls should be avoided. Resisting passive earth pressure from soils subject to frost or heave, or otherwise above prescribed minimum depths of embedment, should usually be neglected in design.

## 6.5 CONCRETE SLAB-ON-GRADE CONSTRUCTION

Concrete slabs should be constructed over at least 4 inches of gravel overlying native soils or structural fill. Disturbed native soils should be compacted to at least 95% of the MDD as determined by ASTM D-1557 (modified proctor) prior to placement of gravel. The gravel should consist of road base or clean drain rock with a ¾-inch maximum particle size and no more than 12 percent fines passing the No. 200 mesh sieve. The gravel layer should be compacted to at least 95 percent of the MDD of modified proctor or until tight and relatively unyielding if the material is non-proctorable. All concrete slabs should be designed to minimize cracking as a result of shrinkage. Consideration should be given to reinforcing the slab with welded wire, re-bar, or fiber mesh. In order to minimize potential movement of the exterior flatwork, the Owner should consider placing 12 inches of structural fill beneath the 4 inches of gravel.

## 6.6 MOISTURE PROTECTION AND SURFACE DRAINAGE

Moisture should not be allowed to infiltrate the soils in the vicinity of the foundations. We recommend the following mitigation measures be implemented at the building location.

- The ground surface within 10 feet of the entire perimeter of the building should slope a minimum of five percent away from the structure. Alternatively, a slope of 5% is acceptable if the water is conveyed to a concrete ditch that will convey the water to a point of discharge that is at least 10 feet from the structures.
- Roof runoff devices (rain gutters) should be installed to direct all runoff a minimum of 10 feet away from the structure and preferably day-lighted to the curb where it can be transferred to the storm drain system. Rain gutters discharging roof runoff adjacent to or within the near vicinity of the structure may result in excessive differential settlement.
- We do not recommend storm drain collection sumps be used as part of this development. However, if necessary, sumps should not be located adjacent to foundations or within roadway pavements due to the presence of potentially collapsible soils.
- We recommend irrigation around foundations be minimized by selective landscaping and that irrigation valves be constructed at least 5 feet away from foundations.
- Jetting (injecting water beneath the surface) to compact backfill against foundation soils may result in excessive settlement beneath the building and is not allowed.
- Backfill against foundations walls should consist of imported fine-grained soils and should be placed in lifts and compacted to 90% modified proctor to create a moisture barrier.

Failure to comply with these recommendations could result in excessive total and differential settlements causing structural damage.

## 6.7 SOIL CORROSION

One (1) representative soil sample was tested for soil chemical reactivity. Chemical reactivity tests were performed to determine soil pH, resistivity, and concentrations of water-soluble sulfate ions. Results from these tests are summarized in the table below.

| Boring Number | Depth (ft) | Sulfate (ppm) | Resistivity (Ω-cm) | Soil pH |
|---|---|---|---|---|
| B-2 | 2.5 | 5.56 | 9,500 | 8.41 |

Test results indicate that the soluble sulfate concentrations of 5.56 ppm. Based on the American Concrete Institute (ACI) Building Code, these concentrations represent "Negligible" degree of sulfate attack on concrete structures. Type I or II Portland Cement Concrete (PCC) may be used for concrete elements in contact with the onsite soils or properly placed and compacted granular structural fill.

Laboratory soil resistivity has a direct impact on the degree of corrosion in underground steel structures. A decrease in resistivity relates to an increase in corrosion activity and therefore dictates that protective treatment to be used. Results from the laboratory resistivity tests indicate a resistivity of 9,500 ohm-cm. Based on the resistivity test results, the onsite soils are considered to be “mildly corrosive” to ferrous metals if saturated in the field.

Results of the ion hydrogen concentration (pH) tests were 8.41. Concentrations greater than 5 and less than 10 are less likely to contribute to corrosion attack on subsurface steel structures.

Anticipated underground steel and concrete structures (i.e., pipes, exposed steel, footings, floor slabs) should be protected against corrosion.

## 6.8 PAVEMENT SECTION

A representative soil sample was collected during our field investigation for laboratory California Bearing Ratio (CBR) testing which resulted in a CBR value of 2.7. No traffic information was available at the time this report was prepared, therefore, GeoStrata has assumed traffic counts for access roads and parking areas. We assumed that the vehicle traffic in and out of the fueling area would consist of approximately 4000 passenger vehicles/day, 25 light duty trucks/day, 15 medium trucks/day, and 2 heavy trucks/day. The following pavement design alternatives have been developed for a 20-year design life assuming an annual growth rate of 0% and an estimated single axle load (ESAL) of approximately 260,000 ESALs. The pavement sections given below are equivalent options that may be selected based on economic considerations.

| Pavement Materials | Recommended Minimum Thickness (inches) | | | |
|---|---|---|---|---|
| | Standard Pavement | | Geogrid Reinforced Pavement | |
| | Pavement 1 | Pavement 2 | Pavement 3 | Pavement 4 |
| Asphaltic Concrete | 3.5 | 3.5 | 3.5 | 3.5 |
| Untreated Base Course | 20 | 6 | 11 | 6 |
| Granular Borrow | --- | 17 | --- | 10 |

All topsoil, or any soil containing organic materials, must be removed from locations where structural loads will be applied. To evaluate its stability, the sub-grade shall be proof rolled with a loaded dump truck. Any unsuitable soils shall be removed and replaced with structural fill according to Section 6.2.5 or stabilized according to Section 6.2.2. Any areas of fill or disturbed areas shall be compacted to 95% of the ASTM D1557 modified proctor. A geotechnical engineer shall observe unsuitable subgrade remediation.

Asphalt has been assumed to be a high stability plant mix; base course material should be composed of crushed stone with a minimum CBR of 70. Asphalt should be compacted to a minimum density of 96% of the Marshall value and base course should be compacted to at least 95% of the MDD of the modified proctor. Granular borrow (subbase) material may be used to reduce the required thickness of untreated base course (road base) and should consist of a granular borrow material as defined in APWA Standard Specifications, Section 31 05 13, “Common Fill”, and should have a CBR of 30.

