



**GEOTECHNICAL INVESTIGATION REPORT
MANTECA WATER QUALITY CONTROL FACILITY
IMPROVEMENTS
2450 WEST YOSEMITE AVENUE
MANTECA, CALIFORNIA**

BSK PROJECT NO. G15-133-10L

PREPARED FOR:

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6200 CENTER STREET
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May 5, 2016



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ATTENTION: Mr. Kurt Gardner (kgardner@herwit.com)

**SUBJECT: Geotechnical Investigation
 Manteca Water Quality Control Facility Improvements
 Manteca, California**

Dear Mr. Gardner:

BSK Associates (BSK) is pleased to submit our geotechnical engineering investigation report for the proposed Manteca Water Quality Control Facility Improvements project in Manteca, California. The enclosed report describes the geotechnical investigation performed and presents our geotechnical recommendations for the design of the planned structures and earthwork for the project.

In summary, it is our opinion that the site is suitable for the proposed construction provided that the geotechnical recommendations presented herein are followed for design and construction of the project. Due to the presence of shallow groundwater and predominantly loose to medium dense sand layers in the upper 20 feet of the site, we consider the potential for the site to experience significant liquefaction-induced settlements during a design-level earthquake to be high. Therefore, we recommend that the planned structures either be designed to handle the estimated maximum amount of liquefaction-induced settlement or that ground improvement be performed underneath the structures prior to their construction. We expect that excavations for the project will need to be properly dewatered, sloped, and/or shored due to shallow groundwater and sandy soils. The planned structures can be supported on mat foundations and spread footings provided they are either designed to handle liquefaction induced-settlement or ground improvement is performed underneath them. Information on our investigative methods, conclusions, and specific recommendations for the design of the planned infrastructure and earthwork are contained in this report. Our report also discusses geologic hazards that could affect the site during a design-level seismic event.

The BSK logo, consisting of the letters "BSK" in a bold, blue, serif font.

The conclusions and recommendations presented in the enclosed report are based on limited subsurface investigation and laboratory testing programs. Consequently, variations between anticipated and actual subsurface soil conditions may be found in localized areas during construction. If significant variation in the subsurface conditions is encountered during construction, BSK should review the recommendations presented herein and provide supplemental recommendations, if necessary.

Additionally, design plans should be reviewed by our office prior to their issuance for conformance with the general intent of our recommendations presented in the enclosed report.

We appreciate the opportunity of providing our services to you on this project and trust this report meets your needs at this time. If you have any questions concerning the information presented, please contact us at (925) 315-3151.

Sincerely,

BSK Associates, Inc.



Cristiano Melo, PE, GE #2756
Geotechnical Group Manager



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Table of Contents

1.	INTRODUCTION	1
1.1	Project Description	1
1.2	Purpose and Scope of Services	2
2.	SITE INVESTIGATION.....	3
2.1	Previous Subsurface Investigations.....	3
2.2	Current Subsurface Investigation	3
2.2	Laboratory Testing.....	4
3.	SITE CONDITIONS	6
3.1	Site Description	6
3.2	Area Geology	6
3.3	Subsurface Conditions.....	7
4.	DISCUSSIONS AND CONCLUSIONS.....	8
4.1	Geologic and Seismic Hazards	8
4.1.1	<i>Seismic Shaking and Faulting</i>	8
4.1.2	<i>Expansive Soils</i>	8
4.1.3	<i>Liquefaction</i>	8
4.1.4	<i>Lateral Spread</i>	10
4.1.5	<i>Dynamic Compaction/Seismic Settlement</i>	10
4.2	Ground Improvement.....	11
4.2.1	<i>Specification and Design</i>	11
4.2.2	<i>Construction Considerations</i>	12
4.3	Foundation and Slab Support.....	13
4.4	Excavations	15
4.5	Temporary Dewatering	15
4.6	Existing Fill	15
4.7	Re-Use of Onsite Soil	16
5.	RECOMMENDATIONS	17
5.1	Foundations.....	17
5.1.1	<i>Mat Foundations</i>	17
5.1.2	<i>Spread Footings</i>	18
5.1.3	<i>Resistance to Lateral Loads</i>	19

5.1.4	<i>Proximity to Below Grade Structures and Underground Utilities</i>	20
5.1.5	<i>Uplift Loading Due to Buoyancy</i>	20
5.2	Below Grade Walls	20
5.3	2013 CBC Seismic Design Parameters	21
5.4	Slabs-on-Grade	22
5.4.1	<i>Concrete Floor Slabs</i>	23
5.4.2	<i>Exterior Flatwork</i>	24
5.5	Earthwork	25
5.5.1	<i>Existing Utilities</i>	25
5.5.2	<i>Site Preparation, Grading, and Compaction</i>	26
5.5.3	<i>Re-Use of Onsite Soil and Imported Fill Material</i>	28
5.5.4	<i>Weather/Moisture Considerations</i>	29
5.5.5	<i>Excavation, Shoring, and Backfill</i>	29
5.5.6	<i>Temporary Dewatering</i>	31
5.5.7	<i>Below-Grade Detention Ponds</i>	32
5.6	Asphalt Concrete Pavements	33
5.7	Storm Water Infiltration	34
5.8	Corrosion	35
5.9	Plan Review and Construction Observation	35
6.	ADDITIONAL SERVICES AND LIMITATIONS	37
6.1	Additional Services	37
6.2	Limitations	37

Plates and Appendices

PLATES

Plate 1 – Vicinity Map

Plate 2 – Site Plan

APPENDIX A – Boring Logs

Exhibit A-1 – Unified Soil Classification System (ASTM D2487)

Exhibit A-2 – Soil Description Key

Exhibit A-3 – Log Key

Log of Borings B-1 through B-4

APPENDIX B – Laboratory Test Results

Plate B-1 – Atterberg Limits (1 plate)

Direct Shear Test (4 plates)

Unconsolidated-Undrained Triaxial Test (1 plate)

Grain Size Analysis of Soils (5 plates)

Consolidation Test (1 plate)

R-value Test Report (1 plate)

Corrosivity Analysis by CERCO Analytical (2 plates)

APPENDIX C – Exhibit 1 – Summary of Compaction Recommendations

APPENDIX D – Subsurface Data from Previous Investigations

1. INTRODUCTION

This report presents the results of our geotechnical engineering investigation for the Manteca Water Quality Control Facility Improvements project in Manteca, California, hereafter referred to as the Manteca WQCF project. A Vicinity Map showing the location of the project site is presented on Plate 1. Our investigation has been performed for and coordinated with HERWIT Engineering.

This report was originally issued as a first draft on September 17, 2015 and has now been revised to incorporate comments by the design team and the City of Manteca. This report contains a description of our site investigation methods and findings, including field and laboratory data. Based on these findings, this report presents conclusions regarding the geotechnical concerns of the planned improvements. It also provides recommendations for the design of the planned structures and construction considerations. Note that the conclusions and recommendations presented in this final report supersede those presented in our previous draft reports.

1.1 Project Description

This project consists of new improvements within the northeast portion of the City of Manteca WQCF as shown on the Site Plan, Plate 2. The project will include the construction of two new digesters, a digester control building, underground pipelines, new paved driveways, and demolition and filling of existing asphalt covered drying beds. The new control building is expected to have a footprint area of approximately 7,000 square feet, be 1-story high, have CMU block walls, and be supported on a shallow foundation system consisting of a continuous perimeter footing and possibly interior isolated footings. Column loads (dead plus live) for isolated interior footings (if applicable) are expected to be about 90 to 100 kips. This building is expected to have a slab-on-grade floor and will sit between the two new digesters. We understand a pipe trench will be installed along the inside of the perimeter of this building. However, the perimeter footing will be deepened below the pipe trench to avoid surcharging it. The new digesters are expected to be about 65 feet in diameter, to have tapered bottoms ranging from 4 feet to 12 feet in depth below the ground surface, and to be supported on a tapered mat foundation. Dead plus live loads for the digester control building and the digesters are anticipated to be about 3,800 pounds per lineal foot and 2,300 pounds per square foot (psf), respectively.

Other improvements will include a flare tower and FOG (Fats, Oil, and Grease), H₂S, siloxane, and food processing facilities. These facilities are expected to be supported on mat foundations

and will be located to the north of the digester control building. New asphalt paved driveways will surround the new structures. In addition, a chemical pump station with two chemical tanks will be constructed immediately east of the existing digester No. 2. This facility will also be supported on a mat foundation.

As shown on Plate 2, four pads will be constructed at the site. Pad 1 will encompass the project site for the improvements discussed above, Pad 2 will house a compressed natural gas (CNG) fill station, Pad 3 will house a future expansion of the parking area at the CNG fill station, and Pad 4 will encompass future parking and buildings for solid waste.

To generate fill for the above pads, four soil borrow areas (labeled A through D on Plate 2) are planned. We understand below-grade detention ponds will be constructed at some of these borrow areas. Several existing stockpiles of soil are located near the southeast corner of Borrow Area D (see Plate 2). We understand the soil contained in these stockpiles was generated from previous excavations at the site and that the stockpiles have vegetation growing on them.

Although grading plans are not currently available for the project, we anticipate the project will require fills up to about 3 feet in thickness to backfill the drying beds and raise the site to finished design grade elevations. Excavations up to approximately 15 feet deep are expected to construct the new digesters, install new underground pipelines, and excavate shallow footings for the digester control building. We expect the below-grade detention ponds to be up to 5 feet deep.

If the actual project differs significantly from that described above, specifically if the grading differs from that we assumed above, we should be contacted to review and/or revise our conclusions and recommendations presented in this report.

1.2 Purpose and Scope of Services

The purpose of this investigation was to explore and evaluate the subsurface conditions at the site in order to provide geotechnical input for the design and construction of the planned improvements and the associated earthwork for this project. The scope of services, as outlined in our April 10, 2015 proposal (File Number: GL15-11606), consisted of subsurface investigation, laboratory testing, engineering analysis, and preparation of this report.

This investigation specifically excludes the assessment of site environmental characteristics, particularly those involving hazardous substances. Our scope of services did not include evaluation of contaminants in the soil, groundwater, or air.

2. SITE INVESTIGATION

2.1 Previous Subsurface Investigations

Previous investigations were performed near the project site in 2005 and 2014 by Kleinfelder and Neil O. Anderson Associates. These investigations were presented in the following documents:

- Neil O. Anderson Associates (2014), Preliminary Geotechnical Investigation, Proposed Solar PV Arrays, City of Manteca Wastewater Quality Control Facility, 2450 W. Yosemite Avenue, Manteca California, dated December 29, 2014 (File No. LGE140037); and
- Kleinfelder (2005), Geotechnical Services Report, Schedule D Project, Manteca Wastewater Treatment Plant, Manteca, California, dated May 12, 2005 (File No. 50876.G01).

Site plans from these reports showing the location of the borings drilled near the site are included in Appendix D along with their subsurface data and laboratory test results. The previous boring locations are also shown on our Site Plan, Plate 2. A discussion of the subsurface conditions encountered at the site is presented in the “Subsurface Conditions” section of this report and takes into consideration the subsurface data contained in the previous reports listed above with regards to the proposed borrow areas.

