GEOTECHNICAL INVESTIGATION
MONTEREY-SALINAS TRANSIT DISTRICT
OPERATIONS AND MAINTENANCE FACILITY
1 RYAN RANCH ROAD
MONTEREY, CALIFORNIA

KLF PROJECT #20153715.001A

JANUARY 15, 2015

January 15, 2015
Project No. 20153715.001A

AECOM
2101 Webster Street, Suite 1900
Oakland, California 94612

Attention: Mr. Rob McKie

SUBJECT: Geotechnical Engineering Investigation for the Proposed Monterey-Salinas Transit District Operations and Maintenance Facility Expansion in Monterey, California

Dear Mr. McKie:

We are pleased to submit our geotechnical investigation report for the proposed Monterey Salinas Transit District Operations and Maintenance Facility Expansion to be located at 1 Ryan Ranch Road in Monterey, California. The accompanying report provides the results of our field investigation, laboratory testing, and engineering analyses. Geotechnical design recommendations are presented for site preparation, grading, engineered fill, surface drainage, utility trench backfill, foundations, retaining walls and seismic design parameters. In addition, we have provided the results of the percolation testing which was performed at the site.

The primary geotechnical considerations at this site are the presence of moderately expansive near surface soils on portions of the site, collapse potential of saturated on-site soils used as engineered fill, erosion of cut and fill slopes, low subgrade support strength of on-site soils in the bus parking areas, and lower permeability of soils containing clayey fines and/or decomposed sandstone at depth.

The moderately expansive soils will require a layer of "non-expansive" fill, or a thickened rock section under the proposed building additions and exterior concrete slabs-on-grade. The collapse potential of saturated on-site soils used as engineered fill will limit the use of the onsite soils for use as fill when subjected to a combination of high loads and potential saturation. Buried stormwater management systems will need to be located in areas that will not be subjected to high ground pressures that exist in areas such as foundation zones and to a lesser extent the bus parking areas. Such improvements will need to be located in light weight vehicle parking areas and landscape areas. Cut and fill slopes will need to protected from erosion. Additionally, animal burrows in the existing slopes could result in piping failures downslope of the buried stormwater management system. This will need to be mitigated by grading or site selection for the system. Low subgrade support strength of on-site soils in the bus parking areas will result in a slight increase in the asphalt concrete and baserock sections recommended for flexible pavements, and a slight increase in the baserock section for Portland cement concrete pavements. Finally, the lower permeability of deeper on-site soils containing clayey fines and/or

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decomposed sandstone will limit the total embedment depth of the buried stormwater management system. These items are discussed in the report.

Based on the results of our investigation and from a geotechnical standpoint, we judge that the proposed improvements may be developed as planned provided the recommendations in the attached report including appendices are incorporated into the design and construction of the project.

As noted in our report, Kleinfelder should be retained to review pre-final project plans and specifications prior to the start of construction, and to observe and test during earthwork and foundation construction. This will allow us to compare conditions exposed during construction with those encountered during our investigation and to present supplemental recommendations if warranted by different site conditions.

We appreciate the opportunity of providing our services to you on this project. If any questions should arise regarding the interpretation of the contents of this report, please contact us at 831.755. 7900.

Sincerely,

KLEINFELDER, INC.

Robert Hasseler, CE 58488
Project Engineer

Brian O'Neill, PE, GE 2516
Principal Geotechnical Engineer

Jeff Richmond, CEG 2425
Project Professional

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Table of Contents

GEOTECHNICAL INVESTIGATION
MONTEREY-SALINAS TRANSIT DISTRICT
OPERATIONS AND MAINTENANCE FACILITY
1 RYAN RANCH ROAD
MONTEREY, CALIFORNIA

Table of Contents
(Continued)

GEOTECHNICAL INVESTIGATION
MONTEREY-SALINAS TRANSIT DISTRICT
OPERATIONS AND MAINTENANCE FACILITY
1 RYAN RANCH ROAD
MONTEREY, CALIFORNIA

1. INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed Monterey Salinas Transit District Operations and Maintenance Facility Expansion to be located at 1 Ryan Ranch Road in Monterey, California. The approximate location of the project is shown on the Site Vicinity Map, Plate 1. The locations of our borings and percolation tests are shown on the Site Plan, Plate 2. Proposed improvements are shown on Plate 3. This geotechnical investigation was performed for AECOM and Monterey Salinas Transit District.

The conclusions and recommendations presented in this report are based on the subsurface soil conditions encountered at the locations of our exploration, and the provisions and requirements outlined in the Limitations section of this report. The findings, conclusions and recommendations presented herein should not be extrapolated to other areas or be used for other projects without our review.

1.1 PROJECT DESCRIPTION

We understand that the project consists of an overhaul of the existing Monterey Salinas Transit District Operations and Maintenance Facility. The improvements will include new additions and renovations to the existing site buildings, bus wash renovation, and enlargement of the existing fuel canopy, grading, pavements, retaining walls, on-site storm water management and new buried utilities.

The new additions are anticipated to be tall one-story lightweight structures with concrete slab-on-grade floors and conventional spread footing foundations. Specific details regarding structure loading are not known at this time. It is anticipated that the retaining walls will be supported on deepened conventional footing foundations or cast in drilled hole piles (i.e. drilled piers). Canopy foundations would be either conventional spread footings or pier foundations.

1.2 PURPOSE AND SCOPE OF SERVICES

The purpose of this geotechnical investigation was to evaluate subsurface soil conditions at the site of the proposed improvements; and to provide geotechnical recommendations pertaining to earthwork and the foundation aspects of the project. The scope of services performed for this geotechnical investigation consisted of a site reconnaissance, subsurface exploration, laboratory testing, engineering analysis of field and laboratory data, and preparation of this report. Percolation testing was also performed as part of this investigation. The data obtained and analyses performed were for the purpose of providing design and construction recommendations for site preparation and grading, utility trench excavation and backfilling, building and canopy foundations, retaining walls, and site drainage.

Environmental services such as evaluation and chemical analysis of the soil and groundwater for hazardous materials were not included in our scope of services.

2. SITE INVESTIGATION

2.1 SITE DESCRIPTION

The project site is located at 1 Ryan Ranch Road in Monterey, California. The location of the site is shown on the Site Vicinity Map, Plate 1, in the Appendix. The site is bounded to the west by Canyon Del Rey Road, to the south by Ryan Ranch Road, to the north by Del Rey Gardens Drive and a parking lot (not part of the project site) to the northeast, and to the east by undeveloped land. Site access is to the southeast towards Ryan Ranch Road.

The irregularly shaped project site contains several buildings, a bus wash and fuel area, conventional asphalt concrete parking lots and bus parking, sidewalks and landscaping areas. The Site Plan, Plate 2 in the Appendix, shows the existing layout of the site and the location of our exploratory borings and percolation test holes. The upper, developed portion of the site is relatively level with only mild slopes. Between the upper developed portion of the site and the bounding roads to the north, west and south, the ground slopes moderately downwards. These areas are vegetated with trees, brush and grass. To the northeast the ground slopes moderately upwards towards the bounding parking lot to the northeast, and has similar vegetation. The eastern project site parking lot is separated from the main developed part of the site by landscaping. The eastern parking lot is slightly lower than the rest of the site. The perimeter of the eastern parking lot consists of vegetated slopes which slope upwards to the north and west and downwards to the south. Site drainage is generally to the southeast.

2.2 SITE RECONNAISSANCE

A Certified Engineering Geologist from our firm performed a site reconnaissance on December 3, 2014 to observe the current site conditions. The focus of the reconnaissance was to identify potential geologic hazards that could impact the proposed improvements. The following section describes the potential geologic hazards identified during the reconnaissance.

Multiple landslides, landslide scars and erosion rills were identified on the cut slopes adjacent to Canyon Del Rey Boulevard, indicating the slopes are currently in an over-steepened configuration for the poorly to non-indurated deposits exposed. While no evidence of incipient failure was observed above the hinge point of the slope, future up-slope migration of the landslides should be anticipated as the slope attempts to reach its angle of repose. The slopes are beyond the site property boundary, and the most proximal slope is approximately 15 feet in height and currently 100 feet from the proposed improvements. As such, future failure migration will not likely impact the site or proposed improvements.

A broad colluvial swale exists along the southwest property line. A shallow landslide previously occurred where the drainage intersects the Canyon Del Rey Boulevard cut slope and is approximately 85 feet from the proposed improvements. The landslide was previously investigated by Tharp (1993) and Weber-Hayes (1993), but remains largely unmitigated. The drainage is configured at approximately 6.5H:1V (Horizontal:Vertical) and has been locally disturbed by subsurface utility installation. While the existing landslide and colluvial drainage do not necessarily represent a slope stability hazard, thickened weak and/or porous soil should be anticipated within limits of the drainage.

Existing cut slopes throughout the site configured at 1H:1V exhibit accelerated erosion and shallow failures, locally. Slopes which expose silty sand deposits appear most susceptible. The existing slopes and any proposed slopes constructed in a similarly over-steepened configuration will continue to fail, and require future maintenance and/or stabilization.

Existing and proposed improvements constructed on or in close proximity to slopes are susceptible to creep disturbance, particularly during periods of saturation. Disturbance (settlement, lateral movement) of curb and gutter was noted on the site, particularly along the south perimeter.

Abundant large animal burrows where observed throughout the slope located west of the site entrance and directly down slope of percolation test P-2. During periods of saturation, the burrows could potentially contribute to piping of perched groundwater and contribute to slope instability in the area.

Provided the recommendations in this report are incorporated into the design and construction of the proposed improvements at the site, the conditions described above should not adversely affect the project.

2.3 SUBSURFACE INVESTIGATION

Our field investigation consisted of a site reconnaissance and a subsurface exploration program. On November 20 and 21, 2014, nine exploratory borings were drilled to depths of between 5 feet and 30 feet below existing ground surface near the proposed locations of the new facilities using a truck-mounted drill rig equipped with 8-inch diameter hollow stem augers. The approximate locations of our borings are shown on the Site Plan, Plate 2.

The soils encountered in the borings were visually classified in the field, and our engineering staff recorded a log of each boring. Soil samples were obtained from the borings by driving either a 2½ inch inside diameter California tube sampler, or a $1^3/_8$ inch inside diameter Standard Penetration (SPT) split-spoon sampler up to a depth of 18 inches into the underlying soil with a 140-pound hammer falling 30 inches. The number of blows required to drive the sampler was recorded for each 6-inch penetration interval. The borings were backfilled with drilling spoils and were capped with concrete in pavement areas.

Our field engineering staff made visual classification of the soils encountered in our exploratory borings in general accordance with the Unified Soil Classification System (ASTM D2487 and D2488). Keys for the classification of the soil are presented in Appendix A on the Graphics Key, Plate A-1 and the Soil Description Key, Plate A-2. The logs of the borings are presented on Plates A-3 to A-11 in Appendix A.

2.4 PERCOLATION TESTING

On November 21, 2014 three percolation tests were completed to evaluate the average percolation rates of the near-surface soils. The test holes were drilled on November 20 using a truck-mounted drill rig equipped with 8-inch diameter hollow stem augers to a depth of approximately 15 feet below the existing ground surface. A section of 4-inch diameter slotted PVC pipe was installed in the bottom 5 feet of each test hole and solid PVC pipe was installed in the upper 10 feet of each hole. The bottom 6 feet of the hole was backfilled with gravel and a 12 inch thick bentonite seal was installed above the gravel. The holes were then pre-saturated by filling the holes with water. The percolation testing was performed on November 21, 2014 using a 10-minute measurement interval. The locations of the percolation tests are shown as P-1 to P-3 on the Site Plan, Plate 2. The results of the percolation testing are included in Appendix B.

2.5 GEOTECHNICAL LABORATORY TESTING

Representative soil samples were obtained from the exploratory borings at selected depths. The samples were returned to our laboratory for further evaluation and testing. Laboratory testing of selected soil samples from the borings was conducted to evaluate the natural moisture content and density, grain size distribution, Atterberg limits, direct shear strength, one dimensional swell/collapse potential, and R-value. Most of the laboratory test results are presented on the individual boring logs. A summary of our laboratory tests results is presented on Plate C-1. The results of the grain size distribution test are shown on Plate C-2. The results of the Atterberg limits tests are shown on Plate C-3. The results of the direct shear strength test are shown on Plate C-4. The results of the one dimensional swell test are shown on Plate C-5, and the results of the R-Value tests are shown on Plates C-6 and C-7.

2.6 CORROSION POTENTIAL TESTING

A representative near surface soil sample was sent to CERCO Analytical laboratory to evaluate the potential corrosivity of onsite soils. Laboratory chloride concentration, sulfate concentration, pH, oxidation reduction potential, and electrical resistivity tests were performed for the selected soil sample. The results of the tests and a summary letter from CERCO Analytical are attached in Appendix D. If fill materials will be imported to the project site, similar corrosion potential laboratory testing should be completed on the imported material.

Our scope of services does not include corrosion engineering and therefore a detailed analysis of the corrosion test results is not included in this report. A qualified corrosion engineer should be retained to review the test results and design protective systems that may be required. Kleinfelder is able to provide those services if requested.

2.7 SUBSURFACE CONDITIONS

Ground cover in the northern parking areas and bus parking areas (B-1 through B-3 and B-8 and B-9) consisted of about 3½ to 6 inches thickness of asphalt concrete overlying approximately 3 to 6 inches of aggregate base material. Pavement thickness in the southern parking lot (B-6) was approximately 3 inches of asphalt concrete over 4 inches of aggregate base material over silty sand with gravel fill to a depth of about 9 feet below existing grade. Pavement thickness in the eastern parking lot (B-7) was approximately 1½ inches of asphalt concrete over 2 inches of aggregate base material. Boring B-4 was drilled on an asphalt covered fill pad on the west side of the site and ground cover consisted of approximately 2 inches of asphalt concrete over 6 inches of aggregate base material over silty sand fill to a depth of about 3½ feet below existing grade. Boring B-5 was drilled in landscaping and consisted of about 2 inches of aggregate base material.

Subsurface soils consisted primarily of silty sands and clayey sands with some poorly graded sands. These soils extended to a depth of at least 30 feet in Borings B-2, B-3 and B-7. In Borings B-1, B-4, B-5 and B-6 the near surface soils overlaid decomposed to highly weathered weak sandstone, which was encountered at a depth of approximately 8, 3½, 1½ and 9 feet below existing grade, respectively. The coarse grained soils underlying the site were typically medium dense to very dense except for the silty sand fill layer to a depth of about 3½ feet at Boring B-4, which was loose.

Groundwater was not encountered in our borings or percolation test holes. However, it must be noted that seasonal fluctuations in the groundwater level may occur due to variations in rainfall, temperature, groundwater withdrawal, and possibly other factors that were not evident at the time of our investigation. Due to the slow percolation rates in the clayey sand layers below the silty sand layers in our percolation test holes, there is a potential for seasonally perched groundwater at various depths.

The above is a general description of the subsurface conditions encountered in our exploratory borings at the project site. Additional information is provided on the Logs of Borings in Appendix A. Soil and groundwater conditions can deviate from those conditions encountered at the boring locations. Should this be revealed during construction, Kleinfelder should be notified immediately for possible revisions to the recommendation that follow.

3. GEOLOGY AND SEISMIC DESIGN

3.1 REGIONAL GEOLOGY

The site is located approximately 2.5 miles inland (southeast) of Monterey Bay, within the Coast Ranges Geomorphic Province of Central California. This Province is comprised of a discontinuous series of northwest-southeast trending mountain ranges, ridges, and intervening valleys characterized by complex folding and faulting and the dominant structural regime in the region. Geologic structure within the Coast Ranges Province is generally controlled by the San Andreas fault system, which is a major tectonic transform plate boundary. This right-lateral strike-slip fault system extends from the Gulf of California in Mexico, to Cape Mendocino in northern California and forms a portion of the boundary between two tectonic plates. In this portion of the Coast Ranges Province, the Pacific plate (located west of the transform boundary) moves north relative to the North American plate (located east of the transform boundary). Deformation along this plate boundary occurs across a wide zone that is referred to as the San Andreas fault system.

The site is located within the Salinian Block, which is one of the distinct continental terranes of the central Coast Ranges. In the region, the Salinian Block is bounded by the San Andreas fault on the east and the Sur-Nacimiento fault zone on the west (Page, 1966). This basement rock of this block is composed of Cretaceous age (about 140 to 65 million years old) granitic and high-grade metamorphic rocks. Major orogenic features within the Salinian Block in the vicinity of the site include the Gabilan Range to the east/northeast, the Sierra de Salinas to the southeast, and the Santa Lucia Range to the southwest.