Geogrid reinforcement, if used, should consist of Tensar TX5 or equivalent. Geogrid should be placed directly beneath the base course material and in accordance wtih manufacturer’s recommendations including overlap of adjacent geogrid rolls. A non-woven geotextile filter fabric such as Tencate 140N or equivalent should be placed directly on the prepared subgrade soils with the geogrid placed on the non-woven fabric. The filter fabric may be omitted if subbase (granular borrow) is placed beneath the base course.

It is our experience that pavement in areas where trucks frequently turn around, backup, or load and unload, including fueling areas, experience more distress. If the owner wishes to prolong the life of the pavement in these areas, consideration should be given to using a Portland cement concrete (rigid) pavement in these areas. The following rigid pavement section is recommended:

| Concrete (in) | Untreated Base Course (in) |
|---|---|
| 6.5 | 6 |

Concrete should consist of a low slump, low water cement ratio mix with a minimum 28-day compressive strength of 4,000 psi. Base course should be compacted to at least 95% of the MDD as determined by the ASTM D-1557. Additionally, we have assumed that the upper 12 inches of the subgrade will be reworked and compacted to at least 95% of the MDD as determined by the ASTM D-1557.

If traffic conditions vary significantly from our stated assumptions, GeoStrata should be contacted so we can modify our pavement design parameters accordingly. Specifically, if the traffic counts are significantly higher or lower, we should be contacted to revise the pavement section design as necessary. The pavement section thickness above assumes that the majority of the construction traffic including cement trucks, cranes, loaded haulers, etc. has ceased. If a significant volume of construction traffic occurs after the pavement section has been constructed, the owner should anticipate maintenance or a decrease in the design life of the pavement area.

# 7.0 CLOSURE

## 7.1 LIMITATIONS

The recommendations contained in this report are based on our limited field exploration, laboratory testing, and understanding of the proposed construction. The subsurface data used in the preparation of this report were obtained from the explorations made for this investigation. It is possible that variations in the soil and groundwater conditions could exist between and beyond the points explored. The nature and extent of variations may not be evident until construction occurs. If any conditions are encountered at this site that are different from those described in this report, GeoStrata should be immediately notified so that we may make any necessary revisions to recommendations contained in this report. In addition, if the scope of the proposed construction changes from that described in this report, GeoStrata should be notified.

This report was prepared in accordance with the generally accepted standard of practice at the time the report was written. No other warranty, expressed or implied, is made.

It is the Client's responsibility to see that all parties to the project including the Designer, Contractor, Subcontractors, etc. are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the Contractor's option and risk.

## 7.2 ADDITIONAL SERVICES

The recommendations made in this report are based on the assumption that an adequate program of tests and observations will be made during construction. GeoStrata staff should be on site to verify compliance with these recommendations. These tests and observations should include, but not necessarily be limited to, the following:

- Observations and testing during site preparation, earthwork and structural fill placement.
- Observation of foundation soils to assess their suitability for footing placement.
- Observation of soft/loose soils over-excavation.
- Observation of temporary excavations and shoring.
- Consultation as may be required during construction.
- Quality control and observation of concrete placement.

We also recommend that project plans and specifications be reviewed by GeoStrata to verify compatibility with our conclusions and recommendations. Additional information concerning the scope and cost of these services can be obtained from our office.

We appreciate the opportunity to be of service on this project. Should you have any questions regarding the report or wish to discuss additional services, please do not hesitate to contact us at your convenience at (801) 501-0583.

## 8.0 REFERENCES CITED


Carroll, C.J., Crawford, T.A., 2000, Geologic Map of the Colorado Springs Quadrangle, El Paso County, Colorado; United States Geological Survey, Open-File Map 00-3, scale 1:24,000.

Hemborg, H.T., 1996, Basement structure map of Colorado with major oil and gas fields: Colorado Geological Survey Map Series MS-30, scale 1:1,000,000.

Federal Emergency Management Agency [FEMA], 1997, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures,* FEMA 302, Washington, D.C.

International Building Code [IBC], 2018, International Code Council, Inc.

Leyendecker, E.V., Perkins, D.M., Algermissen, S.T., Thenhaus, P.C., and Hanson, S.L., 1995, USGS spectral response maps and their relationship with seismic design forces in building codes: U.S. Geological Survey, Open-File Report 95-596.

Madole R. F., VanSistine, D. P., Michael, J.A., 2005, Distribution of Late Quaternary Wind-Deposited Sand in Eastern Colorado, Scientific Investigation Map 2875

Wair, B.R., DeJong, J.T., Shantz, T., 2012, guidelines for Estimation of Shear Wave Velocity Profiles, Pacific Earthquake Engineering Research Center

# APPENDIX A


300 0 300 600 900 1200 ft

1:8000

N

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Maverik, Inc.
Fountain Blvd & Union Blvd
Colorado Springs, CO
Project Number: 1092-066

**Site Vicinity Map**

**Plate A-1**

Legend

Approximate Site Boundary


GeoStrata


## Legend

Approximate Boring Location

Approximate Site Boundary

Maverik, Inc.
Fountain Blvd & Union Blvd
Colorado Springs, CO
Project Number: 1092-066

**Exploration Location Map**

**Plate A-2**

# APPENDIX B

| DATE | |
|---|---|
| STARTED: | 4/8/20 |
| COMPLETED: | 4/8/20 |
| BACKFILLED: | 4/8/20 |

Maverik, Inc.
Fountain Blvd & Union Blvd
Colorado Springs, CO
Project Number 1092-066

GeoStrata Rep: J. Sage
Rig Type: CME 55
Boring Type: HSA

BORING NO:
# B-1
Sheet 1 of 1

LOCATION
STATION OFFSET ELEVATION

| DEPTH (FEET) | UNIFIED SOIL CLASSIFICATION | MATERIAL DESCRIPTION | N | N* | Dry Density(pcf) | Moisture Content % | Percent minus 200 | Liquid Limit | Plasticity Index |
|---|---|---|---|---|---|---|---|---|---|
| 0 | SP-SM | Poorly Graded SAND with silt - dense, slightly moist, tan-grey, sand is fine-grained | | | | | | | |
| | | | 20 | 41 | | | | | |
| 5 | | - medium dense | 12 | 17 | | 4.6 | 10.3 | NP | NP |
| | | | 9 | 16 | | | | | |
| 10 | | | 12 | 21 | | | | | |
| 15 | | | 13 | 24 | | | | | |
| 20 | | - dense | 18 | 34 | | | | | |
| | | Bottom of Boring @ 21.5 Feet | | | | | | | |