2.2 Current Subsurface Investigation

A geotechnical subsurface investigation was performed on July 28, 29, and 30, 2015 to evaluate the subsurface conditions at the site for the planned construction. This exploration consisted of drilling 4 borings (labeled B-1 through B-4) at the approximate locations shown on Plate 2. The borings were drilled using a truck-mounted drill-rig to depths of approximately 21½ to 51½ feet below the existing ground surface. Hollow-stem augers were used to drill the shallower borings (B-3 and B-4), while rotary wash drilling was used to performed the deeper borings (B-1 and B-2). Exploration GeoServices of San Jose, California was subcontracted to provide drilling services. The borings were logged by a BSK field representative.

Prior to the subsurface investigation, we walked the site with a WQCF representative familiar with the site’s underground utility lines. We selected our boring locations based on his input and the proximity to the planned structures. In addition, Underground Service Alert (USA) was contacted to provide utility clearance prior to drilling the site. A drilling permit was also obtained from the San Joaquin County Environmental Health Department (County). Upon

completion of the subsurface investigation, the borings were backfilled with cement grout per the County permit requirements. Excess cuttings generated during drilling were left in unimproved areas of the site near the boring locations.

The locations of the borings were estimated by our field representative based on rough measurements from existing features at the site. Elevations shown on the boring logs were based on the United States Geological Survey (USGS) 7.5-Minute Series Topographic Map for the 1996 Lathrop Quadrangle. As such the elevations and locations of the borings should be considered approximate to the degree implied by the methods used.

Relatively undisturbed samples of the subsurface materials were obtained using a split barrel sampler with a 2.5-inch inside diameter (I.D.) and a 3-inch outside diameter (O.D.) fitted with stainless steel liners. In addition, a 1.4-inch I.D. Standard Penetration Test (SPT) sampler was driven at selected depths in general accordance with ASTM D1586 test procedures. The samplers were driven 18 inches using a 140-pound, semi-automatic trip hammer falling 30 inches. Blow counts for successive 6-inch penetration intervals were recorded on the boring logs. After the samplers were withdrawn from the boreholes, the samples were removed, sealed to reduce moisture loss, labeled, and returned to our laboratory. Prior to sealing the samples, strength characteristics of the cohesive soil samples recovered were evaluated using a hand-held pocket penetrometer. The results of these tests are shown adjacent to the sample locations on the boring logs. In addition, bulk samples of the near-surface soils were collected for Resistance (R)-Value testing.

Soil classifications made in the field from auger cuttings and samples were re-evaluated in the laboratory after further examination and testing. The soils were classified in the field in general accordance with the Unified Soil Classification System (Visual/Manual Procedure - ASTM D2488). Where laboratory tests were performed, the designations reflect the laboratory test results in general accordance with ASTM D2487 as presented on Exhibit A-1. A Soil Description Key is presented on Exhibit A-2 and a key to the symbols used in the boring logs is presented on the Log Key, Exhibit A-3. Sample classifications, blow counts recorded during sampling, and other related information were recorded on the soil boring logs. Logs of borings B-1 through B-4 are presented in Appendix A. A discussion of the subsurface conditions encountered at the site is presented in the "Subsurface Conditions" section of this report.

2.2 Laboratory Testing

Laboratory tests were performed on selected soil samples to evaluate their physical characteristics and engineering properties. The laboratory testing program included dry density and moisture content, Atterberg limits, sieve analysis, hydrometer analysis, direct shear,

unconsolidated-undrained triaxial compression (TXUU), consolidation, and R-Value testing. Some of the testing was performed by Cooper Testing Labs of Palo Alto, California. Most of the laboratory test results are presented on the individual boring logs. The results of the Atterberg limits, direct shear, TXUU, grain size analysis, hydrometer analysis, and R-Value tests are also presented graphically in Appendix B.

Analytical testing was performed as part of our investigation on soil samples obtained from depths of about 16 and 5 feet at borings B-1 and B-3, respectively, to assist in evaluating the corrosion potential of the on-site soils. The corrosivity testing was performed by CERCO Analytical of Concord, California using ASTM methods as described in CERCO Analytical's August 12, 2015 report. CERCO's letter and corrosion test results are presented in Appendix B.

3. SITE CONDITIONS

3.1 Site Description

The project site is situated within the City of Manteca's WQCF located at 2450 West Yosemite Avenue in Manteca, California. The project site for the planned improvements is currently occupied by asphalt paved driveways, asphalt covered drying beds surrounded by a gravel access road, and agricultural land. As shown on Plate 2, two existing digesters and a digester control building are located immediately south of the planned digesters and control building. The site is relatively level, but there appears to be an approximately 1- to 2-foot drop in ground surface elevation between the existing plant facilities and the surrounding cropland. According to a preliminary topographic plan provided by HERWIT Engineering dated December 2015 (Drawing No. C-10), site grades range from an approximate elevation of 21 feet (at the bottom of the existing asphalt covered drying beds) to 25 feet (top of the existing gravel access road surrounding the site) above Mean Sea Level. The agricultural land surrounding Pad 1 lies at an elevation of about 24 feet. A section of the French Camp Outlet Canal (FCOC) is located approximately 500 feet northwest of the project site. The canal is about 15 feet deep at the closest point to the project site and its banks have a gradient of approximately 2H:1V (horizontal to vertical) based on our field observations and rough measurements.

3.2 Area Geology

The site area is located in the Great Valley geomorphic province, just east of the San Joaquin River, which forms a broad syncline with deposits of marine and overlying continental sediments, Jurassic to Holocene in age. The thickness of the sediments increases to the west and reaches a thickness of as much as 20,000 feet on the west side of the San Joaquin Valley syncline. East of the site area are the Sierra Nevada Mountains consisting of Mesozoic folded metamorphic rocks and Mesozoic plutonic rocks. To the west are the Coastal Ranges characterized by north-south trending ridges and valleys that are typically highly folded with numerous faults.

The site area is situated on the Pleistocene Modesto Formation (Wagner, 1991)¹, which consists primarily of sand and gravel in the fan areas while clay, silt, and sand are dominant in the inter-fan areas. The formation thickness ranges from a thin layer on the east side of the valley to approximately 150 feet thick in the central part of the basin (CDWR, 2003)². Based on our

¹ Wagner, D.L., Bortugno, E.J., and McJunkin R.D. (1991), Geologic Map of the San Francisco - San Jose Quadrangle, California Geological Survey, Regional Geologic Map No. 5A.

² California Department of Water Resources (CDWR, 2003), California's Groundwater, Bulletin 118.

observations during our subsurface investigation, the site surficial materials generally consist of sand, silty sand, and sandy silt.

3.3 Subsurface Conditions

Based on our (current) borings, the upper approximately 20 feet of the subsurface consists predominantly of loose to medium dense sand. Laboratory sieve analyses indicate that the sands grade from silty, to poorly graded with silt, to poorly graded. Below a depth of about 20 feet below the ground surface (bgs), the sands are dense to very dense and there is an approximately 5 to 10 foot thick firm clay layer at a depth of about 25 feet bgs. Clay was also encountered at a depth of about 40 feet bgs extending to the maximum depth of our exploration (about 51½ feet). These subsurface conditions appear to be consistent with the findings from previous geotechnical investigations performed proximate to the site.

Groundwater was encountered in our hollow-stem auger borings (B-3 and B-4) at a depth of about 12 feet below the ground surface. The 2014 investigation by Neal Anderson encountered water at depths of 8 to 10 feet and the 2005 investigation by Kleinfelder encountered water at depths of 10 to 18 feet. However, according to the California Department of Water Resources (CDWR) website (<http://www.water.ca.gov/waterdatalibrary>), a nearby groundwater monitoring well (referenced as Station 377971N1212517W001) located approximately a half a mile to the northeast of the proposed digester control building has measured groundwater as shallow as about 6 feet below the ground surface in the past. According to the CDWR website, this depth corresponds to an elevation of approximately 19 feet³. In addition, we understand the City of Manteca has historical groundwater elevation records from several on-site wells and that the historical highwater level at the facility has ranged from elevations of approximately 17 to 20 feet (based on NGVD29 datum) at wells NW11 and NW9W. It should be noted that groundwater levels can fluctuate depending on factors such as seasonal rainfall, groundwater withdrawal, and construction activities on this or adjacent properties.

The above is a general description of soil and groundwater conditions encountered at the site in our borings. For a more detailed description of the soils encountered, refer to the boring log data in Appendix A.

It should be noted that subsurface conditions can deviate from those conditions encountered at the boring locations. If significant variation in the subsurface conditions is encountered during construction, it may be necessary for BSK to review the recommendations presented herein and recommend adjustments as necessary.

³ Vertical datum is NGVD29.

4. DISCUSSIONS AND CONCLUSIONS

Based on the results of our subsurface investigation, it is our opinion that the planned improvements are feasible geotechnically and that the site may be developed as presently planned. This conclusion is based on the assumption that the recommendations presented in this report will be incorporated into the design and construction of this project.

Additional discussions of the conclusions drawn from our investigation, including general recommendations, are presented below. Specific recommendations regarding geotechnical design and construction aspects for the project are presented in the “Recommendations” section of this report.

4.1 Geologic and Seismic Hazards

4.1.1 *Seismic Shaking and Faulting*

We expect the site to be subjected to strong ground shaking during the life of the project. Therefore, the seismic design parameters presented in the “2013 CBC Seismic Design Parameters” section of this report should be incorporated into the design of the planned structures.

The site is not located within an Alquist-Priolo Earthquake Fault Zone and no mapped fault traces are known to transverse the site. Therefore, we conclude that the potential for surface fault rupture to occur across the site is low.

4.1.2 *Expansive Soils*

The surficial soils are composed predominantly of fine grained silty sand to poorly graded sand with silt with very low to low expansion potential.

4.1.3 *Liquefaction*

Soil liquefaction is a condition where saturated, granular soils undergo a substantial loss of strength and deformation due to pore pressure increase resulting from cyclic stress application induced by earthquakes. In the process, the soil acquires 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, and fine-grained sand deposits. If liquefaction occurs, foundations and improvements resting above or within the liquefiable layer may undergo settlements and/or a loss of bearing capacity.

Based on our findings, the site has a high susceptibility to experience liquefaction-induced settlements during a design-level earthquake. We performed liquefaction analyses for our 50-foot deep rotary wash borings and total liquefaction-induced settlements are estimated to be on the order of about 2½ to 6½ inches. The potentially liquefiable soils extend from a design groundwater depth of 6 feet to a depth of about 20 feet bgs.

Our liquefaction analyses were based on the methods by Youd et al (2001)⁴, Seed et al. (2003)⁵, and Idriss and Boulanger (2004)⁶ using the following input parameters:

- A design groundwater depth of 6 feet.
- A PGA_M of 0.427g (refer to the “2013 CBC Seismic Design Parameters” section later in this report).
- An earthquake magnitude of M6.6 based on the USGS Interactive Deaggregation website (USGS, 2008)⁷.

It should be noted that the method by Seed et al. (2003) identified the clay layer extending from a depth of approximately 25 to 31 feet bgs at boring B-2 as potentially liquefiable. Because only one of the three methods used indicates this clay layer is susceptible to liquefaction and because we have accounted for consolidation settlement in this layer in our total settlement estimates, we are not adding the calculated liquefaction-induced settlement from this clay layer to the total settlement estimates.

Note also that the amount of total liquefaction-induced settlement estimated by the Seed et al. (2003) method does not exceed the maximum value of 6½ inches discussed above if the clay layer at a depth of 25 feet bgs at boring B-2 is included as potentially liquefiable layer.

⁴ Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Liam Finn, W.D.L., Harder, L.F., Jr., Hynes, M.E., Ishihara, K., Koester, J.P., Liao, S.S.C., Marcuson, W.F., III, Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R.B., Stokoe, K.H., II (2001), Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, ASCE, Journal of Geotechnical and Geoenvironmental Engineering, V. 127, No. 10, p 817-833.