Overlying the granitic basement rocks of the Salinian block are Cretaceous and Tertiary (about 65 to 1.8 million years old), marine and continental sedimentary rocks and occasional Tertiary volcanic rocks. These Cretaceous and Tertiary age rocks are typically folded and faulted into a series of generally northwest-southeast trending blocks, largely as a result of stresses related to movement along the San Andreas fault system. The inland valleys, including Salinas Valley, are filled with unconsolidated to semi-consolidated alluvium (stream channel and over-bank deposits) of Quaternary age (1.8 million years old to current). In the vicinity of the project site, the bedrock is overlain by Quaternary age terrace deposits, eolian sands, and alluvium.

3.2 SITE GEOLOGY

The geology of the site has been mapped by Dibblee and Minch (2007), Dupre (1990) and Clark et al. (1997), among others. Dibblee and Minch (2007) map the site as underlain by Quaternary age, dissected older alluvium. The reference indicates the Canyon Del Rey Boulevard road cut southwest of the site exposes diatomite bedrock of the Miocene age (23 to 5.3 million years old) Monterey Formation. Dupre (1990) and Clark, Dupre and Rosenberg (1997) show the site to be underlain by Quaternary age (Pleistocene) coastal terrace deposits, described as semi-consolidated, well sorted marine sand, locally indurated and containing thin discontinuous gravel layers. Comparable to Dibblee and Minch (2007), Dupre (1990) indicates the road cut exposes undivided, pre-Quaternary bedrock. Clark et al. (1997) have also mapped the exposure as Monterey formation diatomite bedrock, described as pale orange to white, punky and locally silty. The reference indicates the terrace deposits and diatomite are in faulted contact along the southwestern trace of the Chupines fault.

All three (3) references show the drainage within which Canyon Del Rey Boulevard and the lower section of Ryan Ranch Road are located as being underlain by Holocene age (11,000 years old to present). Dupre (1990) characterizes this deposit as highly susceptible to liquefaction, while the terrace deposits and bedrock underlying the site are characterized as having very low liquefaction susceptibility.

3.3 FAULTING AND SEISMICITY

According to the California Geological Survey (CGS, 2010), the site is not located within an Alquist-Priolo Earthquake Fault Zone. The nearest zoned active fault is the creeping section of the San Andreas fault, located approximately 24.3 miles northeast of the site, which is capable of producing a maximum earthquake magnitude event of M8.05. Moderate to major earthquakes generated on the San Andreas fault can be expected to cause strong ground shaking at the site.

The United States Geological Survey (USGS) Quaternary Fault and Fold Database (available at: http://earthquake.usgs.gov/hazards/qfaults/map/) identifies several other faults within the site vicinity. Table 3.1 below identifies the significant faults in the area and their corresponding parameters. In addition, the database indicates the southwest trace of the Chupines fault zone (Del Rey Oaks section) transects the site along its southwest property line. The USGS characterizes this segment of the Chupines fault as a "Late Quaternary active (rupture/deformation in the last 15,000 years) dextral-reverse slip fault with generally up-on-north vertical component of displacement." This segment of the Chupines fault zone is not considered a potential source for seismic shaking by the USGS, and has not been zoned as active by the CGS.

Table 3.1
Significant Faults

Fault Name	Fault Length (miles)	Closest Distance to Site* (miles)	Magnitude of Characteristic Earthquake**	Slip Rate (millimeters /year)
Monterey Bay-Tularcitos	51.6	1.6	7.3	0.5
Rinconada	118.7	7.3	7.5	1
San Gregorio Connected	109.4	9.7	7.5	5.5
Zayante-Vergales	36.0	19.8	7.0	0.1
San Andreas-SAS+SAP+SAN+SAO	293.3	24.3	8.05	17-24
Calaveras-CN+CC+CS	76.4	29.2	7.0	6-15
Hosgri	106.3	30.8	7.3	2.5

* Closest distance to the potential rupture.

** Moment magnitude: An estimate of an earthquake's magnitude based on the seismic moment (measure of an earthquake's size utilizing rock rigidity, amount of slip, and area of rupture).

According to Petersen et al. (2008), characterization of the San Andreas, San Gregorio and Calaveras faults are based on the following fault rupture segments and fault rupture scenarios:

- The San Gregorio Connected fault has been characterized by two segments and three rupture scenarios, plus a floating earthquake. The two segments are San Gregorio South (SGS) and San Gregorio North (SGN).

- The San Andreas Fault has been characterized by four segments and nine rupture scenarios, plus a floating earthquake. The four segments are Santa Cruz Mountains (SAS), Peninsula (SAP), North Coast (SAN), and Offshore (SAO).
- The Calaveras fault includes three segments and six rupture scenarios, plus a floating earthquake. The three segments are southern (CS), central (CC), and northern (CN).

3.4 SEISMIC DESIGN CRITERIA

The table below presents the recommended seismic design parameters for the proposed development.

Table 3.2 Recommended 2013 CBC (ASCE 7) Seismic Design Parameters

Design Parameter	Symbol	Recommended Value	2013 CBC (ASCE 7) Reference
Site Class	--	C	Table 20.3-1 (ASCE 7-10)
Mapped Spectral Acceleration for Short Periods	S_s	1.473 g	Section 1613.3.1 (1)
Mapped Spectral Acceleration for a 1-Second Period	S_1	0.535 g	Section 1613.3.1 (2)
Site Coefficient	F_a	1.0	Table 1613A.3.3 (1)
Site Coefficient	F_v	1.3	Table 1613A.3.3 (2)
MCE* Spectral Response Acceleration for Short Periods	S_{MS}	1.473 g	Equation 16A-37
MCE* Spectral Response Acceleration at 1-Second Period	S_{M1}	0.696 g	Equation 16A-38
Design Spectral Response Acceleration (5% damped) at Short Periods	S_{DS}	0.982 g	Equation 16A-39
Design Spectral Response Acceleration (5% damped) at 1-Second Period	S_{D1}	0.464 g	Equation 16A-40

*Maximum Considered Earthquake

3.5 LIQUEFACTION POTENTIAL AND DYNAMIC COMPACTION

Soil liquefaction is a condition where saturated, predominantly granular soils undergo a substantial loss of strength and potential deformation due to pore pressure increase resulting from cyclic stress application induced by earthquakes. In the process, the soil acquires a mobility sufficient to permit both horizontal and vertical movements if the soil mass is not confined. Soils most susceptible to liquefaction are saturated, loose, clean, uniformly graded, fine sand deposits.

Near surface coarse grained soils were typically medium dense to very dense overlying decomposed to highly weathered weak sandstone. No groundwater was encountered to a depth of 30 feet below existing grade at the time of our subsurface exploration, although perched groundwater could occur in unpaved or buried stormwater management systems areas for a brief time after significant rains. Therefore, the potential for liquefaction of the soils encountered in our borings is judged to be low.

Another type of seismically induced ground failure that can occur as a result of seismic shaking is dynamic compaction or seismic settlement. Such phenomena typically occur in unsaturated, loose granular material or uncompacted fill soils. In the event of a major earthquake in the site vicinity, we estimate that less than ¼ inches of total and differential settlement could occur as a result of dynamic compaction.

4. DISCUSSION AND CONCLUSIONS

Based upon the data collected during this investigation and the results of our engineering analysis, it is our opinion the site may be developed as proposed provided our recommendations are incorporated in the design and construction of the project. The opinions, conclusions and recommendations presented herein are based on our field and office studies, the properties of the soils encountered in our borings, and the results of our laboratory testing program. Geotechnical recommendations for site preparation, grading, engineered fill, surface drainage, utility trench backfill, foundations, and retaining walls are presented in the remaining portions of this report, along with the results of the percolation testing.

4.1 EARTHWORK

The improvements will include new additions and renovations to the existing site buildings, bus wash renovation, and enlargement of the existing fuel canopy, grading, pavements, retaining walls, on-site storm water management and new buried utilities. Earthwork is expected to be limited to that required for site clearing and leveling, cut and fill slopes, and excavations for footings and drilled piers, the installation of underground utilities, new fuel areas, and the buried stormwater management system.

Temporary construction slopes (if required) should be formed at no steeper than 1.5:1 (horizontal: vertical). Permanent cut slopes on the northeastern portion of the site should be cut to no steeper than 1.5:1 (horizontal: vertical). However, we understand that plans for the proposed improvement to the bus parking area include a 1:1 (H:V) cut slope above the lot, to a maximum slope height of about 10 feet. As described in report Section 2.2, cut slopes of 1:1 (H:V) are marginally stable and are expected to require continuous erosion control and periodic maintenance for surficial sloughing. Fill slopes should be constructed at 2:1 (Horizontal: Vertical) or flatter.

4.1.1 Site Clearing and Stripping

Prior to the start of construction, obstacles and deleterious material should be removed from the construction areas. Active utilities to be reused should be carefully located and protected during site clearing and construction.

All abandoned or underground utilities designated for removal, any concrete slabs on grade, foundations, and other obstacles and deleterious material encountered should be removed from the construction areas. Excavations from removal of foundations, underground utilities or other below ground obstructions should be cleaned of loose soil and deleterious material, and backfilled with compacted engineered fill compacted to the requirements given in the “Summary of Compaction Recommendations,” in Exhibit 1 of Appendix E. All clearing and backfill work should be performed under the observation of a representative from Kleinfelder.

Surface vegetation present at the time of construction should be stripped together with the organic-laden topsoil. Soils containing more than 3 percent of organic matter by weight or excessive visible organics as determined by a representative of Kleinfelder should be considered organic. The actual stripping depth should be determined at the time of construction. For planning purposes, the average stripping depth may be assumed to be approximately 3 inches in vegetated areas. Stripped material should be removed from the site or stockpiled for use in landscaping areas if approved by the project landscape architect.

4.1.2 Grading and Subgrade Preparation

Upon completion of site clearing and excavations, the exposed soil subgrades should be properly prepared prior to placement of fill or other construction activities. In areas to receive engineered fill or concrete slabs on grade, the upper 12 inches of soil should be scarified, moisture conditioned and compacted as recommended in the “Summary of Compaction Recommendations,” in Exhibit 1 of Appendix E. For the proposed buildings, the areas to be processed should include the entire building pads, extending no less than 5 feet beyond the limits of the buildings and any adjoining sidewalk areas unless obstructed by improvements to remain. In exterior walkways not adjacent to building areas, and in pavement areas, subgrade preparation should extend laterally no less than 2 feet beyond the back of curb, or edge of pavements, and sidewalks unless obstructed by improvements to remain. After the subgrades are properly prepared, the areas may be raised to design grades by placement of engineered fill.

All loose or wet subgrade soil encountered during construction should be stabilized prior to placement of new fill and further construction. The method of stabilization should be evaluated by a representative of Kleinfelder at the time of construction depending on the exposed conditions. Moisture conditioning of subgrade and fill soils will consist of adding water if the soils are too dry, and allowing the soils to dry if the soils are too wet.

4.1.3 Concrete Slabs-on-Grade

To reduce the effects of seasonal volume changes of the on-site expansive subgrade soils, we recommend that interior concrete slabs-on-grade be constructed on a layer of compacted “non-expansive” engineered fill at least 12 inches thick. Exterior concrete slabs-on-grade should be constructed on a layer of compacted “non-expansive” engineered fill at least 6 inches thick. Exterior concrete slabs-on-grade that will be subject to vehicle traffic should be designed as Portland cement concrete pavements as discussed in Section 4.4.2 “Rigid Concrete Pavements,” herein.

The non-expansive fill should extend laterally outward from the perimeter of the structure and adjoining perimeter walkways a minimum of 5 feet on every side unless obstructed by improvements to remain. In exterior walkways not adjacent to building areas, and in pavement areas, the “non-expansive fill” should extend laterally no less than 2 feet beyond the back of curb, or edge of pavements, and sidewalks unless obstructed by improvements to remain. The “non-expansive” fill should meet the requirements given in Section 4.1.4, “Material for Engineered Fill” and should be placed and compacted in accordance with Section 4.1.5, “Fill Placement and Compaction.”

4.1.4 Material for Engineered Fill

Inorganic on-site soils approved by a representative of Kleinfelder may be used as engineered fill, except in areas where “non-expansive” import fill is recommended.

Additionally, on-site soils should not be used as engineered fill in areas that will be subject to a combination of heavy loading, such as from footing foundations combined with saturated subsurface conditions. This condition is generally not expected to occur at the site, provided the buried stormwater management system is located under light weight vehicle parking areas and/or landscape areas. If heavily loaded and potentially saturated areas are later determined to exist, import fill would be required for backfill in those areas. In general, areas that are covered in pavements or concrete slabs-on-grade, have proper surface drainage, and that do not store concentrations of water for extended lengths of time, are not expected to become saturated. For example the bus wash area, if properly paved and drained of surface water, is not expected to become saturated.

Inorganic soils may be defined as soils containing less than 3 percent of organic matter by weight or free of visible organic matter deemed excessive by a representative of Kleinfelder. In general, material for use as engineered fill should be free of deleterious or oversized material, debris, or hazardous substances; should not contain rocks or lumps larger than 3 inches in greatest dimension; should not contain more than 15 percent of material larger than 1½ inches; and should contain sufficient fines (8% to 40% fines) to allow excavations to be made without caving.

Imported soils should be “non-expansive” and meet the above requirements, should be predominantly granular, and should have a plasticity index of 15 or less.

All proposed import fill must be approved by the project geotechnical engineer prior to delivery to the site. At least five (5) working days prior to importing to the site, a representative sample of the proposed import fill should be delivered to our laboratory for evaluation and possible testing.

4.1.5 Fill Placement and Compaction

Fill materials should be placed and compacted in horizontal lifts, each not exceeding 8 inches in uncompacted thickness. Compaction of fill should be performed by mechanical means only. Due to equipment limitations, thinner lifts may be necessary to achieve the recommended level of compaction. Relative compaction or compaction is defined as the in-place dry density of the compacted soil divided by the laboratory compacted maximum dry density as determined by ASTM Test Method D 1557 (latest edition), expressed as a percentage. A summary of our compaction recommendations is included in table format in Appendix E.

Permanent cut slopes should be constructed no steeper than 1.5:1 (horizontal: vertical). As previously discussed, we understand that plans include a 1:1 (H:V) cut slope above the bus parking lot, to a maximum slope height of about 10 feet. Cut slopes of 1:1 (H:V) are expected to require continuous erosion control and periodic maintenance for surficial sloughing.

Permanent fill slopes should be fully keyed and benched into the back slope. Fill slopes should be keyed at least 8 feet width and 2 feet deep below adjacent grade. The bottom of the keyway should be sloped at least 2 percent downwards back into the slope. Benches should be level or slope back into the back slope and should be no more than 3 feet in vertical separation and wide enough to allow compaction equipment. All fill slopes steeper than 3:1 (Horizontal:Vertical) should be compacted to at least 95 percent relative compaction full height. Based on the results of our slope stability analysis, compacted engineered fill for slopes shall have a minimum remolded cohesion of 100 pounds per square foot and a minimum remolded phi angle of 30 degrees. Fill slopes up to 10 feet in height may be constructed at 2:1 (Horizontal: Vertical) or flatter.

If taller slopes are planned, Kleinfelder should be consulted for additional recommendations. Some surficial slumping could occur; proper erosion control and timely maintenance will need to be implemented. Setback distances are essential and are generally based on slope height and the sensitivity of the structure (building or road). Kleinfelder should perform a detailed review if the proximity of the structure (from the improvement to the top of the slope) is less than the slope height.

Grading operations during the wet season or in areas where the soils are saturated may require provisions for drying prior to compaction. If the project necessitates fill placement and compaction in wet conditions, Kleinfelder can provide alternatives for drying the soil. Conversely, additional moisture may be required during the dry months. Water trucks should be available in sufficient number to provide adequate watering during subgrade preparation, fill placement and compaction.

4.1.6 Excavations and Utility Trench Backfill

Excavations for utility trenches, fuel islands, buried stormwater management systems, and foundations should be readily made with either a conventional backhoe or excavator. The walls of temporary trenches less than 5 feet in height, in the medium dense to very dense silty sand and clayey sand soils, should stand near vertical with minimal bracing, provided proper soil moisture content is maintained. Deeper trenches, or trenches into loose sands, must be properly shored and/or braced. Alternatively, temporary trenches may be constructed using sloping trench sidewalls. Sloping trench sidewalls should be constructed no steeper than 1:1 (horizontal: vertical). In addition, excavations should be located so that no structures are located above a plane projected

1.5 horizontal to 1 vertical upward from any point in an excavation, regardless of whether it is shored or unshored. Trench stability should be evaluated prior to occupation by construction personnel. All trenches should be constructed in accordance with OSHA and Cal-OSHA Safety Standards. Safety in and around utility trenches is the responsibility of the underground contractors.