SPT BLOW COUNT: 10 20 30 40 50 60 70 80 90

Moisture Content and Atterberg Limits: Plastic Limit, Moisture Content, Liquid Limit; 10 20 30 40 50 60 70 80 90

N - OBSERVED UNCORRECTED BLOW COUNT
N* - CORRECTED N1(60) EQUIVALENT SPT BLOW COUNT

GeoStrata



SAMPLE TYPE
- 2" O.D./1.38" I.D. Split Spoon Sampler
- 2.5" O.D./2" I.D. California Split Spoon Sampler
- 3" O.D. Thin-Walled Shelby Sampler
- Grab Sample
- 2" O.D./1.625" I.D. Liner Sampler

NOTES:

WATER LEVEL
- MEASURED
- ESTIMATED

Plate
B - 1

| DATE | |
|---|---|
| STARTED: | 4/8/20 |
| COMPLETED: | 4/8/20 |
| BACKFILLED: | 4/8/20 |

# Maverik, Inc.
## Fountain Blvd & Union Blvd
## Colorado Springs, CO

Project Number 1092-066

GeoStrata Rep: J. Sage
Rig Type: CME 55
Boring Type: HSA

BORING NO: **B-2**

Sheet 1 of 1

LOCATION

STATION OFFSET ELEVATION

| DEPTH (FEET) | SAMPLES | UNIFIED SOIL CLASSIFICATION | MATERIAL DESCRIPTION | N | N* | Dry Density(pcf) | Moisture Content % | Percent minus 200 | Liquid Limit | Plasticity Index |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | SM | Silty SAND - medium dense, slightly moist, tan-grey, sand is fine-grained | | | | | | | |
| | X | | - dense | 16 | 33 | | | | | |
| 5 | X | | - medium dense | 9 | 17 | | | | | |
| | X | SP-SM | Poorly Graded SAND with silt - medium dense, slightly moist, tan-grey, sand is fine-grained | 16 | 30 | | 2.8 | 9.9 | NP | NP |
| 10 | X | | | 9 | 16 | | | | | |
| 15 | X | | | 10 | 18 | | | | | |
| 20 | X | | | 8 | 15 | | | | | |
| | | | Bottom of Boring @ 21.5 Feet | | | | | | | |

SPT BLOW COUNT: 10 20 30 40 50 60 70 80 90

Moisture Content and Atterberg Limits: Plastic Limit, Moisture Content, Liquid Limit: 10 20 30 40 50 60 70 80 90

METERS: 0, 1, 2, 3, 4, 5, 6, 7

N - OBSERVED UNCORRECTED BLOW COUNT
N* - CORRECTED N1(60) EQUIVALENT SPT BLOW COUNT

GeoStrata



SAMPLE TYPE
- 2" O.D./1.38" I.D. Split Spoon Sampler
- 2.5" O.D./2" I.D. California Split Spoon Sampler
- 3" O.D. Thin-Walled Shelby Sampler
- Grab Sample
- 2" O.D./1.625" I.D. Liner Sampler

NOTES:

WATER LEVEL
- MEASURED
- ESTIMATED

**Plate B - 2**

| DATE | |
|---|---|
| STARTED: | 4/8/20 |
| COMPLETED: | 4/8/20 |
| BACKFILLED: | 4/8/20 |

# Maverik, Inc.
## Fountain Blvd & Union Blvd
## Colorado Springs, CO

Project Number 1092-066

GeoStrata Rep: J. Sage
Rig Type: CME 55
Boring Type: HSA

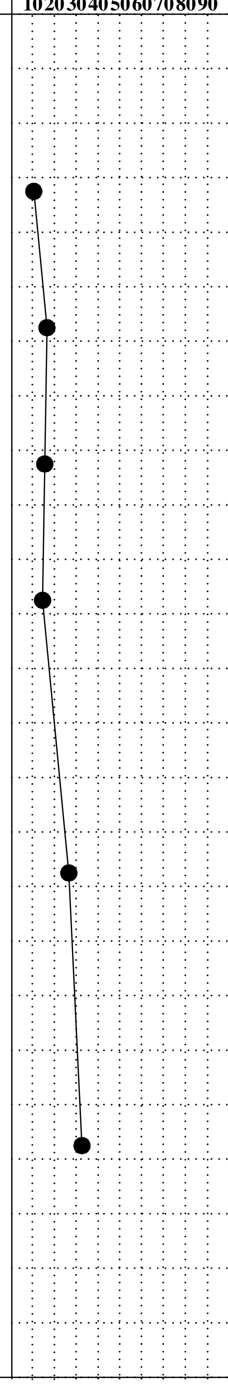
BORING NO: **B-3**

Sheet 1 of 1

LOCATION: STATION / OFFSET / ELEVATION

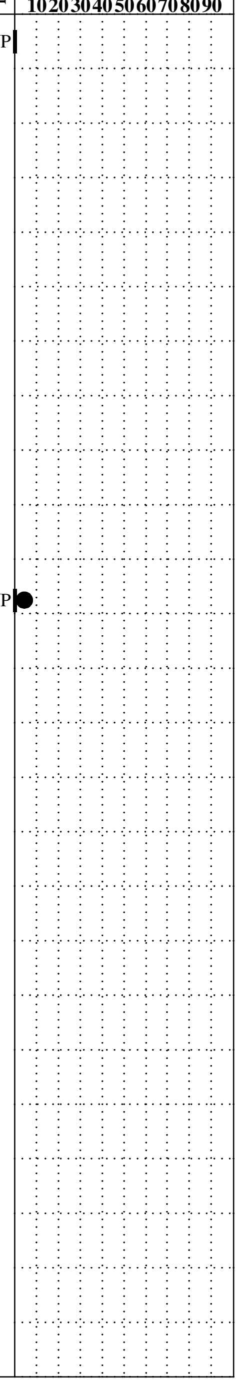
| Depth (feet) | Unified Soil Classification | Material Description | N | N* | Dry Density (pcf) | Moisture Content % | Percent minus 200 | Liquid Limit | Plasticity Index |
|---|---|---|---|---|---|---|---|---|---|
| 0 | SM | Silty SAND - medium dense, slightly moist, tan-grey, sand is fine-grained | | | | | 16.4 | NP | NP |
| | | | 5 | 10 | | | | | |
| 5 | SP-SM | Poorly Graded SAND with silt - medium dense, slightly moist, tan-grey, sand is fine-grained | 8 | 16 | | | | | |
| | | | 8 | 15 | | | | | |
| 10 | | | 8 | 14 | | 4.4 | 11.2 | NP | NP |
| 15 | | | 14 | 26 | | | | | |
| 20 | | - dense | 17 | 32 | | | | | |
| 21.5 | | Bottom of Boring @ 21.5 Feet | | | | | | | |

SPT BLOW COUNT



Moisture Content and Atterberg Limits

Plastic Limit — Moisture Content — Liquid Limit



N - OBSERVED UNCORRECTED BLOW COUNT

N* - CORRECTED N1(60) EQUIVALENT SPT BLOW COUNT

GeoStrata



SAMPLE TYPE
- 2" O.D./1.38" I.D. Split Spoon Sampler
- 2.5" O.D./2" I.D. California Split Spoon Sampler
- 3" O.D. Thin-Walled Shelby Sampler
- Grab Sample
- 2" O.D./1.625" I.D. Liner Sampler

NOTES:

WATER LEVEL
▼ - MEASURED ▽ - ESTIMATED

**Plate B - 3**

| DATE | |
|---|---|
| STARTED: | 4/8/20 |
| COMPLETED: | 4/8/20 |
| BACKFILLED: | 4/8/20 |

Maverik, Inc.
Fountain Blvd & Union Blvd
Colorado Springs, CO
Project Number 1092-066

GeoStrata Rep: J. Sage
Rig Type: CME 55
Boring Type: HSA

BORING NO: **B-4**



LOCATION: STATION | OFFSET | ELEVATION

| Depth (feet) | Samples | USCS | Material Description | N | N* | Dry Density (pcf) | Moisture Content % | Percent minus 200 | Liquid Limit | Plasticity Index |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | SP-SM | Poorly Graded SAND with silt - medium dense, slightly moist, tan-grey, sand is fine-grained | | | | | | | |
| ~2.5 | Split spoon | | | 8 | 16 | | | | | |
| ~5 | Split spoon | | | 7 | 14 | | 4.7 | 11.3 | NP | NP |
| ~7.5 | Split spoon | | - dense | 18 | 33 | | | | | |
| ~10 | Split spoon | | - medium dense | 12 | 21 | | | | | |
| ~15 | Split spoon | | | 16 | 29 | | | | | |
| ~20 | Split spoon | | - dense | 23 | 43 | | | | | |
| 21.5 | | | Bottom of Boring @ 21.5 Feet | | | | | | | |

N - OBSERVED UNCORRECTED BLOW COUNT

N* - CORRECTED N1(60) EQUIVALENT SPT BLOW COUNT

SAMPLE TYPE
- 2" O.D./1.38" I.D. Split Spoon Sampler
- 2.5" O.D./2" I.D. California Split Spoon Sampler
- 3" O.D. Thin-Walled Shelby Sampler
- Grab Sample
- 2" O.D./1.625" I.D. Liner Sampler

NOTES:

WATER LEVEL
- MEASURED
- ESTIMATED

GeoStrata



**Plate**

B - 4

## UNIFIED SOIL CLASSIFICATION SYSTEM

| MAJOR DIVISIONS | | | USCS SYMBOL | TYPICAL DESCRIPTIONS |
|---|---|---|---|---|
| COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve) | GRAVELS (More than half of coarse fraction is larger than the #4 sieve) | CLEAN GRAVELS WITH LITTLE OR NO FINES | GW | WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES |
| | | | GP | POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES |
| | | GRAVELS WITH OVER 12% FINES | GM | SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES |
| | | | GC | CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES |
| | SANDS (More than half of coarse fraction is smaller than the #4 sieve) | CLEAN SANDS WITH LITTLE OR NO FINES | SW | WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES |
| | | | SP | POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES |
| | | SANDS WITH OVER 12% FINES | SM | SILTY SANDS, SAND-GRAVEL-SILT MIXTURES |
| | | | SC | CLAYEY SANDS SAND-GRAVEL-CLAY MIXTURES |
| FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve) | SILTS AND CLAYS (Liquid limit less than 50) | | ML | INORGANIC SILTS & VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, CLAYEY SILTS WITH SLIGHT PLASTICITY |
| | | | CL | INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS |
| | | | OL | ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY |
| | SILTS AND CLAYS (Liquid limit greater than 50) | | MH | INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT |
| | | | CH | INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS |
| | | | OH | ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY |
| HIGHLY ORGANIC SOILS | | | PT | PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS |

## LOG KEY SYMBOLS

BORING SAMPLE LOCATION

TEST-PIT SAMPLE LOCATION

WATER LEVEL (level after completion)

WATER LEVEL (level where first encountered)

## CEMENTATION

| DESCRIPTION | DESCRIPTION |
|---|---|
| WEAKELY | CRUMBLES OR BREAKS WITH HANDLING OR SLIGHT FINGER PRESSURE |
| MODERATELY | CRUMBLES OR BREAKS WITH CONSIDERABLE FINGER PRESSURE |
| STRONGLY | WILL NOT CRUMBLE OR BREAK WITH FINGER PRESSURE |