⁵ Seed, R.B., K, O., Cetin, R.E.S., Moss, A., Kammerer, J., Wu, J.M., Pestana, M.F., Riemer, R.B., Sancio, J.D., Bray, R.E., Kayen, R.E., Faris, A. (2003), "Recent Advances in Soil Liquefaction Engineering: a unified and consistent framework," Keynote Address, 26th Annual Geotechnical Spring Seminar, Los Angeles Section of the Geoinstitute, American Society of Civil Engineers, H.M.S. Queen Mary, Long Beach, California, USA.

⁶ Idriss, I.M. and Boulanger, R.W. (2004), "Semi-empirical procedures for evaluating liquefaction potential during earthquakes," in Proceedings, 11th International Conference on Soil Dynamics and Earthquake Geotechnical Engineering, and 3rd International Conference on Earthquake Geotechnical Engineering, D. Doolin et al., eds., Stallion Press, Vol. 1, pp. 32-56.

⁷ USGS (2008), 2008 Interactive Deaggregations, <http://geohazards.usgs.gov/deaggint/2008/>

Based on our review of Youd and Garris (1995)⁸ and interpretation of the subsurface conditions encountered in our borings, we conclude that the potential to be high for ground surface disruption (such as sand boils, ground fissures, etc.) to occur at the site if it were to experience liquefaction.

4.1.4 *Lateral Spread*

Lateral spread is a potential hazard commonly associated with liquefaction where extensional ground cracking and settlement occur as a response to lateral migration of subsurface liquefiable material. These phenomena typically occur adjacent to free faces such as slopes, creek channels, and levees. Because most of the site is relatively flat, we conclude that the potential for lateral spread to occur within the project limits is low. Nonetheless, we ran lateral spread analyses for the portion of the FCOC closest to the project using the methodology provided in Youd et al (2002)⁹ and the subsurface conditions encountered in boring B-2. Based on our analyses, we conclude that the potential for lateral spread to adversely impact the site is low.

4.1.5 *Dynamic Compaction/Seismic Settlement*

Another type of seismically induced ground failure, which 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 order to evaluate the potential impact seismic settlement could have on the planned surficial structures, we performed seismic settlement analysis on the surficial loose to medium dense layers encountered in the upper 6 feet in our borings using the method developed by Tokimatsu and Seed (1987)¹⁰. For our analyses, we used an earthquake magnitude of M6.6 and a PGA of 0.427g. Based on our analysis, we conclude that the magnitude of potential seismic settlement underneath the planned structures should be negligible.

⁸ Youd, T.L., and Garris, C.T. (1995), Liquefaction-Induced Ground Surface Disruption, Journal of Geotechnical Engineering, ASCE, Vol. 121, No. 11, in press.

⁹ Youd, TL, Hansen, CM, and Bartlett, SF (2002), Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement, Journal of Geotechnical and Geoenvironmental Engineering, No. 128, Vol. 12, pp. 1007-1017.

¹⁰ Tokimatsu, K. and Seed, H.B. (1987), Evaluation of Settlements in Sands Due to Earthquake Shaking, Journal of Geotechnical Engineering, ASCE, Vol. 113, No. 8, August, pp. 861-878.

4.2 Ground Improvement

We understand that ground improvement will be utilized to mitigate the liquefaction potential below the planned structures. Based on the subsurface conditions encountered at the site, we believe one of the following ground improvement methods could be used effectively at the site:

- Stone Columns (or similar methods, such as Impact Rammed Aggregate Piers, IRAP)
- Compaction Grouting

These ground improvement methods could be used to eliminate or reduce the amount of liquefaction-induced settlement estimated for the site. The potential for ground surface disruption to occur within areas of the site that are mitigated using one of these methods would be low.

Although initial consideration was given to vibrocompaction¹¹ as a potential ground improvement alternative for this site, the potentially liquefiable layers appear to have a clay content greater than 2 percent based on our hydrometer testing. This clay content typically negates the effectiveness of this ground improvement method. Therefore, vibrocompaction is not considered a viable ground improvement alternative for this site.

At this time, we anticipate that the zone requiring ground improvement would extend from a depth of about 6 feet to 20 feet below the ground surface underneath the new structures and a lateral distance from the structures ranging from one-half to one times the depth of the ground improvement as measured from the ground surface (i.e., 10 to 20 feet).

4.2.1 Specification and Design

From a ground improvement standpoint, the purpose of this geotechnical report is to help characterize the subsurface conditions present at the site. BSK will issue a separate document presenting performance-type requirements for the ground improvement. We understand performance-type specifications will then be prepared by the design team for one of these two ground improvement methods in consultation with BSK. The specifications will stipulate the desired outcome of the ground improvement and how it should be monitored and validated during construction. Although these specifications will also provide minimum criteria for implementation of the ground improvement methods, after the ground improvement contractor is selected via a bidding process, the contractor will be responsible for designing the various aspects of the ground improvement method to be used and for meeting the

¹¹ Also known as vibroflotation.

performance criteria. The contractor will also be responsible for making adjustments to its ground improvement layout during construction in order to meet the performance criteria.

4.2.2 Construction Considerations

The two ground improvement methods being considered would result in the creation of hard points underneath the digester foundations where the stone columns or the grout columns are installed. Therefore, we recommend overexcavating the soil below the digester foundations to a depth of 18 inches below the bottom of the foundations. The overexcavated zone should be backfilled with Caltrans Class 2 permeable material (refer to Section 68 of the 2010 Caltrans Standard Specifications). This material should extend laterally a minimum of 1 foot beyond the outside edge of the digesters mat foundation.

Prior to ground improvement operations, existing underground utilities within the areas to undergo ground improvement (i.e., treatment areas) should be identified and relocated outside the influence zone of the areas to be treated. The influence zone is expected to extend about 5 to 10 feet laterally from the outer edge of the treatment areas. Monitoring of adjacent structures and surface deformation within this influence zone should be performed during the ground improvement process. The ground improvement process could have the potential for affecting existing structures such as foundations and slabs within this influence zone. Underground utility pipes located within this influence zone could be displaced or filled with grout (if used). If surface manifestations such as cracks or heave are observed during the ground improvement process, the operation should be halted so that BSK can evaluate site conditions and recommend appropriate modifications.

If underground utility lines are located outside the areas to be treated, but within the treatment influence zone, and they cannot be relocated, they could be protected by excavating a temporary trench between the treatment areas and the utility line. The trench should extend at least one foot below the pertinent utility line. If the trench walls cannot stand vertically while the ground improvement is taking place, then the trench walls could either be sloped or the trench could be filled with a slurry-bentonite mix or an equivalent compressible material that can later be pumped out during backfill of the trench. The bottom of such trenches should not be located below an imaginary plane projected 1H:1V from the edge of nearby foundations or shoring/underpinning may be required.

Note that unmitigated areas of the site could still experience the maximum liquefaction-induced settlement previously discussed. This settlement would be differential when compared to areas of the site to undergo ground improvement. Therefore, flexible joints should be installed along the transition zone of underground pipelines where they cross between areas of

the site that undergo ground improvement mitigation and unmitigated areas of the site. Depending on how much vertical offset these joints can handle, multiple joints installed in series may be required.

4.3 Foundation and Slab Support

Due to the presence of potentially liquefiable conditions at the site, the planned structures should either be supported on deep foundations that extend below the liquefiable layers, or the liquefiable zone beneath the structures should be mitigated by ground improvement. Of these alternatives, we understand that ground improvement is the desired option as discussed in the “Ground Improvement” section above.

The planned structures can be supported on mat foundations and shallow footings provided the liquefiable zone beneath them is properly mitigated by ground improvement. We estimate that elastic settlement underneath these foundations would be limited to less than 1 inch of total settlement and less than ½ inch of differential settlement over a horizontal distance of 30 feet. Note that this differential settlement should not be extrapolated beyond this distance. Most of the elastic settlements are expected to occur during or shortly after construction of the structures as loads are applied.

According to our consolidation test results and engineering analyses, we estimate about 1 and ½ inch of total long-term consolidation settlement could occur underneath the digesters and other structures supported on shallow mat foundations, respectively. Consolidation settlement is not expected be significant underneath the digester control building (i.e., less than ¼ inch).

Note that the depth of the clay layer with a potential to undergo consolidation settlement is located below the expected ground improvement zone discussed in the “Ground Improvement” section above. Therefore, structures would still be susceptible to long-term consolidation settlement after ground improvement takes place below them. Also, structures founded on areas of the site that do not undergo ground improvement would still be susceptible to the 2½ to 6½ inches of liquefaction-induced settlement during a design-level earthquake as previously discussed in the “Liquefaction” section of this report.

The table below summarizes the estimated total and differential settlements associated with liquefaction and long-term consolidation for the planned foundations for this project.

SUMMARY OF LIQUEFACTION-INDUCED AND LONG-TERM CONSOLIDATION SETTLEMENTS							
Foundation Type	Allowable Bearing Pressure ¹ (psf)	Liquefaction-induced Settlement (inches)		Long-Term Consolidation Settlement ² (inches)		Combined Settlement (inches)	
		Total	Differential	Total	Differential	Total	Differential
Shallow Mat Foundations with no ground improvement	1,500	2½ to 6½	Half of total over a horizontal distance equal to half the mat's length	Up to about ½	Up to about ¼ over a horizontal distance equal to half the mat's length	3 to 7	Up to about 1½ to 3½ over a horizontal distance equal to half the mat's length
Digester Control Building Footings	1,500 to 3,000	Negligible if ground improvement is properly performed		Less than ¼	Half of total over a horizontal distance of 30 feet	Less than ¼	Half of total over a horizontal distance of 30 feet
Digester Mats	2,500	Negligible if ground improvement is properly performed		Up to about ¾	Up to about ¼ over a horizontal distance equal to half the mat's diameter	Up to about ¾	Up to about ¼ over a horizontal distance equal to half the mat's diameter
	3,000			Up to about 1	Up to about ½ over a horizontal distance equal to half the mat's diameter	Up to about 1	Up to about ½ over a horizontal distance equal to half the mat's diameter
Notes: 1. Refer to the "Foundations" section of this report for allowable bearing pressure recommendations. 2. Maximum long-term consolidation settlement occurs near the center of the mats and tapers to the minimum near the edges, thus the differential settlement over half the length of the mat.							

We understand that the City of Manteca may elect not to perform ground improvement underneath the new structures to be located north of the digester control building. If these mat-supported structures were to be damaged due to liquefaction-induced settlements during a seismic event, their foundations may need to be repaired. Possible repair alternatives would include slab jacking (a process of injecting grout under pressure to raise and relevel slabs) or applying a self-leveling compound.

The digester building slabs can be supported on grade due to the low expansion potential of the near-surface soils. However, in order to provide enhanced subgrade support, we recommend supporting interior floor slabs with 6 inches of compacted crushed rock or Caltrans Class 2 aggregate baserock (refer to Section 5.4.1 of this report for details). Exterior concrete flatwork may be supported directly on moisture conditioned and compacted native sand subgrade soils. The subgrade soils for interior slabs and exterior flatwork should be properly moisture conditioned prior to the placement of concrete.