Since groundwater is at least 30 feet below existing grade, we do not anticipate the need for dewatering of excavations. Wet weather can impact construction, especially where on-site soils have been compacted or where imported material is used. Refer to Section 4.1.9 on “Wet Weather Construction.” If different soil or groundwater conditions are encountered during construction than those encountered during our subsurface exploration, Kleinfelder should be contacted to provide additional recommendations.

Utility trench pipe zone backfill, extending from the bottom of the trench to at least 1 foot above the top of pipe, should consist of free-draining sand unless lean concrete is specified. Above the pipe zone, underground utility trenches should be backfilled with compacted engineered fill. Either approved on-site soil or imported sand may be used for backfilling utility trenches. Trench backfill should be capped with at least 12 inches of compacted, on-site soil similar to that of the adjoining subgrade. A summary of our compaction recommendations is included in Appendix E. Compaction should be performed by mechanical means only. Water-jetting or flooding to attain compaction of backfill should not be permitted.

4.1.7 Surface Drainage

Final site grading should provide surface drainage away from existing and new buildings, concrete slabs-on-grade and pavements to reduce the percolation of water into the underlying soils. Ponding of surface water should not be allowed adjacent to structures and on exterior flatwork and pavements. The ground surface should be sloped away from the buildings a minimum of 4 percent in landscaped areas and 2 percent in paved areas. Rainwater collected on the roofs of buildings should be transported through gutters, downspouts and closed pipes, which discharge on pavements or lead directly to the site storm sewer system. If discharging onto the pavement, safety of pedestrian traffic should be considered.

On-site soils are highly erodible and should be planted with erosion resistant vegetation as soon as practicable. The erosion control vegetation should be planted early enough before the winter rainy season to allow the vegetation to take root before the rainy season. Ground cover will require periodic maintenance and a maintenance schedule should be developed. Specific details regarding erosion control should be determined by the Civil Engineer.

4.1.8 Seepage Control

We do not anticipate any significant seepage problems due to the porous sandy nature of the on-site near-surface soils, provided water is not allowed to pond near proposed pavements, foundations and slabs-on-grade. Features that do not impound water for extended periods of time, such as drainage swales or where water sheets over embankments do not require setbacks from road areas provided they drain away from the pavements and do not trap water adjacent to the pavements. However, water erosion of such features will need to be addressed.

We also understand that storm water retention/infiltration features are planned for the project. These may consist of surface features and buried improvements. These features will act as impoundment areas for on-site storm water, and water will concentrate in these areas for extended time periods.

4.1.8.1 Surface Infiltration Features

We recommend that surface infiltration features be located to minimize the impact of the storm water on the nearby pavements and foundations. To minimize impact on pavements, we recommend infiltration features be located with the pavement soil subgrade elevation (i.e. bottom of the aggregate base layer, or the bottom of the asphalt concrete layer where aggregate base is not used) located at least 2 feet above a plane projected 5:1 (horizontal: vertical) downward from the highest retained water elevation (design water surface) of the adjacent impoundment area. Assuming level ground we anticipate that this would result in off sets of about 10 to 15 feet from the pavements, or closer where the bottoms of the features are excavated or lowered below the adjacent grade. To minimize impact on foundations, we recommend infiltration features be located so that the bottoms of the footings are at least 1 footing width or at least 2 feet, whichever is greater length, above a plane projected 5:1 (horizontal:vertical) downward from the highest retained water elevation (design water surface) of the adjacent impoundment area.

In areas of the site where surface infiltration features must be located adjacent to areas of extended flat pavements, such as in parking areas with bioswales, consideration should be given to deepening curbs at the edges of the infiltration features to the bottom of the drain rock layer. This will reduce the potential for rotation of the curb at the edge of the feature due to infiltrating water softening the supporting soils.

4.1.8.2 Buried Storm Water Management Systems

Buried storm water management systems should be located in areas that will not be subjected to high ground pressures that exist in areas such as foundation zones and bus parking or traffic areas. Such improvements will need to be located in light weight vehicle parking areas and landscape areas. Additionally, animal burrows in the existing slopes could result in piping failures downslope of the buried storm water management system. This will need to be mitigated by grading or site selection for the system. The lower permeability of deeper on-site soils containing clayey fines and/or decomposed sandstone will limit the total embedment depth of the buried storm water management system. Our percolation test results are presented in Section 4.5, “Percolation Characteristics of In-Situ Soils.” Bottoms of buried storm water management systems should be located no closer than 30 feet horizontally from structures or the free face of a slope and above a 2:1 (Horizontal:Vertical) slope projected downwards from the bottoms of adjacent footings.

4.1.9 Wet Weather Construction

If site grading and construction is to be performed during the winter rainy months, the owner and contractors should be fully aware of the potential impact of wet weather. Rainstorms can cause delay to construction and damage to previously completed work, such as saturating a compacted subgrade, or flooding an excavation. Runoff can also cause erosion.

Earthwork during rainy months will require extra effort and caution by the contractors. The soil may be too wet to compact which will require processing to dry the soil. The grading contractor should be responsible to protect his work to avoid damage by rainstorms, including smooth rolling to seal off a pad or subgrade surface to facilitate drainage and to reduce rain damage, and covering the trenches with plastic sheeting. Ponding water should be pumped out immediately. Construction in wet weather should be addressed in the project construction bid documents and/or specifications. We recommend the grading contractor submit a wet weather construction plan outlining procedures they will employ to protect their work and to minimize damage to their work by rainstorms.

4.1.10 Construction Observation

Variations in soil types and conditions are possible and may be encountered during construction. In order to permit correlation between the soil data obtained during this investigation and the actual soil conditions encountered during construction, we recommend that Kleinfelder be retained to provide observation and testing services during site earthwork and foundation construction. This will allow us the opportunity to compare actual conditions exposed during construction with those encountered in our investigation and to expedite supplemental recommendations if warranted by the exposed conditions. All earthwork should be performed in accordance with the recommendations presented in this report, or as recommended by Kleinfelder during construction. Kleinfelder should be notified at least 2 working days before the start of construction, and before the time when observation and testing services are needed.

We also recommend that Kleinfelder be retained to review your pre-final foundation and grading plans and specifications. It has been our experience that this review provides an opportunity to detect misinterpretation or misunderstandings before the start of construction.

4.2 BUILDING FOUNDATIONS AND SETTLEMENT

4.2.1 Shallow Footing Foundations

Based on our investigation, the loads for the proposed structures can be supported by spread footings bearing on onsite soils. The foundation elements should be embedded at least 18 inches below pad grade or lowest adjacent finished grade whichever provides a deeper embedment. The recommended allowable soil bearing pressures, depth of embedment, and width of footings are presented below.

Table 4.1 Foundation Bearing Capacity Recommendations

Footing Type	Allowable Bearing Pressure (psf)*	Minimum Embedment (in)**	Minimum Width (in)
Continuous Footing	2,750	18	18
Isolated Footing	3,000	18	24

* Pounds per square foot, dead plus live load. Includes a factor of safety (FS) of 3.

** Below lowest adjacent grade defined as bottom of slab on the interior and finish grade at the exterior.

Allowable soil bearing pressures may be increased by one-third for transient loads such as wind and seismic loads.

Lateral loads may be resisted by a combination of friction between the foundation bottoms and the supporting subgrade, and by passive resistance acting against the vertical faces of the foundations, including grade beams. An ultimate friction coefficient of 0.35 between the foundation and supporting subgrade may be used. For passive resistance, an ultimate equivalent fluid pressure of 350 pounds per cubic foot may be used. Where ground slopes downwards away from the footings, such as may occur at the proposed retaining wall, passive pressure should be neglected on the upper portions of the footing within 5 feet horizontally of the slope face. (For example for a 5:1 horizontal: vertical slope downwards from the footing face, neglect the upper buried 1 foot of the footing for passive pressure resistance. For a 2:1 H:V slope, neglect the upper buried 2½ feet of the footing.) Passive pressure should also be neglected in the upper one foot unless the adjacent surface is confined by paving or flatwork. The friction coefficient and passive resistance may be used concurrently.

Total estimated settlement of an individual spread foundation will vary depending on the plan dimensions of the foundation and the actual load supported. Based on anticipated foundation dimensions and loads, we estimate maximum settlement of foundations designed and constructed in accordance with the preceding recommendations to be on the order of 1 inch. Differential settlement between similarly loaded, adjacent footings is expected to be less than ½ inch provided footings are founded on similar materials. Settlement of all foundations is expected to occur rapidly and should be essentially complete shortly after initial application of the loads. In the event of a major earthquake in the site vicinity, we estimate that the additional total and differential ground settlement as a result of dynamic compaction would be less than ¼ inch.

Where footings are located adjacent to below-grade structures or near major underground utilities, the footings should extend below a 1.5:1 (horizontal to vertical) plane projected upward from the structure footing (or bottom if no footings) or the bottom of the underground utility to avoid surcharging the below grade structure and underground utility with building loads. Also, where utilities cross the perimeter footings line, the trench backfill should consist of a vertical barrier of impervious type of material or lean concrete. In addition, where utilities cross through or under exterior footings, flexible waterproof caulking should be provided between the sleeve and the pipe. Utility plans should be reviewed by Kleinfelder prior to trenching for conformance to these requirements.

Concrete for footings should be placed neat against native soil or engineered fill. It is critical that footing excavations not be allowed to dry before placing concrete. If shrinkage cracks appear in the footing excavations, the excavations should be thoroughly moistened to close all cracks prior to concrete placement. The footing excavations should be monitored by a representative of Kleinfelder for compliance with appropriate moisture control and to confirm the adequacy of the bearing materials. If soft or loose materials are encountered at the bottom of the footing excavations, they should be removed and replaced with lean concrete or engineered fill. Kleinfelder should also be present during the excavation. If desired, unit prices for such excavation and backfilling should be obtained during contractor bidding for this project.

4.2.2 Cast-In Drilled-Hole Pile Foundations (Drilled Piers)

The proposed retaining wall and fuel canopy structures may be supported on a cast-in-drilled-hole (CIDH) pile (i.e. drilled pier) foundation system designed to derive support from end bearing at the bottom of the pier (due to the low skin friction of the on-site materials), and lateral resistance from passive soil pressure against the side of the pier. The recommended allowable foundation loads presented in this section include a factor of safety (FS) of 3.

Drilled pier embedment length may be controlled by various load cases in either axial loading (compression or uplift) or lateral loading; however, drilled piers should be at least 10 feet in depth and at least 18 inches in diameter. The drilled piers should be located no closer together than three pier diameters on-center.

The allowable end bearing capacity for drilled piers should be taken as 8,000 psf in the native soil materials. The weight of the buried portion of the pier may be neglected when calculating the downward axial loads on the pier. A one-third increase in the allowable capacity may be used for consideration of transient loads such as wind or seismic.

For resistance to uplift of the foundation an ultimate skin friction of 3500 pounds total per pier, plus the weight of the pier may be used. The skin friction assumes an 18 inch diameter 10 foot deep pier. The skin friction may be scaled up proportionally to the surface area of the pier due to increased width or depth.

For drilled shafts designed and constructed in accordance with the recommendations presented in this report, total settlement of each drilled shaft is expected to be less than about 1 inch, with differential settlement between adjacent supports of up to about 1/2 inch. The majority of the settlement should occur during and shortly after application of the structure loads.

Pier foundation resistance to lateral loads will be provided by passive resistance of the soil against shafts, pier caps, and grade beams (if present) and by the bending stiffness of the pier shafts. The lateral resistance of a drilled pier is a function of the surrounding soil strength and stiffness, size and stiffness of the pier, pier top connection, and induced moments and forces at the top of the pier. For pier caps and grade beams, the ultimate passive pressure available in undisturbed native soil or compacted engineered fill may be taken as equivalent to the pressure exerted by a fluid weighing 350 pounds per cubic foot (pcf) acting on two pier diameter for the portion of the pier foundation embedded in firm soil. Where ground slopes downwards away from the pier foundation, such as may occur at the proposed retaining wall, passive pressure should be neglected on the upper portions of the pier (and grade beam) within 5 feet horizontally of the slope face. (For example for a 5:1 horizontal: vertical slope downwards from the downslope edge of the pier, neglect the upper buried 1 foot of the pier for passive pressure resistance. For a 2:1 H:V slope, neglect the upper buried 2½ feet of the pier.) This passive pressure value is an allowable value derived using an estimated shaft head deflection of about ½ inch. We anticipate that there may be a variety of pier lateral loading conditions due to the configuration of the proposed structure. The appropriate factor of safety for lateral load resistance will depend on the design condition and should be selected by the designer.

The structural engineer should determine the actual embedded depth based on the lateral loads transmitted to the foundations. Once the structural loading information is available, if requested, Kleinfelder can assist in determining the shear, moments and lateral displacement for the piers based on the design loads.

We note that attention must be given to the method of drilled pier construction to satisfy the above recommendations. The need for slurry is not anticipated due to groundwater being at least 30 feet below existing grade, and to allow inspection of the bottom of the shaft. Sandy soils were encountered in our borings; therefore, casing should be available onsite to facilitate supporting the excavations if needed. Highly weathered to decomposed sandstone material was encountered in some of our borings; therefore, rock augers or rock drilling buckets will likely be required. Steel reinforcement and concrete should be placed within about 4 to 6 hours of completion of each drilled hole. As a minimum, the holes should be poured the same day they are drilled. The bottom of the drilled holes should be cleaned to remove as much loose soil as practical prior to placement of concrete. A representative from Kleinfelder should be present to observe drilled holes to confirm the soils encountered are capable of carrying the design loads and that bottom conditions are satisfactory prior to placing steel reinforcement.

The steel reinforcement should be centered in the drilled hole. Concrete should be discharged vertically with a tremie pipe from the shaft bottom upward at a rate in which the tremie nozzle does not become separated from the placed concrete by more than three feet. Under no circumstances should concrete be allowed to free-fall against either the steel reinforcement or the sides of the excavation during construction. Sufficient vibration should be performed while the concrete is tremied to minimize voids and properly derive the frictional shaft surface to satisfy uplift design requirements.

Prior to mobilizing drilling equipment to the site, the foundation contractor should submit to Kleinfelder a construction plan describing the procedures it intends to utilize in the CIDH pile (drilled pier) construction process. Kleinfelder should review this plan and confirm that the procedures conform to the recommendations provided herein.

4.3 RETAINING STRUCTURES

A new retaining wall is planned on the western portion of the site above Canyon Del Rey Boulevard. We understand that this wall will be approximately 10 feet in height, and will primarily retain engineered fill and pavements areas.

Retaining walls may be supported on deepened continuous conventional footings or CIDH piles (drilled piers) designed in accordance with our recommendations presented above. Flexible walls that are free to deflect at the top may be designed for an active lateral earth pressure calculated using an equivalent fluid weight of 40 pounds per cubic foot where the backfill is level if the retaining wall is backfilled with on-site soil or approved import fill material. Rigid walls that are constrained against movement at the top should be designed for an "at-rest" lateral earth pressure calculated using an equivalent fluid weight of 60 pounds per cubic foot where the backfill is level if the retaining wall is backfilled with on-site soil or approved import fill material. The above pressure values apply to horizontal backfill and do not include hydrostatic pressures that might be caused by groundwater or water trapped behind the structure.

For seismic lateral surcharge loads, a design peak ground acceleration (PGA) of 0.57g was used in the analysis which resulted in an additional seismic pressure of 15H pounds per square foot (where H is the total height of the wall in feet) for flexible walls that are free to defect at the top, and an additional seismic pressure of 33H pounds per square foot for rigid walls that are constrained against movement at the top. This additional seismic pressure should be applied as a rectangular distribution over the entire depth of the wall.

In addition to lateral earth pressures and seismic surcharges, retaining walls must be designed to resist horizontal pressures that may be generated by surcharge loads applied at the ground surface such as from uniform loads or vehicle loads. For uniform loads, such as floor live loads, an additional uniform lateral surcharge equal to 50 percent of the vertical live loads should be applied on the wall. For occasional fork-lift or light vehicle loads, we recommend adding an additional uniform lateral surcharge pressure of 50 pounds per square foot. Heavy vehicle loads, such as from busses or heavy trucks is best evaluated once the position, type, magnitude and frequency of the loads, is determined. Kleinfelder should be consulted to provide specific recommendations for heavy vehicle loads once this information is available. For initial planning purposes, an estimated additional uniform lateral surcharge pressure of 250 pounds per square foot is recommended. For other loads, such as point or line loads, the additional lateral surcharge will depend on the position, type and magnitude of the loads, and Kleinfelder should be consulted to provide specific recommendations.