## OTHER TESTS KEY

| | | | |
|---|---|---|---|
| C | CONSOLIDATION | SA | SIEVE ANALYSIS |
| AL | ATTERBERG LIMITS | DS | DIRECT SHEAR |
| UC | UNCONFINED COMPRESSION | T | TRIAXIAL |
| S | SOLUBILITY | R | RESISTIVITY |
| O | ORGANIC CONTENT | RV | R-VALUE |
| CBR | CALIFORNIA BEARING RATIO | SU | SOLUBLE SULFATES |
| COMP | MOISTURE/DENSITY RELATIONSHIP | PM | PERMEABILITY |
| CI | CALIFORNIA IMPACT | -200 | % FINER THAN #200 |
| COL | COLLAPSE POTENTIAL | Gs | SPECIFIC GRAVITY |
| SS | SHRINK SWELL | SL | SWELL LOAD |

## MODIFIERS

| DESCRIPTION | % |
|---|---|
| TRACE | <5 |
| SOME | 5 - 12 |
| WITH | >12 |

## MOISTURE CONTENT

| DESCRIPTION | FIELD TEST |
|---|---|
| DRY | ABSENCE OF MOISTURE, DUSTY, DRY TO THE TOUCH |
| MOIST | DAMP BUT NO VISIBLE WATER |
| WET | VISIBLE FREE WATER, USUALLY SOIL BELOW WATER TABLE |

## STRATIFICATION

| DESCRIPTION | THICKNESS |
|---|---|
| SEAM | 1/16 - 1/2" |
| LAYER | 1/2 - 12" |

| DESCRIPTION | THICKNESS |
|---|---|
| OCCASIONAL | ONE OR LESS PER FOOT OF THICKNESS |
| FREQUENT | MORE THAN ONE PER FOOT OF THICKNESS |

## GENERAL NOTES

1. Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual.
2. No warranty is provided as to the continuity of soil conditions between individual sample locations.
3. Logs represent general soil conditions observed at the point of exploration on the date indicated.
4. In general, Unified Soil Classification designations presented on the logs were evaluated by visual methods only. Therefore, actual designations (based on laboratory tests) may vary.

## APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

| APPARENT DENSITY | SPT (blows/ft) | MODIFIED CA. SAMPLER (blows/ft) | CALIFORNIA SAMPLER (blows/ft) | RELATIVE DENSITY (%) | FIELD TEST |
|---|---|---|---|---|---|
| VERY LOOSE | <4 | <4 | <5 | 0 - 15 | EASILY PENETRATED WITH 1/2-INCH REINFORCING ROD PUSHED BY HAND |
| LOOSE | 4 - 10 | 5 - 12 | 5 - 15 | 15 - 35 | DIFFICULT TO PENETRATE WITH 1/2-INCH REINFORCING ROD PUSHED BY HAND |
| MEDIUM DENSE | 10 - 30 | 12 - 35 | 15 - 40 | 35 - 65 | EASILY PENETRATED A FOOT WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER |
| DENSE | 30 - 50 | 35 - 60 | 40 - 70 | 65 - 85 | DIFFICULT TO PENETRATED A FOOT WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER |
| VERY DENSE | >50 | >60 | >70 | 85 - 100 | PENETRATED ONLY A FEW INCHES WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER |

## CONSISTENCY - FINE-GRAINED SOIL

| CONSISTENCY | SPT (blows/ft) | TORVANE UNTRAINED SHEAR STRENGTH (tsf) | POCKET PENETROMETER UNCONFINED COMPRESSIVE STRENGTH (tsf) | FIELD TEST |
|---|---|---|---|---|
| VERY SOFT | <2 | <0.125 | <0.25 | EASILY PENETRATED SEVERAL INCHES BY THUMB. EXUDES BETWEEN THUMB AND FINGERS WHEN SQUEEZED BY HAND. |
| SOFT | 2 - 4 | 0.125 - 0.25 | 0.25 - 0.5 | EASILY PENETRATED ONE INCH BY THUMB. MOLDED BY LIGHT FINGER PRESSURE. |
| MEDIUM STIFF | 4 - 8 | 0.25 - 0.5 | 0.5 - 1.0 | PENETRATED OVER 1/2 INCH BY THUMB WITH MODERATE EFFORT. MOLDED BY STRONG FINGER PRESSURE. |
| STIFF | 8 - 15 | 0.5 - 1.0 | 1.0 - 2.0 | INDENTED ABOUT 1/2 INCH BY THUMB BUT PENETRATED ONLY WITH GREAT EFFORT. |
| VERY STIFF | 15 - 30 | 1.0 - 2.0 | 2.0 - 4.0 | READILY INDENTED BY THUMBNAIL. |
| HARD | >30 | >2.0 | >4.0 | INDENTED WITH DIFFICULTY BY THUMBNAIL. |

GeoStrata



# Soil Symbols Description Key

Maverik, Inc.
Fountain Blvd & Union Blvd
Colorado Springs, CO
Project Number: 1092-066

**Plate B-5**

# APPENDIX C

| Boring No. | Sample Depth (feet) | USCS Soil Classification | Natural Moisture Content (%) | Optimum Moisture Content (%) | Maximum Dry Density (pcf) | Gradation | | | Atterberg | | CBR (%) | Sulfate Content (ppm) | Resistivity (Ω-cm) | pH |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Gravel (%) | Sand (%) | Fines (%) | LL | PI | | | | |
| B-1 | 5 | SP-SM | 4.6 | | | 0.0 | 89.7 | 10.3 | NP | NP | | | | |
| B-2 | 2.5 | SP-SM | | | | | | | | | | 5.56 | 9500 | 8.41 |
| B-2 | 7.5 | SP-SM | 2.8 | | | 0.0 | 90.1 | 9.9 | NP | NP | | | | |
| B-3 | 0.5 | SM | | 10.2 | 119.5 | 0.8 | 82.8 | 16.4 | NP | NP | 2.7 | | | |
| B-3 | 10 | SP-SM | 4.4 | | | 0.0 | 88.8 | 11.2 | NP | NP | | | | |
| B-4 | 5 | SP-SM | 4.7 | | | 0.0 | 88.7 | 11.3 | NP | NP | | | | |