4.4 Excavations

We anticipate that excavations at the site can be made with standard earthwork equipment, such as excavators, dozers, backhoes, and trenchers. Because the site is underlain primarily by sandy soils within the anticipated excavation depths, shoring or sloping of cut faces and trench walls will likely be necessary to protect personnel and to provide temporary stability. OSHA guidelines should be followed for excavations performed at the site.

4.5 Temporary Dewatering

As previously discussed, groundwater could be as shallow as 6 feet below the ground surface at the site. Therefore, we anticipate that excavations extending below this depth will need to be continuously dewatered during construction.

4.6 Existing Fill

The soil encountered in the upper approximately 1 to 3 feet at borings B-1, B-3, and B-4 appears to consist of fill. The fill material is very similar to the nearby onsite surficial soils and visually looks to be relatively free of debris and organic matter. Therefore, we take no exception to its use as general fill material during grading for the project.

4.7 Re-Use of Onsite Soil

Provided it is free of vegetation, organics, debris, deleterious matter, and oversize material, the upper 10 feet below existing grade of the onsite soil, including Borrow Areas A through D (which include surrounding agricultural land), may be used as general fill and backfill during grading for the project. Stripping requirements for these areas are discussed in the “Earthwork” section of this report. A BSK representative should be present onsite during grading to visually confirm that the soil removed from these areas are consistent with our findings. Where excavations extend deeper than 10 feet, such as at the digesters, the material below a depth of 10 feet should not be used as general fill underneath the planned paved driveways/parking lots, mat foundations, the digester control building, exterior concrete flatwork, and underground utility trenches crossing these improvements. Such material should be segregated and placed as general fill or backfill outside these areas. The reason for this is that excavations extending deeper than 10 feet could encounter soil layers that are composed primarily of silt and clay, which typically have a much lower R-Value and higher soil expansion than the predominantly sand soils present at the ground surface. Further discussion on the re-use of onsite soil as fill is presented in the “Earthwork” section of this report.

Soil taken from borrow areas located in previous agricultural areas may experience up to approximately 20 percent loss in volume or higher during placement as compacted fill as a result of densification; especially soil taken from the upper 5 feet below the existing ground surface.

Stripped topsoil from vegetated areas and soil from the existing stockpiles shown on Plate 2 may only be stockpiled for later use in landscaping areas or to line the bottom of the proposed detention ponds.

5. RECOMMENDATIONS

Presented below are recommendations for the design of the planned structures, seismic considerations, earthwork, and construction considerations for this project.

5.1 Foundations

5.1.1 Mat Foundations

Most of the planned structures at the site are anticipated to be supported on mat foundations. The mats should have a minimum depth at the edges of 9 inches if they are wider than 15 feet. Mats having a width less than 15 feet should have a minimum depth at the edges of 12 inches. An allowable bearing pressure of 1,500 psf may be used for dead and long term live loads for shallow mats where no ground improvement is performed, while a value of 2,500 to 3,000 psf may be used for the digester mats. The allowable bearing pressure value may be increased by 1/3 for short term seismic and wind loads. Bearing capacity values include a factor of safety of at least 2. We recommend that the digester mats be underlain by an 18-inch thick section of Caltrans Class 2 permeable material (refer to Section 68 of the 2010 Caltrans Standard Specifications) to serve as cushion for hard points created by the ground improvement and as a pore pressure relief zone. This material should extend laterally a minimum of 1 foot beyond the outside edge of the digester mat foundation. The allowable bearing values provided above were estimated assuming that mats uniformly bear on re-compacted native soils or compacted engineered fill.

We understand portions of the planned shallow mat foundations in unmitigated areas may be exposed to equipment loading and/or occasional vehicle and truck loading. The mats in these areas should be underlain by a minimum of 4 inches of Caltrans Class 2 aggregate baserock (refer to Section 26 of the 2010 Caltrans Standard Specifications).

Due to the presence of shallow groundwater at the site, a modulus of subgrade reaction, K_{V1} , of 60 pounds per square inch per inch (pci) of deflection (based on published data for a one square foot bearing plate) is considered applicable for mat foundations where no ground improvement is performed, while a value of 100 pci may be used for the digester mats (i.e., ground improvement to be performed). These modulus values are typically reduced for mat slab sizes larger than 1 square foot. For various slab sizes, the subgrade modulus may be calculated using the following formulas:

Square:
$$K_S = (K_{V1}) \times \left(\frac{1 \text{ foot}}{B} \right)$$

Rectangular:
$$K_R = (K_{V1}) \times \left(\frac{1 \text{ foot}}{B}\right) \times \left(\frac{m+0.5}{1.5 \times m}\right)$$

Where:

- K_{V1} is the modulus of subgrade reaction for a 1 square foot plate (in units of pci);
- B is the width of the foundation/slab (in units of feet);
- m is the ratio of the foundation/slab length divided by its width; and
- K_S and K_R are the adjusted modulus of subgrade reaction based on the actual dimensions of the foundation/slab (in units of pci).

If a computer program is used to design the project foundations and it requires the input of a modulus of subgrade reaction for the site, the designer should check whether the program requires input of the unadjusted or adjusted modulus of subgrade reaction.

5.1.2 Spread Footings

The planned digester building is anticipated to be supported by a combination of continuous and isolated spread footings. The table below presents our footing recommendations. The allowable bearing pressure values may be increased by 1/3 for short term seismic and wind loads. Due to the loose sandy soil conditions near the surface, we recommend underlying the digester building footings with Caltrans Class 2 aggregate baserock (refer to Section 26 of the 2010 Caltrans Standard Specifications) to provide a better bearing surface and a higher bearing capacity value. Allowable bearing values are provided below for footings uniformly bearing on 1- and 2-foot sections of properly compacted Caltrans Class 2 aggregate baserock. The bottom of the footing excavations need to be properly compacted in accordance to the “Earthwork” section of this report prior to placement of the aggregate baserock section. The aggregate baserock should be compacted in lifts no more than 8 inches thick. **If it is not desired to provide a section of aggregate baserock below the footings, an allowable bearing pressure of 1,500 psf should be used under exterior and interior footings.** The bottom of the footing excavation would still need to be properly compacted prior to the placement of rebar and concrete and the minimum embedment depth and minimum widths shown in the table below would still apply.

FOOTING BEARING CAPACITY RECOMMENDATIONS			
Footing Type	Allowable Bearing Pressure (psf)*	Minimum Embedment Depth (in)**	Minimum Width (in)
Exterior Continuous Footings***	2,000	18	12
Isolated Interior Footings***	2,000	18	18x18
Exterior Continuous Footings****	2,500	18	12
Isolated Interior Footings****	3,000	18	18x18
<p>* Pounds per square foot, dead plus live load. Includes factor of safety (FS) of at least 2. ** Below lowest adjacent grade defined as bottom of slab on the interior and finish grade at the exterior. *** For footings underlain by 12 inches of compacted Caltrans Class 2 aggregate baserock. **** For footings underlain by 24 inches of compacted Caltrans Class 2 aggregate baserock.</p>			

Concrete for footings should be placed neat against properly compacted soil or engineered backfill. It is important that footing excavations not be allowed to dry before placing concrete. Even then, because the subsurface soils at the site consist predominantly of sand, it is possible the excavation sidewalls could slough. Therefore, it may be necessary to form the footings before placing concrete. The footing excavations should be monitored by a BSK representative for compliance with appropriate moisture control and to confirm the adequacy of the bearing materials and proper compaction of footing bottoms. If areas containing soft or loose materials are present at the bottom of the footing excavations after they are compacted, they may need to be removed and replaced with lean concrete, sand-cement slurry, or compacted aggregate baserock under the observation of a BSK representative.

5.1.3 Resistance to Lateral 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. An allowable friction coefficient of 0.30 between the foundation and supporting subgrade may be used. For passive resistance, an allowable equivalent fluid pressure of 300 pounds per cubic foot (pcf) acting against the foundation side may be used above a design groundwater depth of 6 feet bgs. Below this depth, a passive pressure of 200 pcf should be used. The friction coefficient and passive resistance may be used concurrently, and can be increased by one-third for wind and/or seismic loading. We recommend that the upper foot of soil cover be neglected in the passive resistance calculations if the ground surface above is not confined by a slab, pavement or in some similar manner. These values include a factor of safety of about 1½.

5.1.4 Proximity to Below Grade Structures and Underground Utilities

Where footings and mat foundations are located adjacent to below grade structures or near underground utilities, the footings and mat foundations should extend below a 1H:1V imaginary plane projected upward from the bottom of the structure's foundation perimeter or bottom of the underground utility to avoid surcharging the below grade structure and underground utility with building loads. Where this is not feasible, the affected structure/pipeline should be designed to handle the surcharge load imposed by the building. Sometimes, it is also possible to use sand-cement slurry (1- or 2-sack mix) as trench backfill to help protect utility lines from surcharge loads. However, this alternative should be evaluated by the design team on a case-by-case basis prior to implementation during construction.

5.1.5 Uplift Loading Due to Buoyancy

The digesters should be designed to resist a buoyancy force based on a design groundwater depth of 6 feet below the existing ground surface (this is equivalent to a groundwater elevation of about 19 feet).

The weight of the digester (assume empty case) may be used to resist this uplift pressure as well as friction between the digester walls and the surrounding backfill. An allowable friction coefficient of 0.30 between the walls and surrounding backfill may be used. This value includes a factor of safety of about 1½.

We understand the mat foundation for the digesters will extend beyond the outer digester wall limits thus forming a "lip". If that is the case, the weight of the backfill above the lip plus a soil wedge extending upward at a 60-degree angle from the horizontal from the edge of the lip may also be used to resist uplift pressure. Effective soil unit weights of 120 and 58 pcf may be used above and below the design groundwater depth, respectively.

5.2 Below Grade Walls

Below grade walls should be designed to resist the lateral earth pressures exerted by the retained soil or compacted backfill plus additional lateral force that will be applied to the wall due to surface loads placed at or near the wall. An active earth pressure should be used where the walls are allowed to deflect and an at-rest pressure should be used for restrained walls. Fifty percent of uniform area surcharge placed at the top of a restrained wall may be assumed to act as a uniform horizontal pressure over the entire height of the wall. Thirty percent of any uniform surcharge placed at the top of an unrestrained wall may be assumed to act as a uniform horizontal pressure over the entire height of the wall. The active earth pressure

condition will develop only when the wall is allowed to yield sufficiently. The amount of outward displacement at the top of the wall designed for active earth pressures may be up to 0.004H to 0.04H, where H is the height of the wall. Below grade walls may be designed using the lateral earth pressures provided in the table below.

LATERAL EARTH PRESSURES		
Earth Pressures	Equivalent Fluid Pressures (pcf)	
	Above Water*	Below Water*
Active	35	85**
At-Rest	50	90**
Seismic Increment (Flexible)	10	5
Seismic Increment (Rigid)	25	12

* Design groundwater = 6 feet below existing ground surface

**Includes hydrostatic pressure

Section 1803.5.12 of the 2013 CBC requires that the design for foundation walls and retaining walls supporting backfill heights greater than 6 feet include seismic earth pressures. These pressures are expressed as equivalent fluid pressures and would be added to the wall design in addition to the static active or at-rest pressures. The seismic earth pressure should be applied as a triangular distribution with the resultant force acting 1/3 times the wall height above the base of the wall.