Retaining walls higher than 2 feet should be fully drained. Drainage may be provided by a prefabricated drainage system, such as Mirafi Miradrain 6000/6200, or a 1 to 2 foot wide zone of 3/4-inch by No. 4 crushed, clean rock wrapped in a layer of non-woven geotextile filter fabric such as Mirafi 140NC or equivalent. Class 2 Permeable material (Caltrans Standard Specifications, Section 68) may be used in lieu of the clean crushed rock and filter fabric. The gravel drain should extend from the base of the wall to within about one foot of the top of the wall. The upper one foot of the backfill should consist of compacted native soil graded to direct surface water away from the walls. A 4-inch diameter, rigid perforated pipe surrounded by the gravel drainage blanket should be installed at the base of the wall to collect and transport the water away from the wall toward a suitable discharge point. The pipe should be sloped to drain by gravity to appropriate outlets. The pipe should be placed on approximately 4 inches of gravel bedding with perforations placed down. The pipe should consist of solid walled pipe where located away from the base of the wall. Similarly, a collector pipe will be required where drainage panels are installed.

Where migration of moisture through the retaining wall would be detrimental or undesirable, the retaining wall should be waterproofed as specified by the project architect.

Backfill against wall structures should be properly compacted. Over-compaction should be avoided because increased compaction can result in lateral pressures significantly higher than those recommended above. Wall backfill should be spread in level lifts not exceeding 6 inches in thickness. Each lift should be compacted to not less than 90 percent relative compaction, per ASTM D1557 latest edition, at over the optimum moisture content. Retaining walls may be subjected to higher stress during placement of wall backfill where large or heavy grading equipment is used. This should be considered by the wall designer and contractor, and bracing during construction may be required. Compaction of wall backfill within 5 feet of the wall should be performed by hand-operated equipment.

We recommend that design drawings of retaining walls showing height of wall, backfill material type, drainage details and the earth pressures used in design be reviewed by Kleinfelder for conformance to the recommendations given. Certain proprietary wall systems, such as reinforced earth walls, segmental block walls, and criblock walls, are design-built systems requiring close coordination with the Civil Engineer on drainage outlets and connections. If any proprietary walls are planned, we strongly recommend that we review the type of wall proposed and make alternate appropriate lateral earth pressure recommendations for these walls. Furthermore, we recommend that Kleinfelder be retained to review design plans prior to issuance for construction.

4.4 VEHICLE PAVEMENTS

Pavements for this project are anticipated to consist of parking and access areas for passenger cars and light pickup trucks, and heavy traffic areas for busses and garbage trucks. Traffic loading information for this project is based on our experience with similar projects. Based on our experience, we suggest using a Traffic Index (TI) of at least 4.5 for automobile parking areas, a TI of at least 5.5 for automobile and light truck traffic lanes, and a TI of at least 6.5 for garbage truck areas. We estimate that a Traffic Index of 7.0 to 8.0 will be used for areas servicing busses. For heavy vehicle areas, a minimum asphalt concrete section of 4½ inches is recommended. The anticipated traffic and the alternate pavement sections presented should be reviewed by the project civil engineer in consultation with the owner during the development of the final grading and paving plans.

4.4.1 Flexible Asphalt Pavements

Bulk samples of the near surface soil were obtained from the site during our field investigation. The results of our laboratory testing indicate R-Values of 7 and 57. The R-value of 7 material was encountered in the bus parking area on the northern portion of the site, on the flatter upper portion of the lot. The R-Value of 57 material was encountered in the automobile parking lot on the south side of the site, near the south slope. The recommended pavement sections are presented in the tables below. We have made our pavement designs based on the pavement subgrade soil consisting of existing on-site surface material (i.e. clayey sand and silty sand with gravel). If site grading exposes soil other than that utilized in our analysis, we should perform additional tests to confirm or revise the recommended pavement sections to reflect the actual field conditions.

Asphalt concrete should meet the requirements for 1/2- or 3/4-inch maximum, medium Type A or Type B asphalt concrete as specified in Section 39 of the Caltrans Standard Specifications. Class 2 aggregate base materials should conform to the requirements presented in Section 26 of the Caltrans Standard Specifications. Class 2 aggregate subbase materials should conform to the requirements presented in Section 25 of the Caltrans Standard Specifications, with a minimum R-Value of 50. Asphalt concrete and aggregate base, and preparation of the subgrade should conform to, and be placed in accordance with, the California Department of Transportation Standard Specifications, except as noted herein. ASTM Test procedures should be used to assess the percent relative compaction of soils, aggregate base and asphalt concrete. Asphalt concrete should be compacted to between 95 percent and 96 percent of the maximum compacted unit weight.

Table 4.2 Flexible Asphalt Concrete Pavement Alternatives R-Value = 7

FLEXIBLE PAVEMENT SECTION ALTERNATIVES R-Value = 7 (All areas except the existing southern parking lot containing Boring B-6)			
Traffic Index	**Asphalt Concrete (inches)**	**Class 2 Aggregate Base (inches)**	**Class 2 Aggregate Subbase (inches)**
4.5	2.5	9.0	-
	2.5	4.5	5.0
5.0	2.5	10.5	-
	2.5	5.0	6.0
5.5	3.0	11.5	-
	3.0	5.5	6.5
6.0	3.0	13.0	-
	3.0	6.5	7.5
6.5	3.5	14.5	-
	3.5	6.5	8.5
7.0	4.0	15.0	-
	4.0	6.5	9.5
7.5	4.5	16.0	-
	4.5	7.0	10.0
8.0	4.5	17.5	-
	4.5	8.0	10.5

*Note: AC = Type A or B Asphalt Concrete
AB = Class 2 Aggregate Base (Minimum R-Value = 78)
ASB = Class 2 Aggregate Subbase Minimum R-Value = 50

Table 4.3 Flexible Asphalt Concrete Pavement Alternatives R-Value = 57

FLEXIBLE PAVEMENT SECTION ALTERNATIVES R-Value = 57 (Southern parking lot containing Boring B-6)		
Traffic Index	**Asphalt Concrete (inches)**	**Class 2 Aggregate Base (inches)**
4.5	4.0	-
	2.5	4.0
5.0	4.5	-
	2.5	4.0
5.5	5.0	-
	3.0	4.0
6.0	5.5	-
	3.0	4.0
6.5	6.0	-
	3.5	4.0
7.0	6.5	-
	4.0	4.0
7.5	7.5	-
	4.5	4.0
8.0	8.0	-
	4.5	4.0

*Note: AC = Type A or B Asphalt Concrete
AB = Class 2 Aggregate Base (Minimum R-Value = 78)

Parking areas should be sloped at a minimum of 2 percent and drainage gradients maintained to carry all surface water off the site. Surface water ponding should not be allowed anywhere on the site during or after construction. Seepage cut-offs should be constructed as discussed previously in Section 4.1.

4.4.2 Rigid Concrete Pavements

Rigid pavements consisting of Portland cement concrete may also be considered. We recommend that the pavement sections presented be reviewed by the project Civil Engineer in consultation with AECOM and Monterey Salinas Transit District during the development of the final grading plans.

Portland cement concrete pavements should be constructed on a minimum 12-inch thick layer of Class 2 Aggregate Base over the subgrade. Preparation of soil subgrade and compaction of the aggregate base should follow the recommendations given above in Section 4.1. The compacted subgrade and the aggregate base should be non-yielding.

Using the above design parameters and the Portland Cement Association Simplified Design Procedure, we recommend the use of a minimum concrete pavement thickness of 4.5 inches for light vehicle areas and 6.5 inches in areas that will experience heavy truck or bus traffic. Our design is based on a combined modulus of subgrade reaction of approximately 130 pci at the top of the aggregate base, a concrete shoulder or curb without doweled joints, and a modulus of rupture for the concrete of 600 pounds per square inch. If a concrete shoulder or curb will not be used, then the above minimum concrete pavement thicknesses should be increased by on inch. It should be noted that the modulus of rupture for concrete is based on flexural strength, not compressive strength, and should be specified accordingly. Our experience is that the compressive strength will be on the order of 4,500 to 5,000 psi to achieve the required flexural strength. Concrete with a compressive strength of 3,000 psi is not expected to provide the desired flexural strength. Laboratory testing to evaluate the design strength is recommended.

4.5 PERCOLATION CHARACTERISTICS OF IN-SITU SOILS

Presaturation of the percolation test holes was completed on November 20, 2014, 24 hours prior to percolation testing. Presaturation consisted of filling the prepared holes with water.

Percolation testing was performed on the morning and afternoon of November 21, 2014. For the percolation holes we generally used a 10-minute measurement interval. The location of our exploratory borings and percolation test holes are presented on the Site Plan, Plate 2. The results of the percolation testing are presented in Appendix B and summarized in the table below. The raw stabilized rates are presented at the bottom of the tables.

Generally, percolation rates of the upper silty sand soils encountered above approximately 9 to 13 feet bgs were measured to be about 3 to 6 minutes per inch raw stabilized rate (i.e. percolation test holes P-1 to P-3). The percolation rates of the deeper clayey sand soils encountered in P-1 to P-3 were too "slow" to accurately measure over the given testing period. It should be noted that the rates presented are raw stabilized rates. We have not applied any corrections for the hole diameter, pea gravel, or slotted pipe.

Table 4.4 Percolation Test Results

Location	Total Hole Depth (feet)	Soil Type	Percolation Rate (min / inch)	Slow Percolation Below (feet)
P-1	13.4	SM/SC	5.6	10.4
P-2	13.5	SM/SC	2.7	13.1
P-3	13.2	SM/SC	3.6	8.9

Our scope-of-work was limited to testing, and excludes evaluation of the general suitability of the sites for the infiltration system, evaluation of the storage capacity and permeability of the in-situ soils, nor actual design of the infiltration system. The proposed storm water management system design should be performed by the project civil engineer.

5. LIMITATIONS

This work was performed in a manner consistent with that level of care and skill ordinarily exercised by other members of Kleinfelder's profession practicing in the same locality, under similar conditions and at the date the services are provided. Our conclusions, opinions and recommendations are based on a limited number of observations and data. It is possible that conditions could vary between or beyond the data evaluated. Kleinfelder makes no other representation, guarantee or warranty, express or implied, regarding the services, communication (oral or written), report, opinion, or instrument of service provided.

This report may be used only by AECOM and Monterey Salinas Transit District and the registered design professional in responsible charge and only for the purposes stated for this specific engagement within a reasonable time from its issuance, but in no event later than two (2) years from the date of the report.

The work performed was based on project information provided by AECOM and Monterey Salinas Transit District. As part of our scope of services, Kleinfelder will be performing a review of the pre-final plans and specifications for this project. Please forward the pre-final plans and specifications to us when they are completed for our review. If AECOM and Monterey Salinas Transit District does not retain Kleinfelder to review any plans and specifications, including any revisions or modifications to the plans and specifications, Kleinfelder assumes no responsibility for the suitability of our recommendations. In addition, if there are any changes in the field to the plans and specifications, AECOM and Monterey Salinas Transit District must obtain written approval from Kleinfelder's engineer that such changes do not affect our recommendations. Failure to do so will vitiate Kleinfelder's recommendations.

The scope of services was limited to nine borings, percolation testing, laboratory testing of selected soil samples, engineering analysis, and preparation of this report. It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions present due to the limitations of data from field studies. The conclusions of this assessment are based on nine borings to a maximum depth of about 30 feet below the existing ground surface, previous data collected by Kleinfelder on this site, laboratory testing of natural moisture content and density, grain size distribution, plasticity, unconfined compression testing and percolation testing of the site soils, and engineering analyses.

Kleinfelder offers various levels of investigative and engineering services to suit the varying needs of different clients. Although risk can never be eliminated, more detailed and extensive studies yield more information, which may help understand and manage the level of risk. Since detailed study and analysis involves greater expense, our clients participate in determining levels of service, which provide information for their purposes at acceptable levels of risk. The client and key members of the design team should discuss the issues covered in this report with Kleinfelder, so that the issues are understood and applied in a manner consistent with the owner's budget, tolerance of risk and expectations for future performance and maintenance.

Recommendations contained in this report are based on our field observations and subsurface explorations, limited laboratory tests, and our present knowledge of the proposed construction. It is possible that soil, rock or groundwater conditions could vary between or beyond the points explored. If soil, rock or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that Kleinfelder is notified immediately so that we may reevaluate the recommendations of this report. If the scope of the proposed construction, including the estimated structural loads, and the design depths or locations of the foundations, changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid unless the changes are reviewed, and the conclusions of this report are modified or approved in writing, by Kleinfelder.

As the geotechnical engineering firm that performed the geotechnical evaluation for this project, Kleinfelder should be retained to confirm that the recommendations of this report are properly incorporated in the design of this project, and properly implemented during construction. This may avoid misinterpretation of the information by other parties and will allow us to review and modify our recommendations if variations in the soil conditions are encountered.

As a minimum Kleinfelder should be retained to provide the following continuing services for the project:

- Review the project pre-final plans and specifications, including any revisions or modifications;
- Observe and evaluate the site earthwork operations to confirm subgrade soils are suitable for construction and placement of engineered fill;
- Observe and evaluate cut slope and fill slope construction;
- Observe excavations and confirm that engineered fill and backfill for utilities and other buried improvements are placed and compacted per the project specifications;
- Observe foundation excavations to confirm subsurface conditions are as anticipated and to verify adequate geotechnical support for the proposed improvements;
- Observe retaining wall construction and backfill; and
- Observe asphalt concrete and Portland cement concrete pavement construction.

The scope of services for this subsurface exploration and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site.

Kleinfelder cannot be responsible for interpretation by others of this report or the conditions encountered in the field. Kleinfelder must be retained so that all geotechnical aspects of construction will be monitored on a full-time basis by a representative from Kleinfelder, including site preparation, preparation of foundations, and placement of engineered fill and trench backfill. These services provide Kleinfelder the opportunity to observe the actual soil, rock and groundwater conditions encountered during construction and to evaluate the applicability of the recommendations presented in this report to the site conditions. If Kleinfelder is not retained to provide these services, we will cease to be the geotechnical engineer of record for this project and will assume no responsibility for any potential claim during or after construction on this project. If changed site conditions affect the recommendations presented herein, Kleinfelder must also be retained to perform a supplemental evaluation and to issue a revision to our original report.

This report, and any future addenda or reports regarding this site, may be made available to bidders to supply them with only the data contained in the report regarding subsurface conditions and laboratory test results at the point and time noted. Bidders may not rely on interpretations, opinion, recommendations, or conclusions contained in the report. Because of the limited nature of any subsurface study, the contractor may encounter conditions during construction which differ from those presented in this report. In such event, the contractor should promptly notify the owner so that Kleinfelder's geotechnical engineer can be contacted to confirm those conditions. We recommend the contractor describe the nature and extent of the differing conditions in writing and that the construction contract include provisions for dealing with differing conditions. Contingency funds should be reserved for potential problems during earthwork and foundation construction. Furthermore, the contractor should be prepared to handle contamination conditions if encountered at this site, which may affect the excavation, removal, or disposal of soil; dewatering of excavations; and health and safety of workers.

6. REFERENCES

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Dibblee, T.W. and Minch, J.A., 2007, Geologic map of the Monterey and Seaside quadrangles, Monterey County, California: Dibblee Geological Foundation, Dibblee Foundation Map DF-346, scale 1:24,000.

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Donald Tharp and Associates (1993), Soil and Foundation Investigation, Repair of Small Slope Failure, Ryan Ranch Road and Highway 218, Monterey, California.