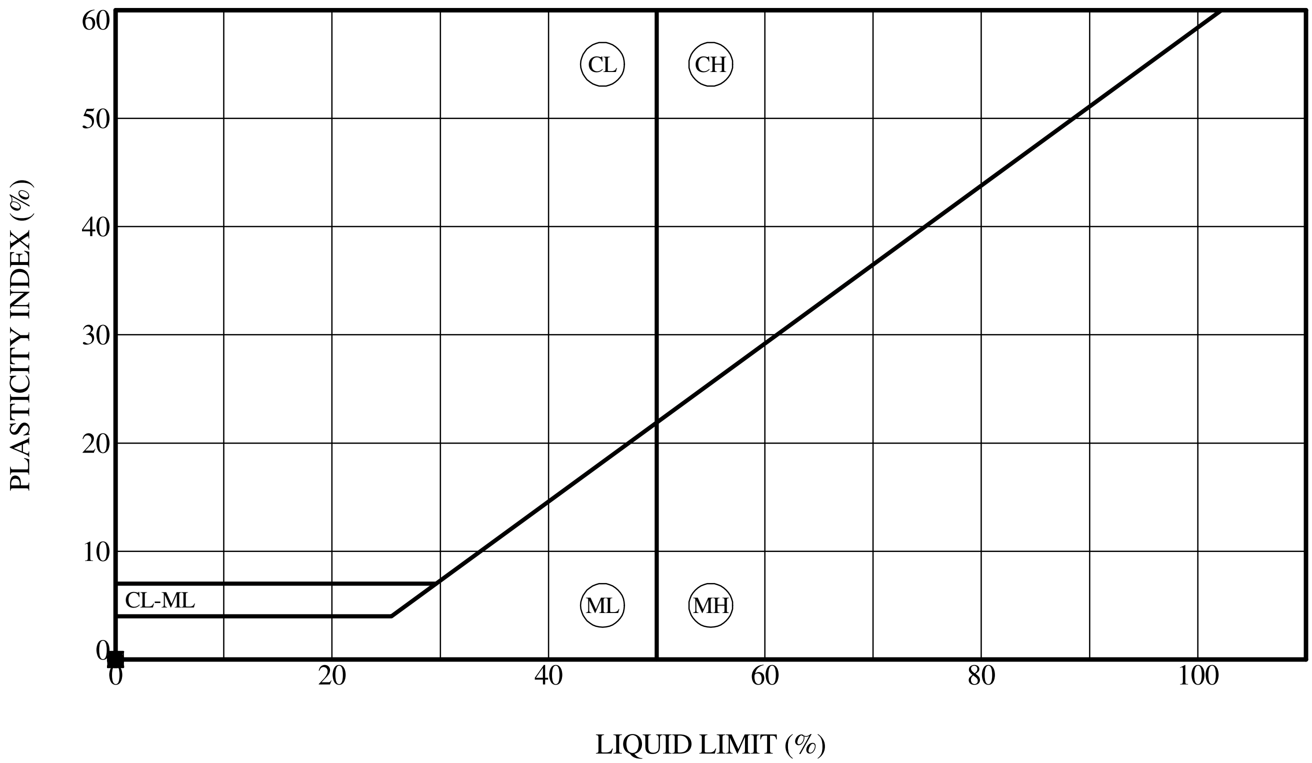
GeoStrata



## Lab Summary Report

Maverik, Inc.
Fountain Blvd & Union Blvd
Colorado Springs, CO
Project Number: 1092-066

**Plate C - 1**




| | Sample Location | Depth (ft) | LL (%) | PL (%) | PI (%) | Fines (%) | Classification |
|---|---|---|---|---|---|---|---|
| ● | **B-1** | **5.0** | **NP** | **NP** | **NP** | **10.3** | **Poorly Graded SAND with silt** |
| ⊠ | **B-2** | **7.5** | **NP** | **NP** | **NP** | **9.9** | **Poorly Graded SAND with silt** |
| ▲ | **B-3** | **0.5** | **NP** | **NP** | **NP** | **16.4** | **Silty SAND** |
| ★ | **B-3** | **10.0** | **NP** | **NP** | **NP** | **11.2** | **Poorly Graded SAND with silt** |
| ⊙ | **B-4** | **5.0** | **NP** | **NP** | **NP** | **11.3** | **Poorly Graded SAND with silt** |