5.3 2013 CBC Seismic Design Parameters

As previously discussed, the project site is not mapped as part of the Alquist-Priolo Earthquake Fault Zoning Act. However, due to potential earthquake motion resulting from nearby faults, seismic design factors should be considered in the design of structures for the project. Based on the results of our analyses, the site subsurface soils are susceptible to liquefaction during a design-level earthquake. Therefore, according to Table 20.3-1 of ASCE 7-10, the site should be classified as Site Class F, which requires site-specific response analysis. However, Sections 11.4.7 and 20.3.1 of ASCE 7-10 state that for a short period (less than ½ second) structure on liquefiable soils, these factors may be based on the assessment of the site class assuming no liquefaction.

Provided the planned structures have fundamental periods of less than about ½ second or the potential liquefiable zone below the planned structures is properly mitigated via ground improvement, we recommend using Site Class D (stiff soil profile) for design of these structures and use of the 2013 CBC mapped seismic design criteria would be considered appropriate for this site. If this is the case, the seismic parameters presented in the table

below should be considered applicable for the design of structural improvements. Otherwise, we should be consulted to evaluate whether a site-specific response analysis is required for the project.

2013 CBC SEISMIC DESIGN PARAMETERS*			
Seismic Design Parameter	Value		Reference
Site Class	D		Table 20.3-1, ASCE 7-10
MCE _R Mapped Spectral Acceleration (g)	S _S = 1.044	S ₁ = 0.367	USGS Mapped Values based on Figures 1613.3.1(1) and 1613.3.1(2), 2013 CBC
Site Coefficients	F _a = 1.082	F _v = 1.666	Tables 1613.3.3(1) and 1613.3.3(2), 2013 CBC
MCE _R Mapped Spectral Acceleration Adjusted for Site Class Effects (g)	S _{MS} = 1.130	S _{M1} = 0.611	Section 1613.3.3, 2013 CBC
Design Spectral Acceleration (g)	S _{DS} = 0.754	S _{D1} = 0.408	Section 1613.3.4, 2013 CBC
Seismic Design Category	D		Section 1613.3.5, 2013 CBC
MCE _G peak ground acceleration adjusted for Site Class effects (g)	PGA _M = 0.427		Section 11.8.3, ASCE 7-10
Definitions: MCE _R = Risk-Targeted Maximum Considered Earthquake MCE _G = Maximum Considered Earthquake Geometric Mean <i>*These seismic design parameters are based on the assumption that the planned structures have fundamental periods of less than about ½ second or the potentially liquefiable zone below the planned structures is properly mitigated via ground improvement. If that is not the case, BSK should evaluate whether a site-specific response analysis is required.</i>			

As shown above, the short period design spectral response acceleration parameter, S_{DS}, is greater than 0.5 and the long period design spectral response acceleration parameter, S_{D1}, is greater than 0.2. These values characterize the site as Seismic Design Category D as specified in Section 1613.3.5 of the 2013 CBC. In accordance with Section 1613.3.5 of the 2013 CBC, each structure shall be assigned to the more severe seismic design category in accordance with Table 1613.3.5(1) or 1613.3.5(2), irrespective of the fundamental period of vibration of the structure.

5.4 Slabs-on-Grade

Slabs-on-grade for this project will consist of interior concrete floor slabs and exterior flatwork. As previously discussed, the near-surface soils at the site have a very low to low expansion

potential and the potential for these slabs to be subjected to shrink/swell cycles with fluctuations in moisture content is low.

5.4.1 Concrete Floor Slabs

We recommend underlying interior floor slabs with 6 inches of ¾-inch compacted crushed rock meeting the gradation criteria for Coarse Aggregate size number 6 specified under ASTM C33, latest edition. If the interior floor slabs are not covered by moisture sensitive flooring materials, 6 inches of Caltrans Class 2 aggregate baserock (refer to Section 26 of the 2010 Caltrans Standard Specifications) may be used in lieu of crushed rock. It is important that the crushed rock material be placed as soon as possible after moisture conditioning and compaction of the subgrade materials to reduce drying of the pad subgrade. A representative of BSK should be present to assess the subgrade condition and observe/test the preparation of the subgrade prior to slab construction.

Slab thickness and reinforcing should be designed by a Structural Engineer. As a minimum, we suggest the concrete floor slabs be at least 5 inches thick and properly reinforced. Special care should be taken to ensure that reinforcement is placed at the slab mid-height, particularly if using welded wire fabric. We prefer at least No. 3 reinforcing steel because it is easier to control the location, especially during concrete placement. The floor slabs should be separated from footings, structural walls, and utilities, and provisions should be made to allow for differential movements at these interfaces. If this is not possible from a structural or architectural design standpoint, it is recommended that the slab connection to footings be reinforced such that there will be resistance to potential differential movement.

Subsurface moisture and moisture vapor naturally migrate upward through the soil and, where the soil is covered by a building or pavement, this subsurface moisture will collect. To reduce the impact of the subsurface moisture and potential impact of future introduced moisture (such as landscape irrigation or precipitation) the current industry standard is to place a vapor retarder on the compacted crushed rock layer. This membrane typically consists of visqueen or polyvinyl plastic sheeting at least 15 mil in thickness. It should be noted that although vapor barrier systems are currently the industry standard, this system may not be completely effective in preventing floor slab moisture problems. These systems will not necessarily ensure that floor slab moisture transmission rates will meet floor-covering manufacturer standards and that indoor humidity levels be appropriate to inhibit mold growth. The design and construction of such systems are dependent on the proposed use and design of the proposed building and all elements of building design and function should be considered in the slab-on-grade floor design. Building design and construction have a greater role in perceived moisture problems

since sealed buildings/rooms or inadequate ventilation may produce excessive moisture in a building and affect indoor air quality.

Various factors such as surface grades, adjacent planters, the quality of slab concrete and the permeability of the onsite soils affect slab moisture and can control future performance. In many cases, floor moisture problems are the result of either improper curing of floors slabs or improper application of flooring adhesives. We recommend contacting a flooring consultant experienced in the area of concrete slab-on-grade floors for specific recommendations regarding your proposed flooring applications.

Special precautions must be taken during the placement and curing of all concrete slabs. Excessive slump (high water-cement ratio) of the concrete and/or improper curing procedures used during either hot or cold weather conditions could lead to excessive shrinkage, cracking, or curling of the slabs. High water-cement ratio and/or improper curing also greatly increase the water vapor permeability of concrete. We recommend that all concrete placement and curing operations be performed in accordance with the American Concrete Institute (ACI) manual.

It is emphasized that we are not floor moisture proofing experts. We make no guarantee nor provide any assurance that use of a capillary break/vapor retarder system will reduce concrete slab-on-grade floor moisture penetration to any specific rate or level, particularly those required by floor covering manufacturers. The builder and designers should consider all available measures for floor slab moisture protection.

Exterior grading will have an impact on potential moisture beneath the floor slabs. Recommendations for exterior drainage are provided in the "Site Drainage" section of this report.

5.4.2 Exterior Flatwork

Exterior concrete flatwork may be supported directly on properly compacted and moisture conditioned native sand subgrade soils. The subgrade to receive exterior concrete flatwork should be moisture conditioned and compacted according to the recommendations in Exhibit 1 in Appendix A. Where concrete flatwork is to be exposed to vehicular traffic, it should be underlain by Caltrans Class 2 aggregate baserock (refer to Section 26 of the 2010 Caltrans Standard Specifications). Exterior flatwork will be subjected to edge effects due to the drying out of subgrade soils.

Flatwork should have control joints spaced a maximum 8 feet on centers. Subgrade for flatwork should be checked by a BSK representative to confirm proper moisture conditioning prior to concrete placement. If the moisture is found to be below the levels required in Exhibit 1, the flatwork areas will need to be soaked until the proper moisture content is reached. Expansion joint material should be used between flatwork and buildings.

5.5 Earthwork

Earthwork at this project will generally consist of the following:

- Mass grading;
- Subgrade preparation;
- Excavation and backfill for existing utility lines to be removed or relocated and new utility lines to be installed;
- Placement of aggregate baserock or crushed rock for concrete flatwork, pavements, and slabs-on-grade;
- Backfill behind below grade walls; and
- Foundation excavations.

Although grading plans are not currently available for the project, we anticipate the project will require fills up to about 3 feet in thickness to backfill the drying beds and raise the site to finished design grade elevations. Excavations up to approximately 15 feet deep are expected to construct the new digesters, install new underground pipelines, and excavate shallow footings for the digester control building. BSK should review the final grading plans for conformance to our design recommendations prior to construction bidding. In addition, it is important that a representative of BSK observe and evaluate the competency of existing soils or new fill during construction. In general, soft/loose or unsuitable materials encountered should be overexcavated, removed, and replaced with compacted engineered fill material.

Site preparation and grading for this project should be performed in accordance with the site-specific recommendations provided below. A summary of soil compaction recommendations for this project is presented in Exhibit 1 in Appendix C. Additional earthwork recommendations are presented in related sections of this report.

5.5.1 Existing Utilities

Prior to ground improvement operations, existing underground utilities within the treatment areas and their influence zone (refer to the "Construction Considerations" section of this report) should be identified and relocated outside or protected within the influence zone of the

areas to be treated. Active or inactive utilities located underneath the planned structures for this project, but beyond the influence zone of the areas to be treated should be protected, relocated, or abandoned. Pipelines that are 2 inches or less in diameter may be left in place provided they are cut off and capped. Pipelines larger than 2 inches in diameter should be removed or filled with a 1-sack sand-cement slurry mix. Active utilities to be reused should be carefully located and protected during demolition and construction activities at the site.

5.5.2 Site Preparation, Grading, and Compaction

Prior to the start of grading and subgrade preparation operations, areas of the site to generate borrow material to be used as engineered fill (i.e., Borrow Areas A through D), to receive fill (such as Pads 1 through 4), or to be covered by pavements, concrete flatwork, and structures, should first be cleared and stripped to remove all surface vegetation, organic-laden topsoil and debris generated during the demolition of existing site improvements located in these areas. Stripping to a minimum depth of 3 inches is expected to remove a majority of loose and organic laden surficial soils in the agricultural land areas of the site. If significant amounts of organics are encountered below this depth, additional stripping may be required. Stripping should extend a minimum of 5 feet laterally beyond the limits of new structures (defined as the outside perimeter of a structure's walls or footing outer limits, whichever results in the greatest footprint), 5 feet laterally beyond the limits of mat foundations, and 2 feet beyond edge of flatwork and pavement, where feasible. Stripped topsoil from vegetated areas and soil from the existing stockpiles shown on Plate 2 may be stockpiled for later use in landscaping areas or to line the bottom of the proposed detention ponds; however, this material should not be reused for engineered fill.

Existing aggregate base, asphalt, and concrete (if broken up to within the grading requirements specified below for imported fill material) may be re-used as general fill, but should not be used within the footprint of the new building and mat foundations without prior approval from the owner. Any buried tree stumps, roots, or major root systems thicker than approximately 1-inch in diameter, abandoned foundations, septic tanks and leach field lines, uncovered during site stripping and/or grading activities should be removed.

Following stripping and removal of deleterious materials in non-agricultural land areas of Pads 1 through 4, the upper 12 inches (minimum) of the areas of the site to receive fill (such as Pads 1 through 4) should be scarified, moisture conditioned, and recompacted as indicated in Exhibit 1 unless otherwise indicated by a BSK representative. Scarification is not anticipated for the digesters. Agricultural land areas of Pads 1 through 4 will likely require up to 24 inches of scarification due to previous ground tilling. This can be achieved by overexcavating the exposed subgrade 12 inches, followed by 12 inches of scarification, followed by backfilling of the

overexcavated areas. Scarification and recompaction should extend laterally a minimum of 2 feet beyond the limits of new fills, where achievable. If any undocumented fill is encountered, the fill should be evaluated by a BSK representative and, if deemed necessary, removed and replaced as engineered fill. All fills should be compacted in lifts of 8-inch maximum uncompacted thickness. Jetting or ponding should not be permitted. A summary of compaction requirements for the project is presented in Exhibit 1. Laboratory maximum dry density and optimum moisture content relationships should be evaluated based on ASTM Test Designation D1557 (latest edition).