Weber Hayes and Associates (1993), Preliminary Geologic Letter Report for the October 1993 Highway 218 Slope Failure, Monterey Salinas Transit, Monterey County, California

PLATES

PLOTTED: 12/8/2014 12:02 PM BY: jeff sala

0 2000 4000

SCALE: 1" = 2000' SCALE IN FEET

REFERENCE:
Google Earth Pro, Imagery date 8/25/2013

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PROJECT NO. 20153715	SITE VICINITY MAP		PLATE
DRAWN BY: JDS			
CHECKED BY: BH	MONTEREY - SALINAS TRANSIT DISTRICT OPERATION & MAINTENANCE FACILITY 1 RYAN RANCH ROAD MONTEREY, CALIFORNIA		1
DATE: 12-08-2014			
REVISED:			

CAD FILE: L:\2014\14Projects\20153715-MST\GRAPHICS\SITE.dwg

0 100 200

SCALE: 1" = 100' SCALE IN FEET

REFERENCE:
Google Earth Pro, Imagery date 8/25/2013

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LEGEND

PROJECT AREA

B-9 SOIL BORING
(By Kleinfelder, 11/2014)

P-3 PERCOLATION TEST
(By Kleinfelder, 11/2014)

NOTE: ALL LOCATIONS ARE APPROXIMATE

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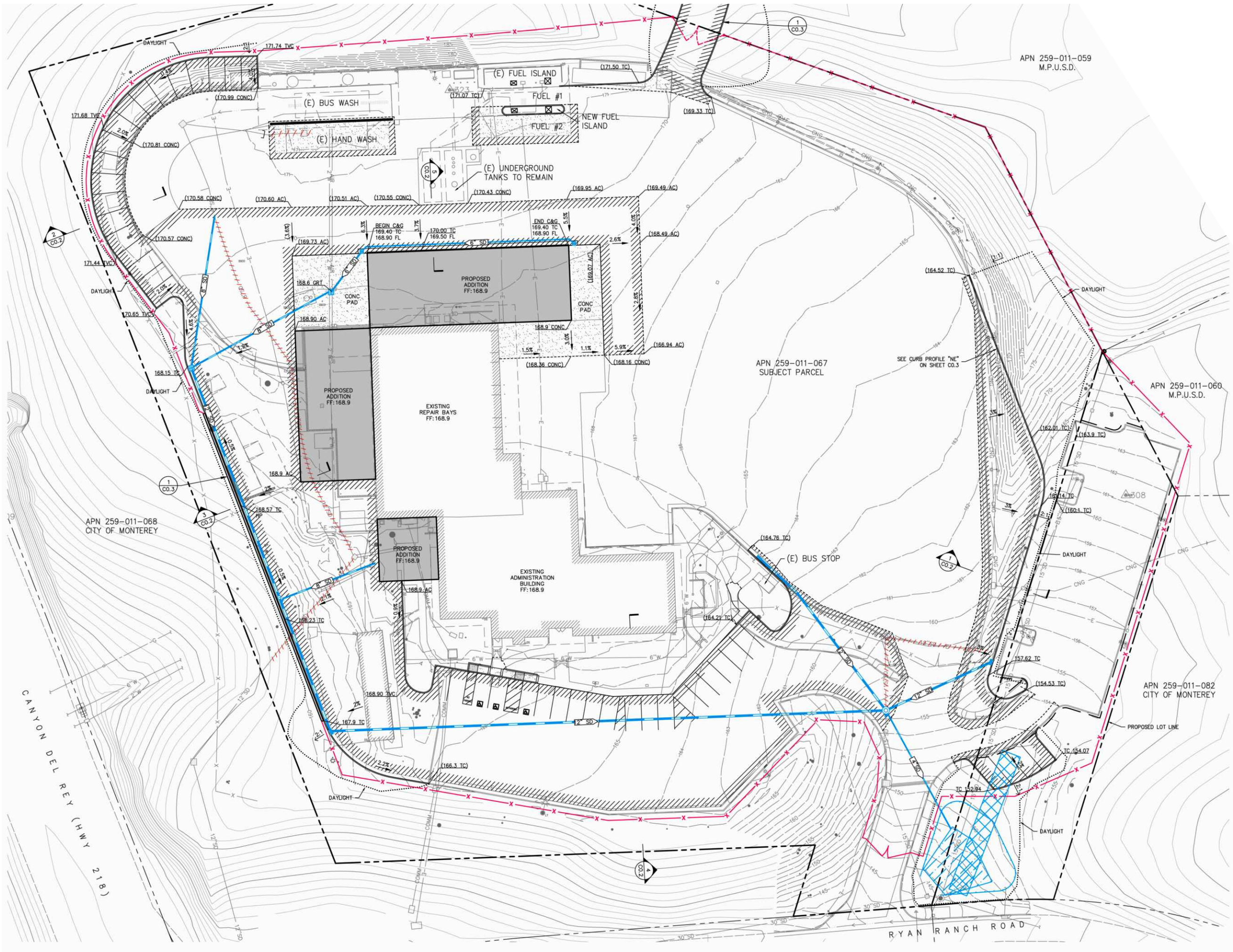
PROJECT NO. 20153715		SITE PLAN	PLATE
DRAWN BY:	JDS		2
CHECKED BY:	BH	MONTEREY - SALINAS TRANSIT DISTRICT OPERATION & MAINTENANCE FACILITY 1 RYAN RANCH ROAD MONTEREY, CALIFORNIA	
DATE:	12-08-2014		
REVISED:			

PLOTTED: 12/8/2014 12:02 PM BY: jeff sala

CAD FILE: L:\2014\14Projects\20153715-MST\GRAPHICS\SITE.dwg

PLOTTED: 1/12/2015 9:15 AM BY: jeff sala



0 100 200

SCALE: 1" = 100' SCALE IN FEET

REFERENCE:
AECOM, Grading and Drainage Plan C1.1, Monterey-Salinas Transit Operations & Maintenance Facility Expansion & Remodel, dated 11/2014

GRADING & DRAINAGE LEGEND

- AC PAVING
- CONCRETE PAVING
- CURB AND GUTTER
- BOUNDARY LINE
- EASEMENT LINE
- 100 EXISTING GRADE CONTOUR
- FLOW LINE
- SC SAWCUT LINE
- DL DAYLIGHT LINE
- FENCE
- FACE OF EXISTING BUILDING/STRUCTURE
- RETAINING WALL
- 12"SD PROPOSED STORM DRAIN
- PROPOSED STORM DRAIN INLET
- 51.54 TC FINISHED SPOT GRADE
- (51.54 TC) EXISTING SPOT GRADE; MATCH, U.O.N.
- 2% FINISHED GRADE
- (2%) EXISTING GRADE
- UTILITY POINT OF CONNECTION
- DETAIL CALLOUT: DETAIL "#" / PAGE "⊙"
- CROSS-SECTION CALLOUT
- SIZE PIPE SIZE & DIRECTION

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PROJECT NO.	20153715
DRAWN BY:	JDS
CHECKED BY:	BH
DATE:	12-17-2014
REVISED:	

PROPOSED IMPROVEMENTS

MONTEREY - SALINAS TRANSIT DISTRICT
OPERATION & MAINTENANCE FACILITY
1 RYAN RANCH ROAD
MONTEREY, CALIFORNIA

PLATE

3

CAD FILE: L:\2014\14Projects\20153715-MST\GRAPHICS\PRO-SITE.dwg

APPENDIX A

Field Explorations

SAMPLE/SAMPLER TYPE GRAPHICS

- BULK SAMPLE
- CALIFORNIA SAMPLER (3 in. (76.2 mm.) outer diameter)
- STANDARD PENETRATION SPLIT SPOON SAMPLER (2 in. (50.8 mm.) outer diameter and 1-3/8 in. (34.9 mm.) inner diameter)

GROUND WATER GRAPHICS

- WATER LEVEL (level where first observed)
- WATER LEVEL (level after exploration completion)
- WATER LEVEL (additional levels after exploration)
- OBSERVED SEEPAGE

NOTES

- The report and graphics key are an integral part of these logs. All data and interpretations in this log are subject to the explanations and limitations stated in the report.
- Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual or differ from those shown.
- No warranty is provided as to the continuity of soil or rock conditions between individual sample locations.
- Logs represent general soil or rock conditions observed at the point of exploration on the date indicated.
- In general, Unified Soil Classification System designations presented on the logs were based on visual classification in the field and were modified where appropriate based on gradation and index property testing.
- Fine grained soils that plot within the hatched area on the Plasticity Chart, and coarse grained soils with between 5% and 12% passing the No. 200 sieve require dual USCS symbols, ie., GW-GM, GP-GM, GW-GC, GP-GC, GC-GM, SW-SM, SP-SM, SW-SC, SP-SC, SC-SM.
- If sampler is not able to be driven at least 6 inches then 50/X indicates number of blows required to drive the identified sampler X inches with a 140 pound hammer falling 30 inches.

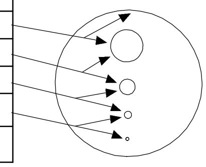
UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

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 GRAVEL WITH <5% FINES	Cu ≥4 and 1≤ Cc≤3	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			Cu <4 and/ or 1>Cc >3	GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH 5% TO 12% FINES	Cu ≥4 and 1≤ Cc≤3	GW-GM	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GW-GC	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
			Cu <4 and/ or 1>Cc >3	GP-GM	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GP-GC	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
		GRAVELS WITH > 12% FINES		GM	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
				GC-GM	CLAYEY GRAVELS, GRAVEL-SAND-CLAY-SILT MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH <5% FINES	Cu ≥6 and 1≤ Cc≤3	SW	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			Cu <6 and/ or 1>Cc >3	SP	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH 5% TO 12% FINES	Cu ≥6 and 1≤ Cc≤3	SW-SM	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SW-SC	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
			Cu <6 and/ or 1>Cc >3	SP-SM	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SP-SC	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
		SANDS WITH > 12% FINES		SM	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-GRAVEL-CLAY MIXTURES
				SC-SM	CLAYEY SANDS, SAND-SILT-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 AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, SILTS WITH SLIGHT PLASTICITY
		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		CL-ML	INORGANIC CLAYS-SILTS OF LOW 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

GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		>12 in. (304.8 mm.)	>12 in. (304.8 mm.)	Larger than basketball-sized
Cobbles		3 - 12 in. (76.2 - 304.8 mm.)	3 - 12 in. (76.2 - 304.8 mm.)	Fist-sized to basketball-sized
Gravel	coarse	3/4 -3 in. (19 - 76.2 mm.)	3/4 -3 in. (19 - 76.2 mm.)	Thumb-sized to fist-sized
	fine	#4 - 3/4 in. (#4 - 19 mm.)	0.19 - 0.75 in. (4.8 - 19 mm.)	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19 in. (2 - 4.9 mm.)	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079 in. (0.43 - 2 mm.)	Sugar-sized to rock salt-sized
	fine	#200 - #10	0.0029 - 0.017 in. (0.07 - 0.43 mm.)	Flour-sized to sugar-sized
Fines		Passing #200	<0.0029 in. (<0.07 mm.)	Flour-sized and smaller

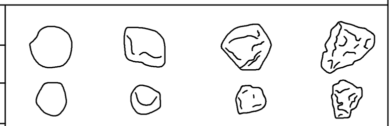


Munsell Color

NAME	ABBR
Red	R
Yellow Red	YR
Yellow	Y
Green Yellow	GY
Green	G
Blue Green	BG
Blue	B
Purple Blue	PB
Purple	P
Red Purple	RP
Black	N

ANGULARITY

DESCRIPTION	CRITERIA
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular	Particles are similar to angular description but have rounded edges
Subrounded	Particles have nearly plane sides but have well-rounded corners and edges
Rounded	Particles have smoothly curved sides and no edges



PLASTICITY

DESCRIPTION	LL	FIELD TEST
Non-plastic	NP	A 1/8-in. (3 mm.) thread cannot be rolled at any water content.
Low (L)	< 30	The thread can barely be rolled and the lump or thread cannot be formed when drier than the plastic limit.
Medium (M)	30 - 50	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump or thread crumbles when drier than the plastic limit
High (H)	> 50	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump or thread can be formed without crumbling when drier than the plastic limit

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 is below water table

REACTION WITH HYDROCHLORIC ACID

DESCRIPTION	FIELD TEST
None	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Violent reaction, with bubbles forming immediately

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT-N_{60} (# blows/ft)	MODIFIED CA SAMPLER (# blows/ft)	CALIFORNIA SAMPLER (# blows/ft)	RELATIVE DENSITY (%)
Very Loose	<4	<4	<5	0 - 15
Loose	4 - 10	5 - 12	5 - 15	15 - 35
Medium Dense	10 - 30	12 - 35	15 - 40	35 - 65
Dense	30 - 50	35 - 60	40 - 70	65 - 85
Very Dense	>50	>60	>70	85 - 100

NOTE: AFTER TERZAGHI AND PECK, 1948

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	UNCONFINED COMPRESSIVE STRENGTH (q_u)(psf)	CRITERIA
Very Soft	< 1000	Thumb will penetrate soil more than 1 in. (25 mm.)
Soft	1000 - 2000	Thumb will penetrate soil about 1 in. (25 mm.)
Firm	2000 - 4000	Thumb will indent soil about 1/4-in. (6 mm.)
Hard	4000 - 8000	Thumb will not indent soil but readily indented with thumbnail
Very Hard	> 8000	Thumbnail will not indent soil

STRUCTURE

DESCRIPTION	CRITERIA
Stratified	Alternating layers of varying material or color with layers at least 1/4-in. thick, note thickness
Laminated	Alternating layers of varying material or color with the layer less than 1/4-in. thick, note thickness
Fissured	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided	Fracture planes appear polished or glossy, sometimes striated
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay; note thickness
Homogeneous	Same color and appearance throughout

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	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

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PROJECT NO.:	20153715
DRAWN BY:	JDS
CHECKED BY:	AB
DATE:	12/2/2014
REVISED:	-

SOIL DESCRIPTION KEY

MONTEREY - SALINAS TRANSIT DISTRICT
OPERATION & MAINTENANCE FACILITY
1 RYAN RANCH ROAD
MONTEREY, CALIFORNIA

PLATE

A-2

Date Begin - End:	11/21/2014	**Drilling Company:**	Exploration Geoservices
Logged By:	A. Bord	**Drill Crew:**	J.R. & R.N.
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	B-53
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger /
Weather:	Overcast	**Exploration Diameter:**	8 in. O.D.

BORING LOG B-1

Hammer Type - Drop: 140 lb. Sandline - 30 in.

Surface Condition: Asphalt

Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	approximate 6 inches of asphalt												
	approximate 6 inches of aggregate baserock	1		BC=11 16 23	12"								
	Poorly-graded SAND (SP): fine grained, light gray, moist, medium dense, with some silt, with trace fine sub-rounded gravel	2		BC=14 16 18	12"						28	4	
5	**Silty SAND (SM)**: fine to coarse grained, light olive brown, moist, medium dense, trace fine sub-rounded gravel fine grained at 4'												
10	**Decomposed to highly weathered Sandstone-as Clayey Sand (SC)**: fine to coarse grained, medium to high plasticity, dark brown, with pale gray, moist, dense, with layers of orange-brown mottled pale gray sandstone	3		BC=14 15 20									
15		4		BC=21 16 20									
20	fine grained, low plasticity	5		BC=16 20 27									
25	medium plasticity, medium dense	6		BC=8 9 14									
30	low plasticity, dense	7		BC=12 14 22									

The exploration was terminated at approximately 30 ft. below ground surface. The exploration was backfilled with auger cuttings and patched at surface on November 21, 2014.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.
GENERAL NOTES:

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PROJECT NO.:	20153715
DRAWN BY:	JDS
CHECKED BY:	AB
DATE:	12/3/2014
REVISED:	12/19/2014

BORING LOG B-1

MONTEREY - SALINAS TRANSIT DISTRICT
OPERATION & MAINTENANCE FACILITY
1 RYAN RANCH ROAD
MONTEREY, CALIFORNIA

PLATE
A-3

Date Begin - End:	11/21/2014	**Drilling Company:**	Exploration Geoservices		**BORING LOG B-2**
Logged By:	A. Bord	**Drill Crew:**	J.R. & R.N.		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	B-53	**Hammer Type - Drop:**	140 lb. Sandline - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger /		
Weather:	Overcast	**Exploration Diameter:**	8 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Surface Condition: Asphalt

Depth (feet)	Lithologic Description	Sample Number	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	approximate 5-1/2 inches of asphalt											
	approximate 3 inches of aggregate baserock											
	Clayey SAND (SC): fine to coarse grained, medium plasticity, brown, moist, very dense, with some fine to medium sub-rounded gravel	1	BC=30 42 39	12"						37	21	
	Silty SAND (SM): fine grained, low plasticity, gray brown, moist, dense, trace large sub-rounded gravel	2	BC=14 21 32	18"		16.3	107.5					
5												
	Clayey SAND with Gravel (SC): fine grained, yellowish brown, moist, dense, fine sub-rounded gravel	3	BC=14 21 24						29			
10												
	Clayey SAND (SC): fine grained, medium plasticity, light olive, moist, dense, trace fine sub-rounded gravel	4	BC=13 14 18						24			
15												
		5	BC=14 17 27									
20												
	Silty SAND (SM): fine grained, low plasticity, light brown, moist, medium dense	6	BC=11 11 13									
25												
	dense	7	BC=14 19 27									
30												

The exploration was terminated at approximately 30 ft. below ground surface. The exploration was backfilled with auger cuttings and patched at surface on November 21, 2014.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.
GENERAL NOTES:

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.: 20153715	BORING LOG B-2	PLATE
DRAWN BY: JDS		
CHECKED BY: AB	MONTEREY - SALINAS TRANSIT DISTRICT OPERATION & MAINTENANCE FACILITY 1 RYAN RANCH ROAD MONTEREY, CALIFORNIA	A-4
DATE: 12/3/2014		
REVISED: 12/19/2014		PAGE: 1 of 1

Date Begin - End:	11/20/2014	Drilling Company:	Exploration Geoservices	**BORING LOG B-3**
Logged By:	R. Hasseler	**Drill Crew:**	J.R. & R.N.	
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	B-53	**Hammer Type - Drop:** 140 lb. Sandline - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger /	
Weather:	Overcast	**Exploration Diameter:**	8 in. O.D.	