GeoStrata

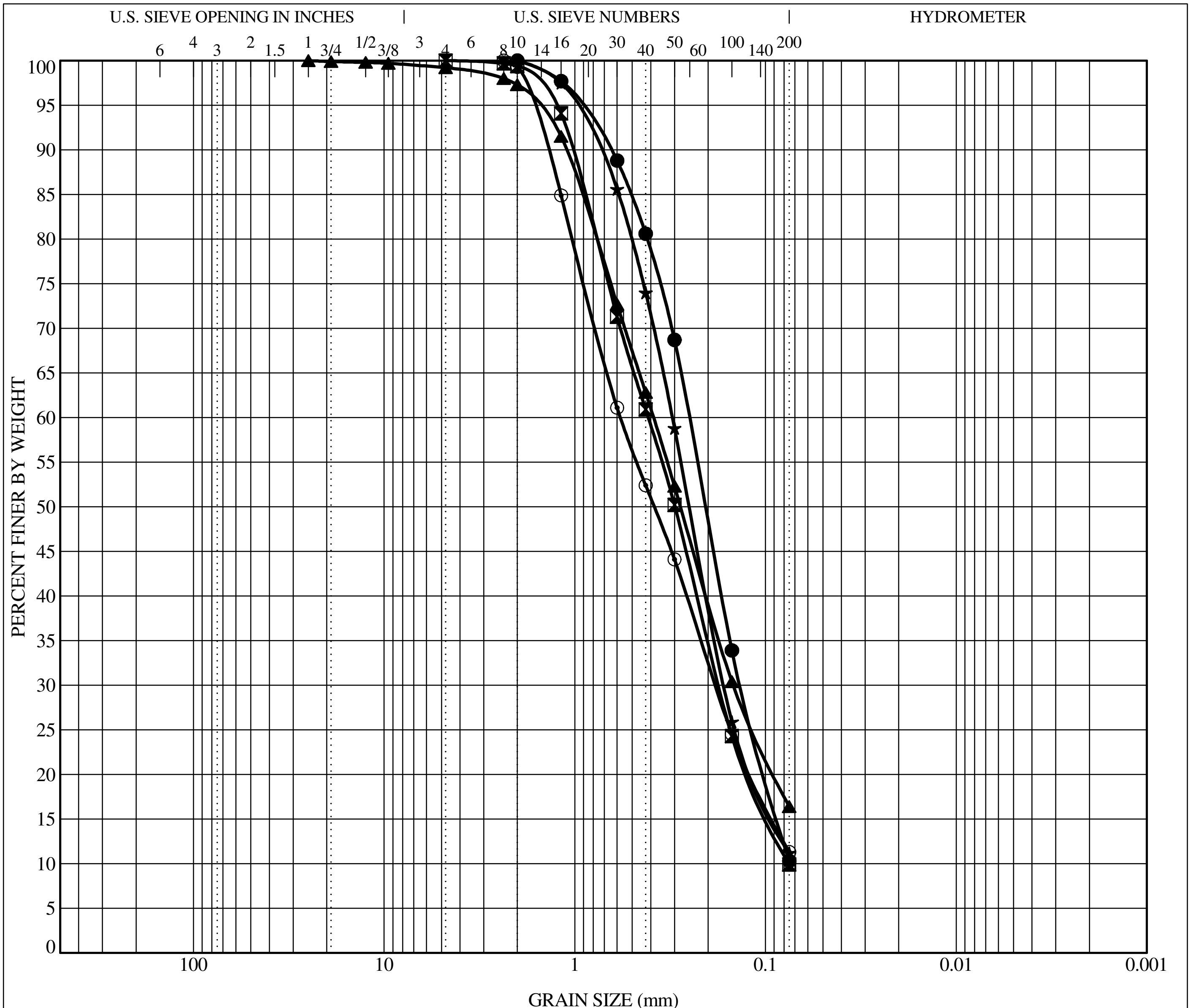
**ATTERBERG LIMITS' RESULTS - ASTM D 4318**

Maverik, Inc.
Fountain Blvd & Union Blvd
Colorado Springs, CO
Project Number: 1092-066

**Plate**

**C - 2**

C_ATTERBERG CS MAVERIK.GPJ GEOSTRATA.GDT 4/23/20




| COBBLES | GRAVEL | | SAND | | | SILT OR CLAY |
|---|---|---|---|---|---|---|
| | coarse | fine | coarse | medium | fine | |

| | Sample Location | Depth | Classification | LL | PL | PI | Cc | Cu |
|---|---|---|---|---|---|---|---|---|
| ● | **B-1** | **5.0** | **Poorly Graded SAND with silt** | **NP** | **NP** | **NP** | **0.95** | **3.39** |
| ⊠ | **B-2** | **7.5** | **Poorly Graded SAND with silt** | **NP** | **NP** | **NP** | **0.98** | **5.48** |
| ▲ | **B-3** | **0.5** | **Silty SAND** | **NP** | **NP** | **NP** | | |
| ★ | **B-3** | **10.0** | **Poorly Graded SAND with silt** | **NP** | **NP** | **NP** | **1.22** | **4.35** |
| ⊙ | **B-4** | **5.0** | **Poorly Graded SAND with silt** | **NP** | **NP** | **NP** | **0.83** | **8.20** |

| | Sample Loctaion | Depth | D100 | D60 | D30 | D10 | %Gravel | %Sand | %Silt | %Clay |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | **B-1** | **5.0** | **2** | **0.252** | **0.134** | | **0.0** | **89.7** | **10.3** | |
| ⊠ | **B-2** | **7.5** | **4.75** | **0.413** | **0.175** | **0.075** | **0.0** | **90.1** | **9.9** | |
| ▲ | **B-3** | **0.5** | **25** | **0.387** | **0.147** | | **0.8** | **82.8** | **16.4** | |
| ★ | **B-3** | **10.0** | **2** | **0.308** | **0.164** | | **0.0** | **88.8** | **11.2** | |
| ⊙ | **B-4** | **5.0** | **2.36** | **0.574** | **0.182** | | **0.0** | **88.7** | **11.3** | |

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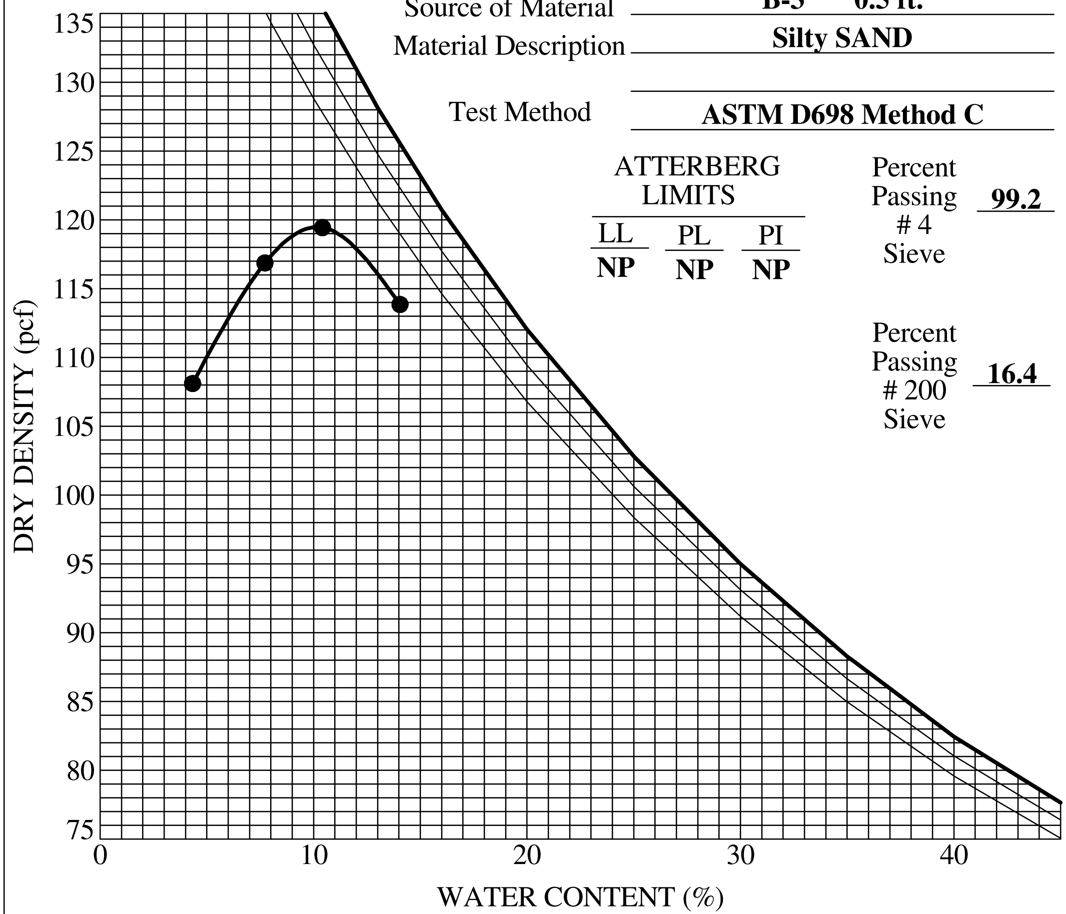
**GRAIN SIZE DISTRIBUTION - ASTM D422**