Subgrade soils, fill, and backfill, including Pads 1 through 4, mat foundations, the building pad for the digester control building, and the bottom of the below-grade detention ponds, should be compacted to a minimum of 90 percent compaction at near optimum moisture content. The permeable material recommended below the digesters needs to be track-walked or vibrated to a firm and stable condition. Where backfills are greater than 7 feet in depth below finish grade, the entire backfill should be compacted to a minimum of 95 percent compaction. In paved areas, including mat foundations exposed to vehicular/truck traffic, the upper 12 inches of the subgrade and the aggregate base above it should be compacted to a minimum of 95 percent compaction at near optimum moisture content.

Where access for compaction testing in deep excavations is limited due to trench stability, safety, and other access concerns, sand-cement slurry or controlled low strength material (CLSM) may be considered as an alternative to soil backfill if permitted by the owner. If this type of backfill material is used, the utility lines should be anchored to prevent the pipe from floating. The slurry or CLSM should be properly vibrated to allow backfilling under the spring line of the pipes affected.

Sand-cement slurry backfill typically consists of a 1- or 2-sack mix. CLSM typically consists of a mixture of cement, fly ash, coarse and fine aggregate, an air entrainment admixture and water. It should have a 28-day compressive strength in the range of 50 to 150 pounds per square inch (psi), density in the range of 115 to 145 pounds per cubic foot (pcf), and a set time on the order of 4 to 6 hours. Such materials are locally produced, and have been used successfully in several local jurisdictions where fast-setting, low-strength backfill is needed. The proposed materials and method of construction should be reviewed by the project designer and BSK prior to their approval and use.

All site preparation and fill placement should be observed by a BSK representative. During the stripping process, it is important that our representative be present to observe whether any undesirable material is encountered in the construction area and whether exposed soils are similar to those encountered during our subsurface investigation.

5.5.3 Re-Use of Onsite Soil and Imported Fill Material

As previously discussed in the "Re-Use of Onsite Soil" section of this report, from a geotechnical standpoint only, excavated onsite soils are suitable for re-use as general engineering fill and backfill, provided vegetation, organic materials, and deleterious matter are removed. A BSK representative should be present onsite during grading to visually confirm the suitability of the soil to be used as fill and backfill, especially the existing stockpiles and the borrow areas shown on Plate 2. Particles larger than 3 inches within the onsite soils to be used as engineer fill should either be removed and disposed offsite or broken down to 3 inches or less. Nesting (i.e., concentration) of larger particles should be avoided to reduce the potential that this could create voids and allow future settlement in the overlaying fill/backfill.

Maximum particle size for fill material should be limited to 3 inches, with at least 90 percent by weight passing the 1-inch sieve. In addition, imported fill should adhere to the above gradation recommendations and conform to the following minimum criteria:

IMPORT FILL CRITERIA	
Plasticity Index	15 or less
Liquid Limit	Less than 30%
% Passing #200 Sieve	8 % – 40%
Corrosivity	Refer to the "Corrosion" section of this report

Where excavations extend deeper than 10 feet, the material below a depth of 10 feet should not be used as general fill underneath the planned paved driveways/parking lots, mat foundations, the digester control building, exterior concrete flatwork, and underground utility trenches crossing these improvements. Such material should be segregated and placed as general fill or backfill outside these areas.

Imported fill material should not be any more corrosive than the onsite soils and should not be classified as being more corrosive than "moderately corrosive." If the imported fill is used as fill or backfill within the limits of the paved driveways and within 5 feet of finished subgrade elevation, it should have a minimum R-Value of 40.

Prior to transporting proposed import materials to the site, the contractor should make representative samples of the material available to the geotechnical engineer at least 5 working days in advance to allow the engineer enough time to confirm the material meets the above requirements. If prior corrosion testing results are not available for the proposed import fill materials, then the samples should be made available to the geotechnical engineer at least 10 working days in advance so that corrosion testing may be conducted if this testing is deemed

necessary. All on-site or import fill material should be compacted to the recommendations provided for engineered fill in Exhibit 1.

5.5.4 Weather/Moisture Considerations

If earthwork operations and construction for this project are scheduled to be performed during the rainy season (usually November to May) or in areas containing saturated soils, provisions may be required for drying and/or stabilizing the soil through the use of scarification and air drying, geotextile fabric and dryer soils, and/or via admixtures, such as lime- or cement-treatment of the soil prior to compaction. Conversely, additional moisture may be required during dry months. Water trucks should be made available in sufficient numbers to provide adequate water during earthwork operations.

5.5.5 Excavation, Shoring, and Backfill

We anticipate that excavations can be made with standard earthwork equipment, such as excavators, dozers, backhoes, and trenchers. Because the site is underlain primarily by silty to poorly graded sand with silt soils, shoring or sloping of cut faces and trench walls will be necessary to protect personnel and to provide temporary stability.

All excavations made at the site should be evaluated to monitor stability prior to personnel entering them. All trenches and excavations should conform to the current OSHA requirements for work safety. Based on our findings, we anticipate that a maximum slope inclination of 1½H:1V for excavations up to 20 feet in depth could be used at this site. However, it is the contractor's responsibility to follow OSHA temporary excavation guidelines and grade the slopes with adequate layback or provide adequate shoring and underpinning of existing structures and improvements, as needed. Slope layback and/or shoring measures should be adjusted as necessary in the field to suit the actual conditions encountered in order to protect personnel and equipment within excavations.

Where the stability of adjoining structures could be endangered by excavation operations, support systems such as shoring, bracing, or underpinning may be required to provide structural stability and to protect personnel working within the excavation. The design and installation of shoring, bracing, or underpinning required for the project should be the responsibility of the contractor and should be designed by a professional engineer registered in the State of California. We recommend that the proposed shoring, bracing, and underpinning system design be submitted (along with the appropriate design calculations) in advance for review by the design team. The purpose of the review would be to evaluate whether proper soil

parameters have been used and to confirm whether the anticipated deflections are within the tolerance established by the owner or its designer.

Due to the presence of predominantly fine grained silty sand and poorly graded sand layers underlying the site, shored excavations will likely require use of continuous or solid-type shoring during excavation, such as sheet piles. If soldier piles are to be used, continuous lagging will be necessary to prevent caving of the excavation walls. Discontinuous, conventional shoring, such as trench boxes and hydraulic shores with plywood/steel plates ("speed shores"), may not adequately support excavation walls because the sand may not stand vertically long enough to move shoring into place following excavation. Shoring should be removed as the excavations are backfilled. Shoring should be designed to resist earth pressures exerted by the retained soil plus any applicable surcharge loading, such as construction equipment and stockpiles.

Excavations should be properly dewatered as discussed in the "Temporary Dewatering" section below. Construction equipment and soil stockpiles should be set back a minimum horizontal distance of H away from the edge of excavations, where H is equal to the depth of the excavation. This setback distance also applies to shored excavations unless the shoring design takes into account any surcharge loads associated with the construction equipment and stockpiles.

Care should be taken during construction to reduce the impact of trenching on adjacent structures and pavements. Excavations should be located so that no structures, foundations, and slabs, existing or new, are located above an imaginary plane projected 1H:1V upward from any point in an excavation unless the excavation is properly shored and excavated in stages. If structures are located within this 1H:1V project line, the shoring should be designed to handle the surcharge loading from the adjacent structure and allow no horizontal movement of the excavation. Prior to the installation of the shoring and excavation, monitoring points should be established immediately behind the shoring, at midway points between the adjacent structure and the shoring, and at the edge of the adjacent structure. These points should be surveyed daily during installation of the shoring and staged excavation. If any lateral movement is detected, the excavation operation should be stopped immediately and measures should be taken to halt further movement, such as placing a fill buttress in front of the shoring. The shoring design should then be reevaluated and revised as needed. The design and installation of shoring, bracing, or underpinning required for the project should be the responsibility of the contractor and should be designed by a professional engineer registered in the State of California.

During wet weather, appropriate provisions, such as the use of earthen berms, should be made to prevent water runoff from ponding adjacent to the top of excavations and/or flowing over the sides of the excavations, otherwise the excavations side walls and/or slopes could be compromised. All runoff should be collected and disposed of outside the construction limits. Backfill for excavations should be compacted as noted in Exhibit 1. Special care should be taken in the control of excavation backfilling under structures and pavements. Poor compaction may cause excessive settlements resulting in damage to overlying structures and the pavement structural section.

5.5.6 Temporary Dewatering

We expect that cut and cover excavation methods and temporary shoring will be used to install new utility lines. Some of these excavations will extend to depths of 10+ feet. We expect that excavations deeper than about 5 to 7 feet below the existing ground surface (this is equivalent to a groundwater elevation of about 19 feet) will need to be continuously dewatered during construction (depending on the time of year). The soils encountered within the maximum depth of our exploration (about 50 feet below the ground surface) consisted primarily of silty to poorly graded sand layers, but lesser amounts of clay and silt layers were also present. Depending on their fines content (i.e., amount of material passing the No. 200 sieve), sand layers typically have much higher transmissivity rates than clay and silt layers. Groundwater should be lowered and maintained at least 2 feet below the bottom of the planned excavations in order to maintain the undisturbed state of the supporting soils and to allow proper compaction of backfill after below grade structures and utility lines are installed. After completion of all below grade structures and utility lines, the dewatering operations may be terminated to allow the groundwater table to return to its natural level.

We anticipate that dewatering in the project area will be performed in stages and can be performed using deep wells, well points, sumps, drains, and open pumping. However, because the subsurface conditions consist of loose to medium-dense sand, slope stability and boiling at the bottom of the excavations could pose a problem. **The contractor should be fully responsible for developing and implementing a dewatering program.** This should include making any necessary adjustments to the dewatering program during construction based on actual field conditions encountered.

The successful implementation of the dewatering program for this project will be substantially determined by the experience and performance of the contractor retained to perform the dewatering. **Therefore, we recommend that the general contractor for the project be required to retain the services of a specialty dewatering subcontractor to review the anticipated subsurface conditions, develop, and implement a proper dewatering program for the project.**

We recommend the use of a specialty dewatering subcontractor with a minimum of 5 to 10 years of continuous construction experience in similar subsurface conditions on projects of similar scope (i.e., depth of excavations, proximity to and type of existing structures and utility lines, etc.). The dewatering subcontractor selected should provide examples of dewatering for projects they have successfully completed in the past 5 years under similar subsurface conditions and similar scope to this project. The example projects should note instances when things went wrong during particular projects and how they were successfully remediated during construction.

Temporary dewatering may cause ground subsidence that could result in adverse settlement of structures near the areas being dewatered. Therefore, the dewatering subcontractor should evaluate the need to install observations wells between existing structures and the dewatering activities to monitor changes in groundwater levels. If dewatering-induced settlements are anticipated by the dewatering subcontractor, it should consider implementing modifications to its dewatering program and possibly underpinning existing structures (if allowed by the owner and/or its consultants). If underpinning is anticipated by the dewatering subcontractor, prior to implementation of the underpinning, the project owner and its consultants should review the underpinning plans to evaluate the assumptions made in the underpinning design. This review should not be considered as relieving the dewatering subcontractor from full responsibility for the underpinning plans and its satisfactory implementation.