Surface Condition: Asphalt

Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
0	approximate 3-1/2 inches of asphalt												
	approximate 5 inches of aggregate baserock												
	Clayey SAND (SC): fine grained, low plasticity, reddish brown, moist, dense	1		BC=16 24 30	15"		19.8	97.5					
	Poorly-graded SAND with Silt (SP-SM): fine grained, low plasticity, yellowish brown, moist, very dense	2		BC=12 26 50/6"	17"								
5													
	Silty SAND (SM): fine grained, low plasticity, yellowish brown, moist, dense	3		BC=14 17 17	18"								
10													
	Clayey SAND (SC): fine to coarse grained, low plasticity, yellowish brown, moist, medium dense	4		BC=16 10 15	18"								
15													
		5		BC=14 14 15	18"								
20													
	Silty SAND (SM): fine to medium grained, low plasticity, yellowish brown, moist, dense	6		BC=23 27 21	18"								
25													
		7		BC=12 17 21	18"								
30													

The exploration was terminated at approximately 30 ft. below ground surface. The exploration was backfilled with auger cuttings and patched at surface on November 20, 2014.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.
GENERAL NOTES:

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20153715	BORING LOG B-3	PLATE
	DRAWN BY: JDS		
	CHECKED BY: AB	MONTEREY - SALINAS TRANSIT DISTRICT OPERATION & MAINTENANCE FACILITY 1 RYAN RANCH ROAD MONTEREY, CALIFORNIA	A-5
	DATE: 12/3/2014		
	REVISED: 12/19/2014		PAGE: 1 of 1

Date Begin - End:	11/20/2014	Drilling Company:	Exploration Geoservices	**BORING LOG B-4**
Logged By:	R. Hasseler	Drill Crew:	J.R. & R.N.	
Hor.-Vert. Datum:	Not Available	Drilling Equipment:	B-53	Hammer Type - Drop: 140 lb. Sandline - 30 in.
Plunge:	-90 degrees	Drilling Method:	Hollow Stem Auger /	
Weather:	Overcast	Exploration Diameter:	8 in. O.D.	

Surface Condition: Asphalt

Depth (feet)	Lithologic Description	Sample Number	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	approximate 2 inches of asphalt											
	approximate 6 inches of aggregate baserock											
	Silty SAND (SM): fine grained, low plasticity, light reddish brown, moist, loose, (FILL)	1	BC=6 4 4	18"								
5	**Decomposed to highly weathered Sandstone- as Silty Sand (SM)**: fine grained, low plasticity, pink, dry, medium dense	2	BC=16 15 21	18"								
10	reddish yellow, very dense	3	BC=28 50/5"	11"								
15		4	BC=41 50/6"	12"								
20		5	BC=28 29 35	18"								
25		6	BC=19 50/6"	12"								
		7	BC=50/2"	2"								
30												

The exploration was terminated at approximately 29 ft. below ground surface. The exploration was backfilled with auger cuttings and patched at surface on November 20, 2014.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.

GENERAL NOTES:

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.: 20153715	BORING LOG B-4	PLATE
DRAWN BY: JDS		
CHECKED BY: AB	MONTEREY - SALINAS TRANSIT DISTRICT OPERATION & MAINTENANCE FACILITY 1 RYAN RANCH ROAD MONTEREY, CALIFORNIA	A-6
DATE: 12/3/2014		
REVISED: 12/19/2014		PAGE: 1 of 1

Date Begin - End:	11/21/2014	**Drilling Company:**	Exploration Geoservices			**BORING LOG B-5**
Logged By:	A. Bord	**Drill Crew:**	J.R. & R.N.			
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	B-53	**Hammer Type - Drop:**	140 lb. Sandline - 30 in.	
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger /			
Weather:	Overcast	**Exploration Diameter:**	8 in. O.D.			

Surface Condition: Gravel

Depth (feet)	Graphical Log	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		approximate 2 inches of decorative aggregate baserock												
		Silty SAND (SM): fine grained, low plasticity, dark brown, moist, medium dense	1		BC=17 50/5.5"	11"								
		Decomposed to highly weathered Sandstone-as Sandy Silt (ML): fine grained, low plasticity, light yellowish brown, dry, very dense, with some fine to coarse gravel and sandstone lenses	2		BC=50/3"	3"								
5														
		dense, with some fine to medium gravel	3		BC=15 19 23	18"				100	67			
10														
		very dense	4		BC=18 50/5"	12"								
15														
		dense	5		BC=13 14 18	18"								
20														
			6		BC=16 21 20	18"								
25														
			7		BC=13 16 22	18"								
30														

The exploration was terminated at approximately 30 ft. below ground surface. The exploration was backfilled with auger cuttings and patched at surface on November 21, 2014.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.
GENERAL NOTES:

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.: 20153715	BORING LOG B-5	PLATE
DRAWN BY: JDS		
CHECKED BY: AB	MONTEREY - SALINAS TRANSIT DISTRICT OPERATION & MAINTENANCE FACILITY 1 RYAN RANCH ROAD MONTEREY, CALIFORNIA	A-7
DATE: 12/3/2014		
REVISED: 12/19/2014		PAGE: 1 of 1

Date Begin - End:	11/21/2014	**Drilling Company:**	Exploration Geoservices		**BORING LOG B-6**
Logged By:	A. Bord	**Drill Crew:**	J.R. & R.N.		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	B-53	**Hammer Type - Drop:**	140 lb. Sandline - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger /		
Weather:	Overcast	**Exploration Diameter:**	8 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Surface Condition: Asphalt

Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	approximate 3 inches of asphalt												R-Value
	approximate 4 inches of aggregate baserock	1		BC=36 50/4"	6"								
	Silty SAND with Gravel (SM): fine to medium grained, low plasticity, light brown & gray, dry, very dense, with some coarse sand, fine to medium sub-rounded to angular gravel (FILL)	2		BC=50/4.5"	5"								Direct Shear C = 198 psf Ø = 31°
5	brown with gray												
		3		BC=27 33 41									
10	**Decomposed to highly weathered Sandstone-as Silty Sand (SM)**: fine grained, low plasticity, light olive brown, dry, very dense												difficult drilling - Sandstone
		4		BC=11 18 28						46			
15													
	gravel band, 1.5 feet thick, fine to coarse rounded gravel												
20		5		BC=11 13 17									
		6		BC=31 50/6"									
25													
30		7		BC=14 19 25									

The exploration was terminated at approximately 30 ft. below ground surface. The exploration was backfilled with auger cuttings and patched at surface on November 21, 2014.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.
GENERAL NOTES:

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.:	20153715
DRAWN BY:	JDS
CHECKED BY:	AB
DATE:	12/3/2014
REVISED:	12/19/2014

BORING LOG B-6

MONTEREY - SALINAS TRANSIT DISTRICT
OPERATION & MAINTENANCE FACILITY
1 RYAN RANCH ROAD
MONTEREY, CALIFORNIA

PLATE
A-8

Date Begin - End:	11/20/2014	**Drilling Company:**	Exploration Geoservices		**BORING LOG B-7**
Logged By:	R. Hasseler	**Drill Crew:**	J.R. & R.N.		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	B-53	**Hammer Type - Drop:**	140 lb. Sandline - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger /		
Weather:	Overcast	**Exploration Diameter:**	8 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Surface Condition: Asphalt

Depth (feet)	Lithologic Description	Sample Number	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	approximate 1-1/2 inches of asphalt											
	approximate 2 inches of aggregate baserock	1	BC=50/5"	6"		3.9	91.8					
	Silty SAND (SM): fine grained, low plasticity, gray brown, moist, very dense											
		2	BC=29 50/6"	12"								Swell/Collapse
5												
		3	BC=8 8 11	18"								
10	medium dense											
		4	BC=14 17 22	18"								
15	**Clayey SAND (SC)**: fine grained, low plasticity, reddish yellow to light reddish brown, moist, dense											
	Silty SAND (SM): fine to coarse grained, low plasticity, reddish brown, moist, very dense dense	5	BC=27 41 50/5"	17"								
20												
		6	BC=14 22 50/5.5"	18"								
25												
		7	BC=26 50/5.5"	12"								
30												

The exploration was terminated at approximately 29.5 ft. below ground surface. The exploration was backfilled with auger cuttings and patched at surface on November 20, 2014.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.
GENERAL NOTES:

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20153715	BORING LOG B-7	PLATE
	DRAWN BY: JDS		
	CHECKED BY: AB	MONTEREY - SALINAS TRANSIT DISTRICT OPERATION & MAINTENANCE FACILITY 1 RYAN RANCH ROAD MONTEREY, CALIFORNIA	A-9
	DATE: 12/3/2014		
	REVISED: 12/19/2014		PAGE: 1 of 1

Date Begin - End:	11/20/2014	**Drilling Company:**	Exploration Geoservices		**BORING LOG B-8**
Logged By:	R. Hasseler	**Drill Crew:**	J.R. & R.N.		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	B-53	**Hammer Type - Drop:**	140 lb. Sandline - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger /		
Weather:	Overcast	**Exploration Diameter:**	8 in. O.D.		

Depth (feet)	Graphical Log	FIELD EXPLORATION: Surface Condition: Asphalt — Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	LABORATORY RESULTS: USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		approximate 5 inches of asphalt	1											
		approximate 3-1/2 inches of aggregate baserock												
5		**Silty SAND (SM)**: fine grained, low plasticity, yellowish brown, moist												

The exploration was terminated at approximately 5 ft. below ground surface. The exploration was backfilled with auger cuttings and patched at surface on November 20, 2014.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.
GENERAL NOTES:

10

15

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25

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KLEINFELDER *Bright People. Right Solutions.*	PROJECT NO.: 20153715	BORING LOG B-8	PLATE
	DRAWN BY: JDS		A-10
	CHECKED BY: AB	MONTEREY - SALINAS TRANSIT DISTRICT OPERATION & MAINTENANCE FACILITY 1 RYAN RANCH ROAD MONTEREY, CALIFORNIA	
	DATE: 12/3/2014		
	REVISED: 12/19/2014		PAGE: 1 of 1

Date Begin - End:	11/20/2014	**Drilling Company:**	Exploration Geoservices		**BORING LOG B-9**
Logged By:	R. Hasseler	**Drill Crew:**	J.R. & R.N.		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	B-53	**Hammer Type - Drop:**	140 lb. Sandline - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger /		
Weather:	Overcast	**Exploration Diameter:**	8 in. O.D.		

Depth (feet)	Graphical Log	FIELD EXPLORATION: Surface Condition: Asphalt / Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		approximate 4 inches of asphalt	2											R-Value
		approximate 4 inches of aggregate baserock												
5		**Clayey SAND (SC)**: fine grained, low plasticity, reddish brown, moist												

10

15

20

25

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The exploration was terminated at approximately 5 ft. below ground surface. The exploration was backfilled with auger cuttings and patched at surface on November 20, 2014.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.

GENERAL NOTES:

APPENDIX B

Results of Percolation Testing

SOIL PERCOLATION TEST RECORDED MEASUREMENTS

OWNER/APPLICANT:	Monterey-Salinas Transit	PROJECT:	Monterey-Salinas Transit
SITE LOCATION:	1 Ryan Ranch Road, Monterey, CA	PROJECT NUMBER:	
CONTACT/TELEPHONE:		DATE:	11/20/14 and 11/21/14
		REHS:	

HOLE #:	P-1	PRESATURATE DATE/TIME:	8:45:00 AM 11/20/2014
DIAMETER:	8 inches	PRESATURATE WATER DEPTH:	6.40 feet
HOLE DEPTH:	15.00 feet	HOLE DEPTH (Next Day) /TIME:	13.40 feet
SOIL TYPE:	Silty Sand and Clayey Sand	WATER DEPTH (Next Day):	10.39 feet

READING	DATE	TIME START	TIME FINISH	WATER LEVEL (in) START	WATER LEVEL (in) FINISH	ELAPSED TIME MIN.	WATER FALL INCHES	PERCOLATION RATE MINUTES/INCH*
1	11/21/2014	12:44	12:54	47.95	54.79	10	6.840	1.5
2	11/21/2014	12:54	1:04	54.79	58.15	10	3.360	3.0
3	11/21/2014	1:04	1:14	58.15	59.95	10	1.800	5.6
4	11/21/2014	1:14	1:24	59.95	61.75	10	1.800	5.6
5	11/21/2014	1:24	1:34	61.75	63.55	10	1.800	5.6
6	11/21/2014	1:34	1:44	63.55	65.47	10	1.920	5.2
7	11/21/2014	1:44	1:54	65.47	67.27	10	1.800	5.6
8	11/21/2014	1:54	2:04	67.27	69.07	10	1.800	5.6
9	11/21/2014	2:04	2:14	69.07	70.87	10	1.800	5.6

RATE: 5.6 min/in * Slow Percolation Below 10.4 feet bgs

HOLE #:	P-2	PRESATURATE DATE/TIME:	9:15:00 AM 11/20/2014
DIAMETER:	8 inches	PRESATURATE WATER DEPTH:	6.72 feet
HOLE DEPTH:	15.00 feet	HOLE DEPTH (Next Day) /TIME:	13.52 feet
SOIL TYPE:	Silty Sand and Clayey Sand	WATER DEPTH (Next Day):	13.05 feet

READING	DATE	TIME START	TIME FINISH	WATER LEVEL (in) START	WATER LEVEL (in) FINISH	ELAPSED TIME MIN.	WATER FALL INCHES	PERCOLATION RATE MINUTES/INCH*
1	11/21/2014	9:33	9:43	79.40	87.44	10	8.040	1.2
2	11/21/2014	9:43	9:53	87.44	92.48	10	5.040	2.0
3	11/21/2014	9:53	10:03	92.48	97.64	10	5.160	1.9
4	11/21/2014	10:03	10:13	97.64	100.76	10	3.120	3.2
5	11/21/2014	10:13	10:23	100.76	104.60	10	3.840	2.6
6	11/21/2014	10:23	10:33	104.60	108.44	10	3.840	2.6
7	11/21/2014	10:33	10:43	108.44	112.16	10	3.720	2.7

RATE: 2.7 min/in * Slow Percolation Below 13.1 feet bgs

SOIL PERCOLATION TEST RECORDED MEASUREMENTS

OWNER/APPLICANT:	Monterey-Salinas Transit	PROJECT:	Monterey-Salinas Transit
SITE LOCATION:	1 Ryan Ranch Road, Monterey, CA	PROJECT NUMBER:	
CONTACT/TELEPHONE:		DATE:	11/20/14 and 11/21/14
		REHS:	

HOLE #:	P-3	PRESATURATE DATE/TIME:	9:45:00 AM	11/20/2014
DIAMETER:	8 inches	PRESATURATE WATER DEPTH:	6.58	feet
HOLE DEPTI	15.00 feet	HOLE DEPTH (Next Day) /TIME:	13.18	feet
SOIL TYPE:	Silty Sand and Clayey Sand	WATER DEPTH (Next Day):	8.86	feet

READING	DATE	TIME START	TIME FINISH	WATER LEVEL (in) START	WATER LEVEL (in) FINISH	ELAPSED TIME MIN.	WATER FALL INCHES	PERCOLATION RATE MINUTES/INCH*
1	11/21/2014	11:02	11:12	1.00	26.08	10	25.080	0.4
2	11/21/2014	11:12	11:22	26.08	33.88	10	7.800	1.3
3	11/21/2014	11:22	11:32	33.88	39.28	10	5.400	1.9
4	11/21/2014	11:32	11:42	39.28	44.08	10	4.800	2.1
5	11/21/2014	11:42	11:52	44.08	48.16	10	4.080	2.5
6	11/21/2014	11:52	12:02	48.16	51.04	10	2.880	3.5
7	11/21/2014	12:02	12:12	51.04	54.04	10	3.000	3.3
8	11/21/2014	12:12	12:22	54.04	56.80	10	2.760	3.6
9	11/21/2014	12:22	12:32	56.80	59.56	10	2.760	3.6