Maverik, Inc.
Fountain Blvd & Union Blvd
Colorado Springs, CO
Project Number: 1092-066

**Plate**

**C - 3**




Source of Material: B-3 0.5 ft.

Material Description: Silty SAND

Test Method: ASTM D698 Method C

| ATTERBERG LIMITS | | |
|---|---|---|
| LL | PL | PI |
| **NP** | **NP** | **NP** |

Percent Passing # 4 Sieve: **99.2**

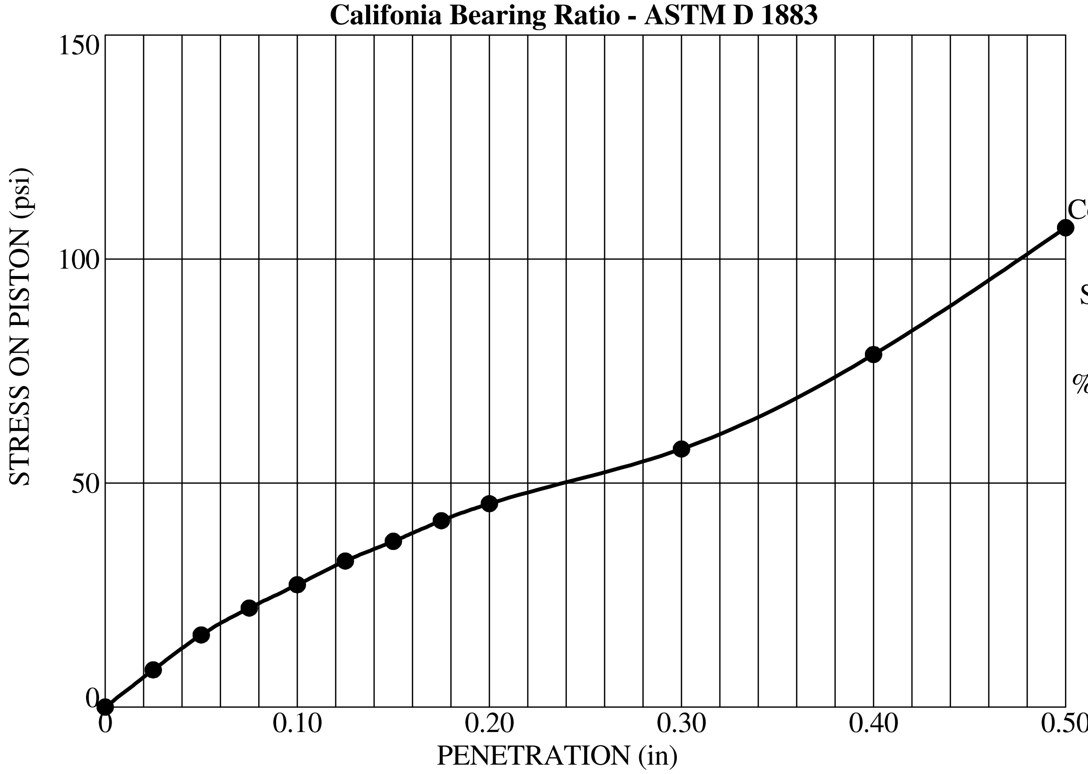
Percent Passing # 200 Sieve: **16.4**

| TEST RESULTS | | |
|---|---|---|
| Maximum Dry Density | **119.5** | (pcf) |
| Optimum Water Content | **10.2** | (%) |
| Percent Rock | **---** | (%) |
| Corrected Maximum Dry Density | **---** | (pcf) |
| Corrected Optimum Water Content | **---** | (%) |

Curves of 100% Saturation for Specific Gravity Equal to:

2.60, 2.70, 2.80

**Califonia Bearing Ratio - ASTM D 1883**




| | | |
|---|---|---|
| Dry Density | **114.7** | (pcf) |
| Relative Compaction | **96** | (%) |
| Surcharge | **50** | (psf) |
| % Standard CBR | **2.70** | |
| Swell | **0.09** | (%) |

GeoStrata

# COMPACTION AND CBR TEST

Maverik, Inc.
Fountain Blvd & Union Blvd
Colorado Springs, CO
Project Number: 1092-066

**Plate C - 4**

# APPENDIX D

# Important Information about This

# Geotechnical-Engineering Report

**Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.**

**While you cannot eliminate all such risks, you can manage them. The following information is provided to help.**

**The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.**

## Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

## Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

*Do not rely on this report* if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

## Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

## You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

## Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

## This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are not final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

## This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

## Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

## Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

## Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

## Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

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