Consideration should also be given by the dewatering subcontractor to installing ground surface settlement monuments adjacent to structures located near areas of the site to be dewatered and monitoring these monuments on a regular basis during dewatering activities. Monitoring records should be made available to the owner and its consultants on a regular basis during construction. If significant movement of the ground surface is noticed during or after the dewatering operation is completed, measures should be immediately taken by the dewatering subcontractor to arrest the settlement. The dewatering subcontractor should then develop and implement a plan for successfully mitigating the settlement.

5.5.7 Below-Grade Detention Ponds

The side slopes for the planned below-grade detention ponds should not be steeper than 2H:1V. Unless the side slopes are protected with a liner, such as fabric, concrete, or rip rap rock, they will likely require periodic maintenance due to surface erosion. After the ponds are excavated, their side slopes should be track-walked by a bulldozer to provide a firm and stable finished surface. If desired, the bottom of the ponds may be overexcavated and backfilled with the strippings generated from the grading for this project. To avoid undermining the pond side slopes, this overexcavation should be setback a minimum of 5 feet laterally from the toe of the

side slopes. The depth of the overexcavation will depend on the number and size of the ponds versus the amount of strippings generated. The bottom of the ponds, including the stripping backfill, should be properly compacted to the requirements presented in Exhibit 1 in Appendix C.

5.6 Asphalt Concrete Pavements

We performed an R-Value test on a combined bulk sample collected from the upper 2 feet at the borings B-2 and B-4, which resulted in a value of 73. Previous geotechnical investigations near the site have obtained R-Value test results ranging from 50 to 72. Due to the potential variability of the silt content contained in the surficial soils at the site and the fact that soil generated from the four planned borrow areas shown on Plate 2 could be used to raise site grade for the planned driveways, we recommend using an R-Value of 40 for the design of the pavement section for this project.

Minimum pavement design sections for various Traffic Indices (TIs) are presented in the table below. Each TI represents a different level of use. The designer should determine which level of use best reflects the project and select appropriate pavement sections. Also, the designer or owner may elect to use thicker pavement sections than those shown in the table below. AC pavement sections for this project were developed using the Caltrans Flexible Pavement Design Method. The recommended pavement sections are presented in the table below and include a factor of safety of 0.2 feet as per the Caltrans Flexible Pavement Design Manual.

ASPHALT CONCRETE PAVEMENT DESIGN (ASSUMED R-VALUE = 40)		
Traffic Index	AC	AB
5.0	2.5	5
5.5	3	5
6.0	3	6
Notes: Thicknesses shown are in inches AC = Type A Asphalt Concrete AB = Caltrans Class 2 Aggregate Baserock (refer to Section 26 of the 2010 Caltrans Standard Specifications)		

We recommend that the subgrade soil over which the pavement section is to be constructed be moisture conditioned and compacted according to the recommendations in Exhibit 1. Subgrade preparation should extend a minimum of 2 feet laterally beyond the pavement limits.

As previously discussed, where excavations extend deeper than 10 feet, the material below a depth of 10 feet should not be used as general fill underneath the planned paved driveways/parking lots because these excavations could extend into layers that are sometimes composed primarily of silt and clay, which typically have a much lower R-Value than the predominantly sand soils present at the ground surface.

The pavement should be sloped and drainage gradients maintained to carry all surface water to appropriate collection points. Surface water ponding should not be allowed anywhere on the site during or after construction.

In addition, we recommend that all pavements conform to the following criteria:

- All excavation backfills should be properly placed and adequately compacted to provide a stable subgrade, in accordance with the compaction recommendations in Exhibit 1;
- An adequate drainage system should be provided to prevent surface water or subsurface seepage from saturating the subgrade soil;
- The asphalt concrete and aggregate base materials should conform to Caltrans Specifications, latest edition; and
- Placement and compaction of pavements should be performed and tested in accordance to appropriate Caltrans test procedures.

5.7 Storm Water Infiltration

Storm runoff regulations require pretreatment of runoff and infiltration of storm water to the extent feasible. Typically, this results in the use of bioretention areas, vegetated swales, infiltration trenches, or permeable pavement near or within parking lots and at the location of roof run-off collection. These features are well-suited for coarse-grained soils. However, the surficial soils at the site are predominantly composed of fine grained silty sand and poorly graded sand with silt. Typically, the higher the silt content of such soils, the lower the infiltration rate.

Bioretention areas, vegetated swales, and infiltration areas should be located in landscaped areas and well away from pavements, buildings, and slopes to reduce potential adverse impacts to these improvements. If it is not possible to locate these infiltration systems away from buildings and pavements, alternatives should be considered that isolate the infiltrated water from planned improvements, such as flow-through planters.

5.8 Corrosion

Samples of the subsurface soils at depths of approximately 16 and 5½ feet at borings B-1 and B-3, respectively, were submitted for corrosion testing. The samples were tested by CERCO Analytical, a State-certified laboratory in Concord, California, for redox potential, pH, resistivity, chloride content, and sulfate content in accordance with ASTM test methods. The test results are presented in Appendix B. Also included is an August 12, 2015 letter by CERCO Analytical evaluating the corrosion test results. Because we are not corrosion specialists, we recommend that a corrosion specialist be consulted for advice on proper corrosion protection for underground piping which will be in contact with the soils and other design details.

Based upon the resistivity measurements, the samples tested are classified as “mildly corrosive” to “moderately corrosive” by CERCO Analytical. They recommend that all buried iron, steel, cast iron, ductile iron, galvanized steel, and dielectric coated steel or iron be properly protected against corrosion depending upon the critical nature of the structure. They also recommend all buried metallic pressure piping, such as ductile iron firewater pipelines, should be protected against corrosion.

The above are general discussions. A more detailed investigation may include more or fewer concerns, and should be directed by a corrosion expert. BSK does not practice corrosion engineering. Consideration should also be given to soils in contact with concrete that will be imported to the site during construction, such as topsoil and landscaping materials. For instance, any imported soil materials should not be any more corrosive than the onsite soils and should not be classified as being more corrosive than “moderately corrosive.” Also, onsite cutting and filling may result in soils contacting concrete that were not anticipated at the time of this investigation.

5.9 Plan Review and Construction Observation

We recommend that BSK be retained by the Client to review the final structural and grading plans and specifications before they go out to bid. It has been our experience that this review provides an opportunity to detect misinterpretation or misunderstandings of our recommendation prior to the start of construction.

Variations in soil types and conditions are possible and may be encountered during construction. To permit correlation between the soil data obtained during this investigation and the actual soil conditions encountered during construction, we recommend that BSK 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 provide supplemental recommendations if warranted by the exposed conditions. Earthwork should be performed in accordance with the recommendations presented in this report, or as recommended by BSK during construction. BSK should be notified at least two weeks prior to the start of construction and prior to when observation and testing services are needed.

6. ADDITIONAL SERVICES AND LIMITATIONS

6.1 Additional Services

The review of plans and specifications, and field observation and testing during construction by BSK are an integral part of the conclusions and recommendations made in this report. If BSK is not retained for these services, the client will be assuming BSK's responsibility for any potential claims that may arise during or after construction due to the misinterpretation of the recommendations presented herein. The recommended tests, observations, and consultation by BSK during construction include, but are not limited to:

- review of plans and specifications;
- observation and monitoring of ground improvement operations;
- observations of site grading, including stripping and engineered fill placement;
- observation of foundation and below grade walls; and
- in-place density testing of fills, backfills, and finished subgrades.

6.2 Limitations

The recommendations contained in this report are based on our field observations and subsurface explorations, limited laboratory tests, review of available geologic maps and publications, review of previous investigations near the site, and our present knowledge of the proposed construction. It is possible that soil conditions could vary between or beyond the points explored. If soil conditions are encountered during construction that differ from those described herein, we should be notified immediately in order that a review may be made and any supplemental recommendations provided. If the scope of the proposed construction, including the proposed loads or structural locations, changes from that described in this report, our recommendations should also be reviewed.

We prepared this report in substantial accordance with the generally accepted geotechnical engineering practice as it exists in the site area at the time of our study. No warranty, either express or implied, is made. The recommendations provided in this report are based on the assumption that an adequate program of tests and observations will be conducted by BSK during the construction phase in order to evaluate compliance with our recommendations. Other standards or documents referenced in any given standard cited in this report, or otherwise relied upon by the author of this report, are only mentioned in the given standard; they are not incorporated into it or "included by reference", as that latter term is used relative to contracts or other matters of law.

This report may be used only by the Client and only for the purposes stated within a reasonable time from its issuance, but in no event later than two (2) years from the date of the report, or if conditions at the site have changed. If this report is used beyond this period, BSK should be contacted to evaluate whether site conditions have changed since the report was issued.

Also, land or facility use, on and off-site conditions, regulations, or other factors may change over time, and additional work may be required with the passage of time. Based on the intended use of the report, BSK may recommend that additional work be performed and that an updated report be issued.


The scope of work for this subsurface investigation and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands at this site.

BSK conducted subsurface exploration and provided recommendations for this project. We understand that BSK will be given the opportunity to perform a formal geotechnical review of the final project plans and specifications. In the event BSK is not retained to review the final project plans and specifications to evaluate if our recommendations have been properly interpreted, we will assume no responsibility for misinterpretation of our recommendations.

We recommend that all earthwork during construction be monitored by a representative from BSK, including site preparation, ground improvement operations, foundation construction, placement of engineered fill, and trench/wall backfill. The purpose of these services would be to provide BSK the opportunity to observe the actual soil conditions encountered during construction, evaluate the applicability of the recommendations presented in this report to the soil conditions encountered, and recommend appropriate changes in design or construction procedures if conditions differ from those described herein.

PLATES




 Approximate Scale
 Not To Scale

Reference: <http://maps.google.com>, 2014



PROJECT NO. **G15-133-10L**
 DRAWN: 12/28/15
 DRAWN BY: B. Steen
 CHECKED BY: C. Melo
 FILE NAME: VicMap.indd

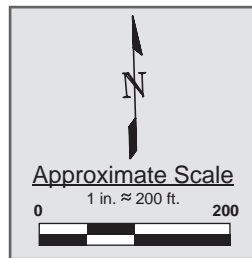
VICINITY MAP

**Manteca Water Quality Control Facility Improvements
Manteca, California**

PLATE

1

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Legend

- B-1 Soil Boring Location (BSK Associates, 2015)
- B-1 Soil Boring Location (Neil O. Anderson Associates, 2014)
- B-1 Soil Boring Location (Kleinfelder, 2005)

REFERENCES:

1. "Digester Improvements Project", Drawing C-10, December 2015, by HERWIT Engineering.
2. Site sketch showing Borrow Areas and Pads, dated December 2015, by HERWIT Engineering.
3. Google Earth Pro (2015).

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Note: Locations and boundaries are approximate.