RATE: 3.6 min/in

* Slow Percolation Below 8.9 feet bgs

APPENDIX C

Laboratory Testing Results

Exploration ID	Depth (ft.)	Sample No.	Sample Description	Water Content (%)	Dry Unit Wt. (pcf)	Sieve Analysis (%)			Atterberg Limits			Additional Tests
						Passing 3/4"	Passing #4	Passing #200	Liquid Limit	Plastic Limit	Plasticity Index	
B-1	4.0		LIGHT OLIVE BROWN SILTY SAND (SM)						28	24	4	
B-2	2.0		BROWN CLAYEY SAND (SC)						37	16	21	
B-2	3.5		GRAY BROWN SILTY SAND (SM)	16.3	107.5							
B-2	8.5 - 10.0	3	YELLOWISH BROWN CLAYEY SAND WITH GRAVEL (SC)					29				
B-2	13.5 - 15.0	4	LIGHT OLIVE CLAYEY SAND (SC)					24				
B-3	2.0		REDDISH BROWN CLAYEY SAND (SC)	19.8	97.5							
B-5	8.5 - 10.0	3	LIGHT YELLOWISH BROWN SANDY SILT (ML)				100	67				
B-6	0.0 - 5.0		LIGHT BROWN AND GRAY SILTY SAND WITH GRAVEL (SM)									R-Value
B-6	1.5 - 3.5		LIGHT BROWN AND GRAY SILTY SAND WITH GRAVEL (SM)									Direct Shear
												C = 198 psf
												Ø = 31°
B-6	13.5 - 15.0	4	LIGHT OLIVE BROWN SILTY SAND (SM)					46				
B-7	1.0	1	GRAY GROWN SILTY SAND (SM)	3.9	91.8							
B-7	4.0											Swell/Collapse
B-9	0.0 - 5.0	2	REDDISH BROWN CLAYEY SAND									R-Value

Refer to the Geotechnical Evaluation Report or the supplemental plates for the method used for the testing performed above.
NP = NonPlastic
NA = Not Available

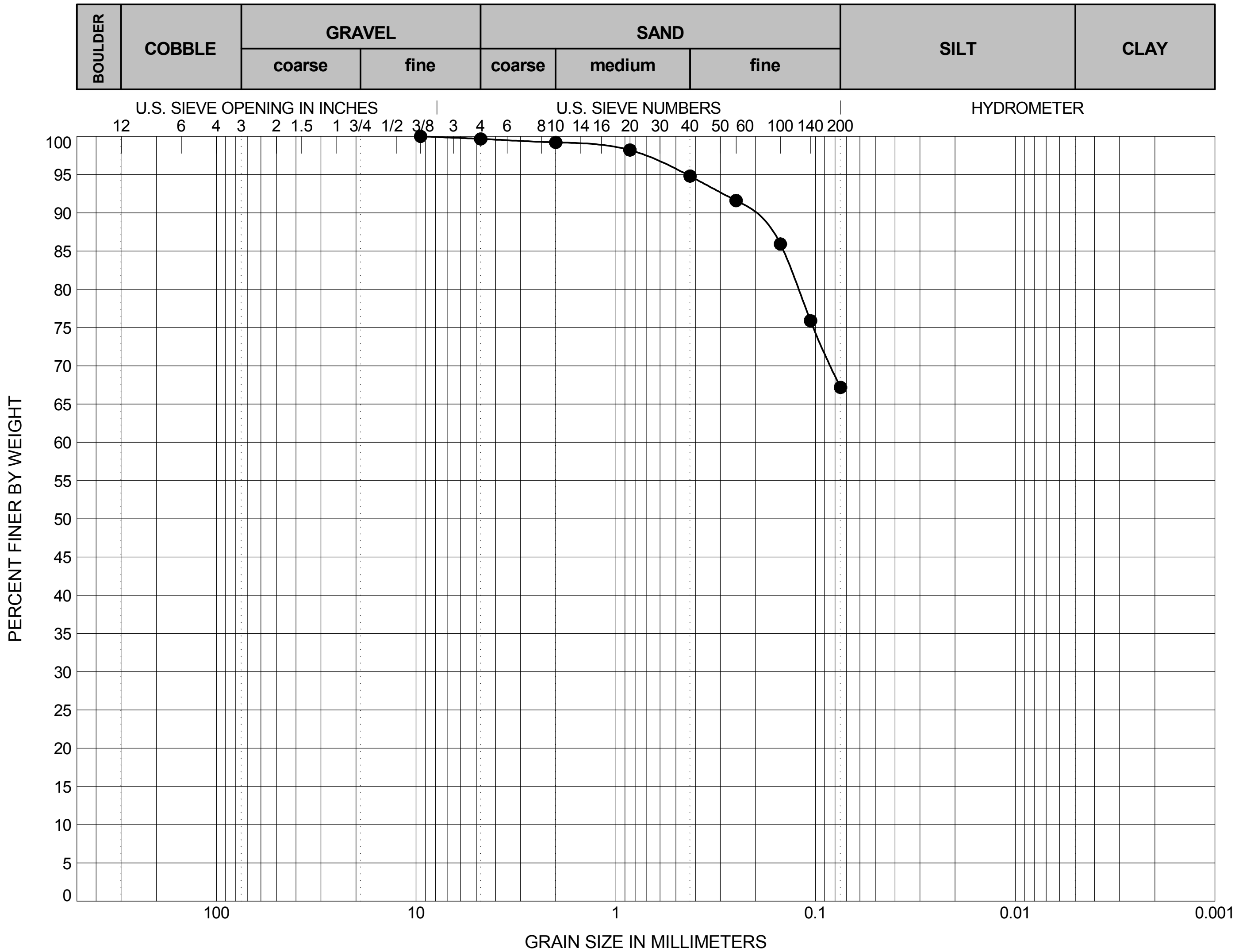
KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.: 20153715
DRAWN BY: JDS
CHECKED BY: AB
DATE: 12/2/2014
REVISED: 12/19/2014

LABORATORY TEST RESULT SUMMARY

MONTEREY - SALINAS TRANSIT DISTRICT
OPERATION & MAINTENANCE FACILITY
1 RYAN RANCH ROAD
MONTEREY, CALIFORNIA

TABLE
C-1



Exploration ID	Depth (ft.)	Sample Number	Sample Description	LL	PL	PI
● B-5	8.5 - 10	3	LIGHT YELLOWISH BROWN SANDY SILT (ML)	NM	NM	NM

Exploration ID	Depth (ft.)	D_{100}	D_{60}	D_{30}	D_{10}	Cc	Cu	Passing 3/4"	Passing #4	Passing #200	%Silt	%Clay
● B-5	8.5 - 10	9.5	NM	NM	NM	NM	NM		100	67	NM	NM

Sieve Analysis and Hydrometer Analysis testing performed in general accordance with ASTM D422.
NP = Nonplastic
NA = Not Available
NM = Not Measured

Coefficients of Uniformity - $C_u = D_{60} / D_{10}$
Coefficients of Curvature - $C_C = (D_{30})^2 / D_{60} D_{10}$
D_{60} = Grain diameter at 60% passing
D_{30} = Grain diameter at 30% passing
D_{10} = Grain diameter at 10% passing

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.:	20153715
DRAWN BY:	JDS
CHECKED BY:	AB
DATE:	12/3/2014
REVISED:	12/19/2014

SIEVE ANALYSIS

MONTEREY - SALINAS TRANSIT DISTRICT
OPERATION & MAINTENANCE FACILITY
1 RYAN RANCH ROAD
MONTEREY, CALIFORNIA

PLATE
C-2

PLOTTED: 01/12/2015 07:28 AM BY: jsala
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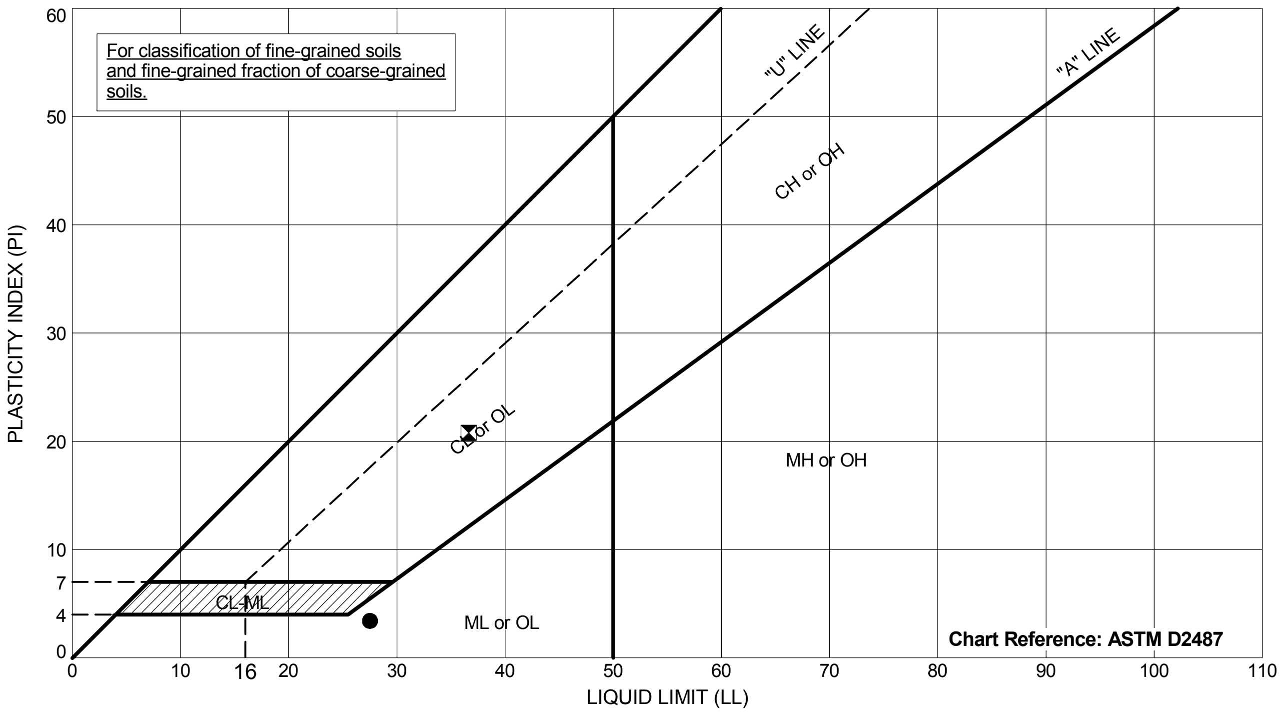
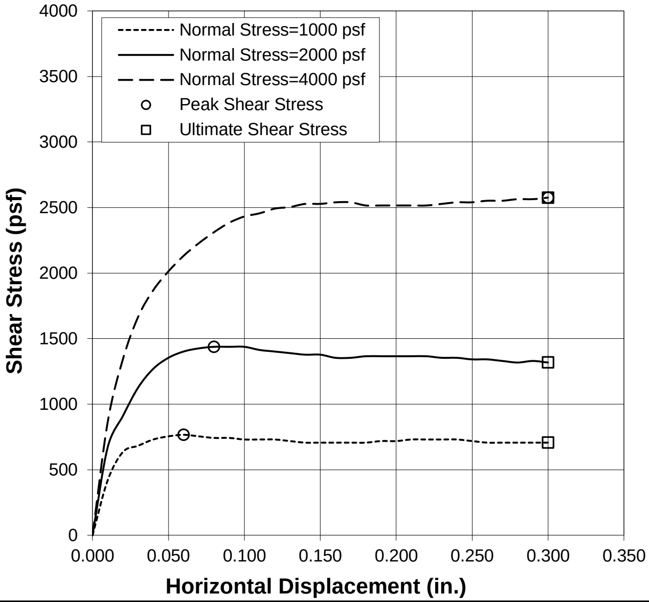
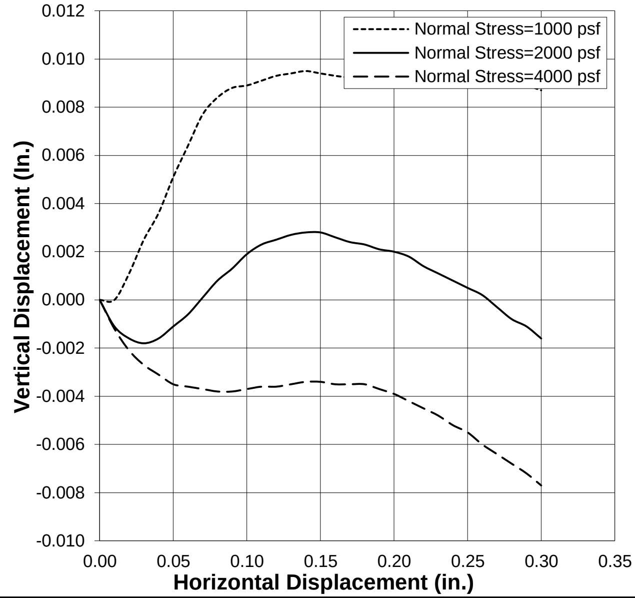


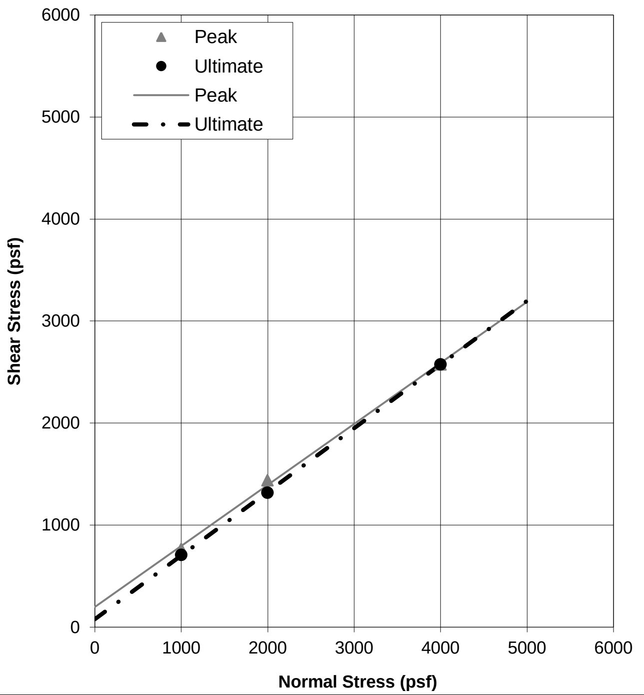
	Exploration ID	Depth (ft.)	Sample Number	Sample Description	Passing #200	LL	PL	PI
●	B-1	4	NA	LIGHT OLIVE BROWN SILTY SAND (SM)	NM	28	24	4
⧗	B-2	2	NA	BROWN CLAYEY SAND (SC)	NM	37	16	21

Testing perfomed in general accordance with ASTM D4318.
NP = Nonplastic
NA = Not Available
NM = Not Measured

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20153715 DRAWN BY: JDS CHECKED BY: AB DATE: 12/3/2014 REVISED: 12/19/2014	ATTERBERG LIMITS MONTEREY - SALINAS TRANSIT DISTRICT OPERATION & MAINTENANCE FACILITY 1 RYAN RANCH ROAD MONTEREY, CALIFORNIA	PLATE C-3







Specimen Number		1	2	3	4
Initial	Water Content, %	7.5	7.5	7.4	na
	Dry Density, pcf	107.5	107.9	107.8	na
	Saturation, %	36.9	37.3	36.5	na
	Void Ratio	0.538	0.532	0.534	na
	Diameter, in	2.42	2.42	2.42	na
	Height, in	1.00	1.00	1.00	na
Preshear	Water Content, %	18.6	18.2	17.3	na
	Dry Density, pcf	108.4	110.0	112.0	na
	Saturation, %	93.9	96.1	96.4	na
	Void Ratio	0.526	0.503	0.477	na
	Diameter, in	2.42	2.42	2.42	na
	Height, in	0.989	0.979	0.963	na
After	Water Content, %	18.6	18.2	17.6	na

Normal Stress, psf	1000	2000	4000	na
Peak Shear Stress, psf	767	1438	2576	na
Horz. Displ. at Peak Shear Stress, in.	0.060	0.080	0.300	na
Ultimate Shear Stress, psf	707	1318	2576	na
Horz. Displ. at Ultimate Shear Stress, in.	0.300	0.300	0.300	na
Strain Rate, in./min.	0.0060	0.0060	0.0060	na
	c, psf	ϕ, deg.	Tan ϕ	na
Peak	198	30.9	0.60	na
Ultimate	78	32.0	0.62	na

LL:	NM	PL:	NM	PI:	NM	G_S:	2.65	Assumed

Test Conditions: Undisturbed / Inundated

Description: Light Brown and Gray Silty Sand (SM)

Boring:	B-6
Sample:	1a&2a
Depth, ft:	1.5-3.5
Test Date:	12/17/14

Remarks: nm = not measured, na = not applicable

KLEINFELDER
Bright People. Right Solutions.

2601 Barrington Ct Hayward, CA

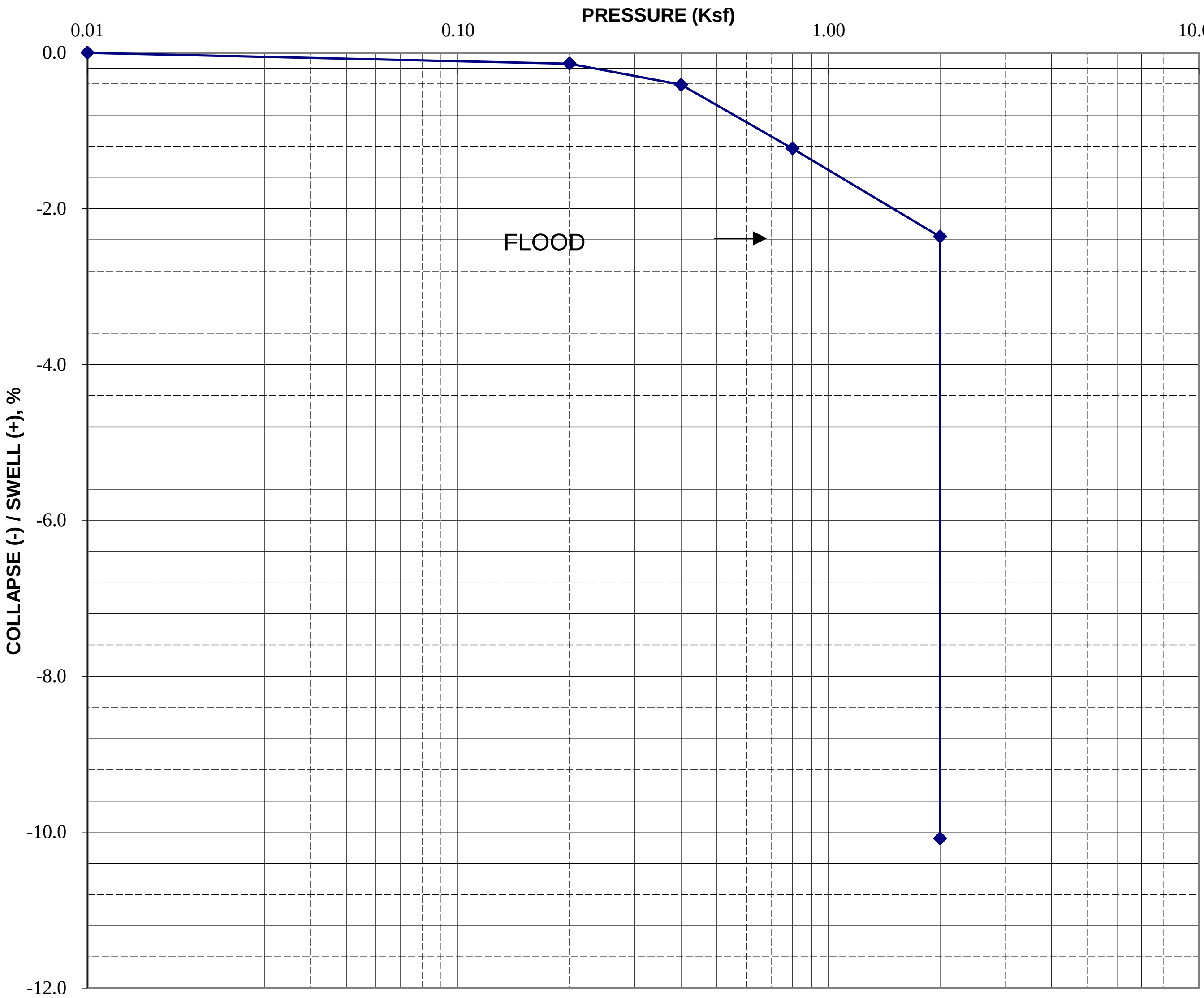
Project No:	20153715
Date:	1/12/15
Tested by:	CP
Checked By	RH
File Name:	HL7396

DIRECT SHEAR TEST ASTM D3080

MONTEREY-SALINAS TRANSIT DISTRICT
OPERATION & MAINTENANCE FACILITY
1 RYAN RANCH ROAD
MONTEREY, CALIFORNIA

Plate

C-4



BORING NO.	B-7
DEPTH, ft	4
SAMPLE DESCRIPTION	Brown Silty Sand (SM)
NET COLLAPSE (-) /SWELL (+), %	-0.4%

	INITIAL	FINAL
DRY DENSITY, psf	92.6	102.0
WATER CONTENT, %	4.1	17.9
SAMPLE HEIGHT, in.	1.0000	0.8992

WATER TYPE & SOURCE	DEIONIZED, TAP
SIZE AND % OF OVERSIZED MATERIAL	NA

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

Pursuant to 2006 IBC Section 1704, the results presented in this report are for the exclusive use of the client and the registered design professional in responsible charge. The results apply only to the samples tested. If changes to the specifications were made and not communicated to Kleinfelder, Kleinfelder assumes no responsibility for pass/fail (meets/does

KLEINFELDER
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Project No.: 20153715

ONE DIMENSIONAL SWELL*

MONTEREY-SALINAS TRANSIT DISTRICT
OPERATION & MAINTENANCE FACILITY
1 RYAN RANCH ROAD
MONTEREY, CALIFORNIA

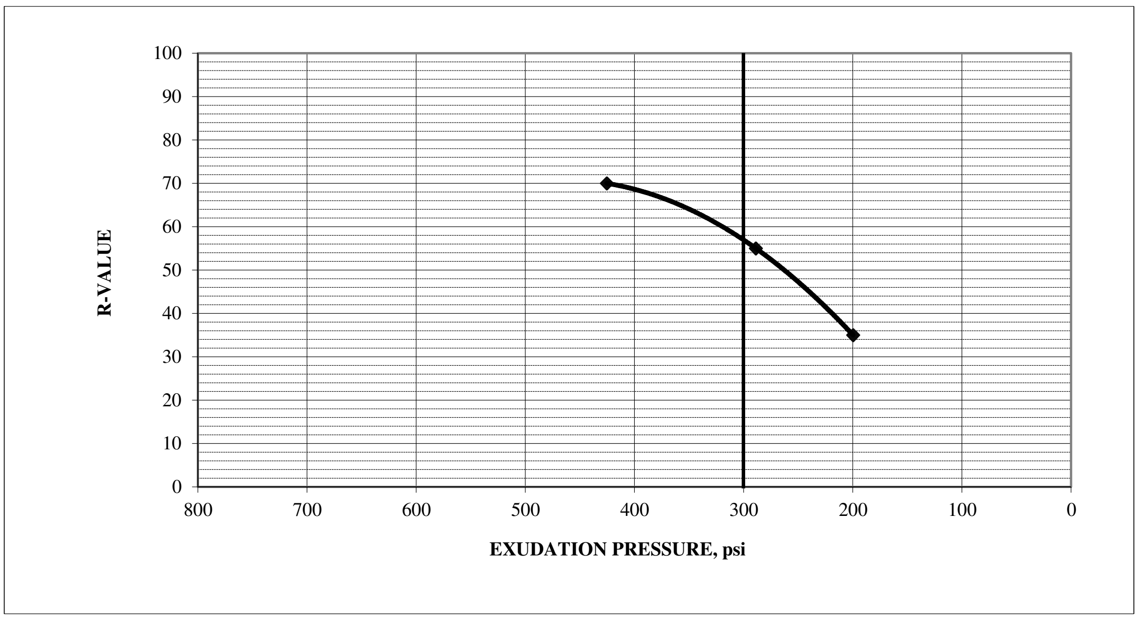
PLATE

C-5

Laboratory Test Report

Project Name: MONTEREY-SALINAS TRANSIT
Project No.: 20153715
Lab No.: HL7396
Sample Date: 11/21/14
Sample No.: Bulk B-6
Sample Location: B-6 @ 0 - 5.0'
Material Description: Light Brown and Gray Silty Sand with Gravel
Report Date: December 18, 2014

Resistance R-Value and Expansion Pressure of Compacted Soils (ASTM D2844, CTM 301)



Briquette No.	A	B	C
Moisture at Test, %	16.6	15.7	14.8
Dry Unit Weight at Test, pcf	108.2	108.3	109.7
Expansion Pressure, psf	4	13	22
Exudation Pressure, psi	200	289	425
Resistance Value	35	55	70
R - Value at 300 psi Exudation Pressure:			**57**

Reviewed By on 12/18/2014:

Aaron Kidd
Laboratory Manager

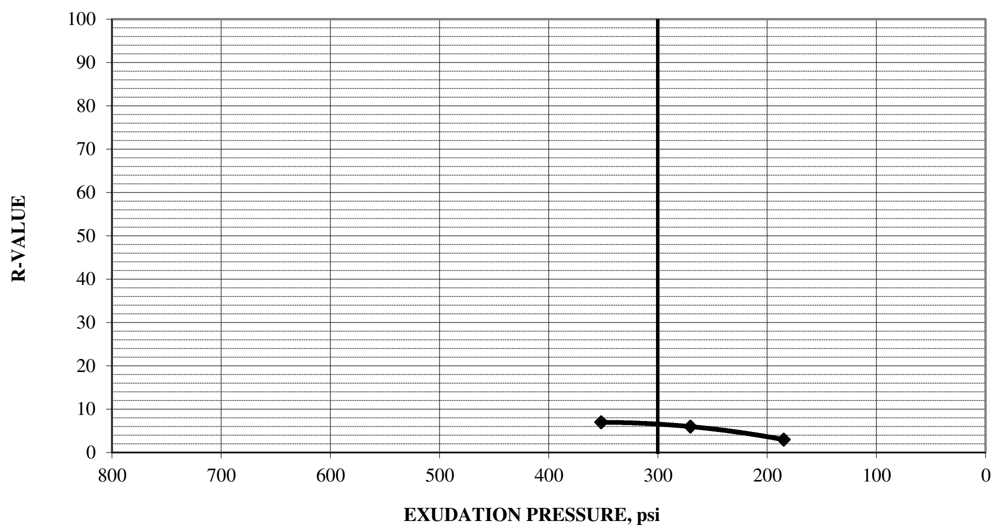
Limitations: Pursuant to applicable building codes, the results presented in this report are for the exclusive use of the client and the registered design professional in responsible charge. The results apply only to the samples tested. If changes to the specifications were made and not communicated to Kleinfelder, Kleinfelder assumes no responsibility for pass/fail statements (meets/did not meet), if provided.

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20153715 DRAWN BY: JDS CHECKED BY: RH DATE: 1/12/2015 REVISED:	R-VALUE MONTEREY - SALINAS TRANSIT DISTRICT OPERATION & MAINTENANCE FACILITY 1 RYAN RANCH ROAD MONTEREY, CALIFORNIA	PLATE C-6

Laboratory Test Report

Project Name: MONTEREY-SALINAS TRANSIT
Project No.: 20153715
Lab No.: HL7396
Sample Date: 11/20/14
Sample No.: Bulk B-9
Sample Location: B-9 @ 0 - 5.0'
Material Description: Reddish Brown Clayey Sand
Report Date: December 18, 2014

Resistance R-Value and Expansion Pressure of Compacted Soils (ASTM D2844, CTM 301)



Briquette No.	A	B	C
Moisture at Test, %	17.4	16.5	15.4
Dry Unit Weight at Test, pcf	111.9	0.0	111.9
Expansion Pressure, psf	4	13	22
Exudation Pressure, psi	185	270	352
Resistance Value	3	6	7
R - Value at 300 psi Exudation Pressure:			**7**

Reviewed By on 12/18/2014:

Aaron Kidd
Laboratory Manager

Limitations: Pursuant to applicable building codes, the results presented in this report are for the exclusive use of the client and the registered design professional in responsible charge. The results apply only to the samples tested. If changes to the specifications were made and not communicated to Kleinfelder, Kleinfelder assumes no responsibility for pass/fail statements (meets/did not meet), if provided.

PROJECT NO.: 20153715 DRAWN BY: JDS CHECKED BY: RH DATE: 1/12/2015 REVISED:	R-VALUE MONTEREY - SALINAS TRANSIT DISTRICT OPERATION & MAINTENANCE FACILITY 1 RYAN RANCH ROAD MONTEREY, CALIFORNIA	PLATE C-7

Appendix D

Corrosion Testing Laboratory Results

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

11 December, 2014

Job No.1412066
Cust. No.11454

Mr. Robert Hasseler
Kleinfelder
40 Clark Street, Suite J
Salinas, CA 93901

Subject: Project No.: 20153715
Project Name: Monterey Salinas Transit
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Hasseler:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on December 05, 2014. Based on the analytical results, a brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurement, this sample is classified as "corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentration is 58 mg/kg and is determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentration is 88 mg/kg and is determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at this location.

The pH of the soil is 5.59, which does present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures. Any soils with a pH of <6.0 is considered to be corrosive to buried iron, steel, mortar-coated steel and reinforced concrete structures. Therefore, corrosion prevention measures need to be considered for structures to be placed in this acidic soil.

The redox potential is 450-mV, which is indicative of aerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call JDH Corrosion Consultants, Inc. at (925) 927-6630.

We appreciate the opportunity of working with you on this project. If you have any questions, or if you required further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

Cheryl McMillen for

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: Kleinfelder
Client's Project No.: 20153715
Client's Project Name: Monterey Salinas Transit
Date Sampled: 4-Dec-14
Date Received: 5-Dec-14
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 11-Dec-2014

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1412066-001	B-2, 2A @ 4-4.5'	450	5.59	-	1,700	-	58	88

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	10	-	50	15	15
Date Analyzed:	10-Dec-2014	9-Oct-2014	-	8-Dec-2014	-	9-Dec-2014	9-Dec-2014

* Results Reported on "As Received" Basis

Cheryl McMillen
Laboratory Director

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

Appendix E

Summary of Compaction Recommendations

Exhibit 1

Summary of Compaction Recommendations

Area	Compaction Recommendation [1,2,3]
General Engineered Fill	Compact to a minimum of 90 percent compaction at a moisture content above the optimum moisture content. Compact fill slopes steeper than 3:1 (Horizontal:Vertical) to a minimum of 95 percent compaction at a moisture content above the optimum moisture content.
Imported Fill [4]	Compact to a minimum of 90 percent compaction at a moisture content above the optimum moisture content.
Trenches [5]	Compact to a minimum of 90 percent compaction at a moisture content above the optimum moisture content.
Exterior Flatwork [6]	Compact to a minimum of 90 percent compaction at a moisture content above the optimum moisture content. Compact baserock to a minimum of 95 percent compaction at near the optimum moisture content.
Parking and Access Driveways [6]	Compact to a minimum of 95 percent compaction at a moisture content above the optimum moisture content. Compact baserock to a minimum of 95 percent compaction at near the optimum moisture content.

Notes:

1. All compaction requirements refer to relative compaction as a percentage of the laboratory standard described by ASTM D 1557.
2. All lifts to be compacted shall be a maximum of 8 inches loose thickness, unless otherwise recommended.
3. All compacted surfaces should be firm, stable, and unyielding under compaction equipment.
4. Includes building and/or equipment pads.
5. In landscaping areas, this percent compaction in trenches may be reduced to 85 percent.
6. Depths are below finished subgrade elevation.

APPENDIX F

GBA Information Sheet

Important Information about Your
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.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical-engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical-Engineering Report Is Based on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk-management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical-engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical-engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical-engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations *only* by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical-engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical-engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of 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 equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical-engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold-prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, many mold-prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical-engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold-prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your GBA-Member Geotechncial Engineer for Additional Assistance

Membership in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk confrontaton techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your GBA-member geotechnical engineer for more information.

GBA GEOPROFESSIONAL BUSINESS ASSOCIATION

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@geoprofessional.org www.geoprofessional.org