PROJECT NO. G15-133-10L	SITE PLAN	PLATE
DRAWN: 12/28/15		
DRAWN BY: B. Steen		
CHECKED BY: C. Melo		
FILE NAME: SitePlan.indd	Manteca Water Quality Control Facility Improvements Manteca, California	2

APPENDIX A

BORING LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

MAJOR DIVISIONS		GRAPHIC LOG		TYPICAL DESCRIPTIONS		
COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	GRAVELS (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVELS WITH <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		
	SILTS AND CLAYS (Liquid limit greater than 50)		OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY		
			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				



PROJECT NO. G15-133-10P
DRAWN: 12/28/15
DRAWN BY: B. Steen
CHECKED BY: C. Melo
FILE NAME: Legend.indd

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

Manteca Water Quality Control Facility Improvements
Manteca, California

PLATE

A-1

SOIL DESCRIPTION KEY

MOISTURE CONTENT

DESCRIPTION	ABBR	FIELD TEST
Dry	D	Absence of moisture, dusty, dry to the touch
Moist	M	Damp but no visible water
Wet	W	Visible free water, usually soil is below water table

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

PLASTICITY

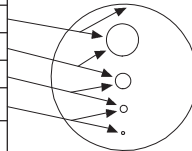
DESCRIPTION	ABBR	FIELD TEST
Non-plastic	NP	A 1/8-in. (3 mm) thread cannot be rolled at any water content.
Low (L)	LP	The thread can barely be rolled and the lump or thread cannot be formed when drier than the plastic limit.
Medium (M)	MP	The thread is easy to roll and not much time is required to reach 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)	HP	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

GRAIN SIZE

DESCRIPTION	SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE	
Boulders	>12"	>12"	Larger than basketball-sized	
Cobbles	3 - 12"	3 - 12"	Fist-sized to basketball-sized	
Gravel	coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19"	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock salt-sized
	fine	#200 - #10	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines	Passing #200	<0.0029	Flour-sized and smaller	

REACTION WITH HCl

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



ANGULARITY

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

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	ABBR	SPT (# blows/ft)	MODIFIED CA SAMPLER (# blows/ft)	CALIFORNIA SAMPLER (# blows/ft)	RELATIVE DENSITY (%)	FIELD TEST
Very Loose	VL	<4	<4	<5	0 - 15	Easily penetrated with 1/2-inch reinforcing rod by hand
Loose	L	4 - 10	5 - 12	5 - 15	15 - 35	Difficult to penetrate with 1/2-inch reinforcing rod pushed by hand
Medium Dense	MD	10 - 30	12 - 35	15 - 40	35 - 65	Easily penetrated a foot with 1/2-inch reinforcing rod driven with 5-lb. hammer
Dense	D	30 - 50	35 - 60	40 - 70	65 - 85	Difficult to penetrate a foot with 1/2-inch reinforcing rod driven with 5-lb. hammer
Very Dense	VD	>50	>60	>70	85 - 100	Penetrated only a few inches with 1/2-inch reinforcing rod driven with 5-lb. hammer



PROJECT NO. G15-133-10P
 DRAWN: 12/28/15
 DRAWN BY: B. Steen
 CHECKED BY: C. Melo
 FILE NAME: Legend.indd



SOIL DESCRIPTION KEY

**Manteca Water Quality Control
 Facility Improvements
 Manteca, California**

PLATE

A-2

LOG SYMBOLS

	BULK / BAG SAMPLE	-4	PERCENT FINER THAN THE NO. 4 SIEVE (ASTM Test Method C 136)
	SPLIT BARREL SAMPLER (2-1/2 inch outside diameter)	-200	PERCENT FINER THAN THE NO. 200 SIEVE (ASTM Test Method C 117)
	SPLIT BARREL SAMPLER (3 inch outside diameter)	LL	LIQUID LIMIT (ASTM Test Method D 4318)
	STANDARD PENETRATION SPLIT SPOON SAMPLER (2 inch outside diameter)	PI	PLASTICITY INDEX (ASTM Test Method D 4318)
	CONTINUOUS CORE	TXUU	UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION (EM 1110-1-1906)/ASTM Test Method D 2850
	SHELBY TUBE	EI	EXPANSION INDEX (UBC STANDARD 18-2)
	ROCK CORE	COL	COLLAPSE POTENTIAL
	GROUNDWATER LEVEL (encountered at time of drilling)	UC	UNCONFINED COMPRESSION (ASTM Test Method D 2166)
	GROUNDWATER LEVEL (measured after drilling)		
	SEEPAGE	MC	MOISTURE CONTENT (ASTM Test Method D 2216)

GENERAL NOTES

Boring log data represents a data snapshot.

This data represents subsurface characteristics only to the extent encountered at the location of the boring.

The data inherently cannot accurately predict the entire subsurface conditions to be encountered at the project site relative to construction or other subsurface activities.

Lines between soil layers and/or rock units are approximate and may be gradual transitions.

The information provided should be used only for the purposes intended as described in the accompanying documents.

In general, Unified Soil Classification System designations presented on the logs were evaluated by visual methods.

Where laboratory tests were performed, the designations reflect the laboratory test results.



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 Livermore, CA 94551
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 Fax: 925-315-3152

LOG OF BORING NO. B-1

Project Name: **Manteca Water Quality Control Facility**
 Project Number: **G15-133-10L**
 Project Location: **Manteca, California**
 Logged by: **K. Johnston**
 Checked by: **C. Melo**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / Foot	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
		Surface El.: 23 feet Location: B-1										
	GRAVEL : fill											
	POORLY GRADED SAND WITH SILT (SP-SM) : olive brown, slightly moist, fine grained sand (FILL)		X									
	POORLY GRADED SAND WITH SILT (SP-SM): olive to olive brown, moist, medium dense, fine to medium grained sand		X									
5		pocket of well graded sand, fine to medium grained sand	█	1A 1B 1C	9 11 14			111	5			
	POORLY GRADED SAND (SP): olive brown, moist, medium dense, fine to medium grained sand		▴	2	4 7 9		3					
	SANDY SILT (ML): olive to olive yellow, moist, firm, slight plasticity		█	3A 3B 3C	10 12 13	3-3.5						
	POORLY GRADED SAND (SP): gray, moist, dense, fine to medium grained sand		▴	4	10 15 18							
	SANDY LEAN CLAY (CL): olive brown, moist, firm, low to medium plasticity		█	5A 5B 5C	6 8 13	3-3.5		83	37			

GEO_TARGET G15-133-10L MANTECA WWTP.GPJ GEOTECHNICAL 08.GDT 1/6/16

Completion Depth: 51.5
Date Started: 7/29/15
Date Completed: 7/29/15
California Sampler: 2.5 inch inner diameter
SPT Sampler: 1.4 inch inner diameter

Drilling Equipment: Exploration Geoservices Inc., Mobile B-53
Drilling Method: 4-inch tricone, rotary wash
Drive Weight: 140 lb
Hole Diameter: 4-inches
Drop: 30 in
Remarks: Surface Conditions: Gravel



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LOG OF BORING NO. B-1

Project Name: **Manteca Water Quality Control Facility**
 Project Number: **G15-133-10L**
 Project Location: **Manteca, California**
 Logged by: **K. Johnston**
 Checked by: **C. Melo**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / Foot	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
Surface El.: 23 feet Location: B-1												
	[Hatched pattern]	SANDY LEAN CLAY (CL): olive brown, moist, firm, low to medium plasticity (<i>continued</i>)	▲	6	4 5 13		50					
	[Dotted pattern]	POORLY GRADED SAND (SP): olive brown, moist, very dense, fine to medium grained sand, mica specs										
35			■	7A 7B 7C	26 36 38			105	20			
		olive brown to olive, dense	▲	8	15 19 28							
		olive yellow, medium dense	■	9A 9B 9C	17 20 19							
45		LEAN CLAY (CL): olive brown, moist, firm, low to medium plasticity										
	[Hatched pattern]											
50		medium plasticity	▲	10	6 9 12							
		Boring terminated at approximately 51.5 feet. Groundwater depth masked by rotary wash drilling method. Boring backfilled with cement grout.										
55												
60												

GEO_TARGET G15-133-10L MANTECA WWTP.GPJ GEOTECHNICAL 08.GDT 1/6/16

Completion Depth: 51.5
Date Started: 7/29/15
Date Completed: 7/29/15
California Sampler: 2.5 inch inner diameter
SPT Sampler: 1.4 inch inner diameter

Drilling Equipment: Exploration Geoservices Inc., Mobile B-53
Drilling Method: 4-inch tricone, rotary wash
Drive Weight: 140 lb
Hole Diameter: 4-inches
Drop: 30 in
Remarks: Surface Conditions: Gravel



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LOG OF BORING NO. B-2

Project Name: **Manteca Water Quality Control Facility**
 Project Number: **G15-133-10L**
 Project Location: **Manteca, California**
 Logged by: **K. Johnston**
 Checked by: **C. Melo**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / Foot	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
Surface El.: 24 feet Location: B-2												
5		POORLY GRADED SAND WITH SILT (SP-SM): olive brown, slightly moist to dry, fine grained sand moist, medium dense		1A 1B 1C	5 8 8		11 9					
10		POORLY GRADED SAND (SP): olive brown, moist, loose to medium dense, fine to coarse grained sand		2	4 4 6							
15		POORLY GRADED SAND WITH SILT (SP-SM): yellowish brown to olive, moist, loose, iron oxide staining, mica specks		3A 3B 3C	5 3 9		7	101	23			
20		POORLY GRADED SAND (SP): gray to olive yellow, wet, dense, medium grained sand		4	14 18 19		5					
25		LEAN CLAY (CL): olive to olive brown, moist, firm, low plasticity		5A 5B 5C	6 8 10	1.5-2	87	94	31	33	22	11
30												

GEO_TARGET G15-133-10L MANTECA WWTP.GPJ GEOTECHNICAL 08.GDT 1/6/16

Completion Depth: 51.5
Date Started: 7/28/15
Date Completed: 7/28/15
California Sampler: 2.5 inch inner diameter
SPT Sampler: 1.4 inch inner diameter

Drilling Equipment: Exploration Geoservices Inc., Mobile B-53
Drilling Method: 4-inch tricone, rotary wash
Drive Weight: 140 lb
Hole Diameter: 4-inches
Drop: 30 in
Remarks: Surface Conditions: Agricultural Land



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LOG OF BORING NO. B-2

Project Name: **Manteca Water Quality Control Facility**
 Project Number: **G15-133-10L**
 Project Location: **Manteca, California**
 Logged by: **K. Johnston**
 Checked by: **C. Melo**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / Foot	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
		Surface El.: 24 feet Location: B-2										
		SANDY LEAN CLAY (CL): olive to olive brown, moist, firm, low plasticity, fine grained sand <i>(continued)</i>	▲	6	4 4 16							
		SILTY SAND (SM): yellowish brown, moist, medium dense, fine to medium grained sand, slight cementation										
35		POORLY GRADED SAND (SP): light brownish gray, moist, very dense, fine to medium grained sand, 1/4" subrounded gravel	■	7A 7B 7C	30 35 38							
		LEAN CLAY (CL): gray to bluish gray, moist, firm, low to medium plasticity	▲	8	7 9 15							
45		olive brown to pale olive, hard, medium plasticity, iron oxide staining	■	9A 9B 9C	15 24 27	4-4.5		101	26			
50		firm, low to medium plasticity	▲	10	4 6 9							
		Boring terminated at approximately 51.5 feet. Groundwater depth masked by rotary wash drilling method. Boring backfilled with cement grout.										
55												
60												

GEO_TARGET G15-133-10L MANTECA WWTP.GPJ GEOTECHNICAL 08.GDT 1/6/16

Completion Depth: 51.5
Date Started: 7/28/15
Date Completed: 7/28/15
California Sampler: 2.5 inch inner diameter
SPT Sampler: 1.4 inch inner diameter

Drilling Equipment: Exploration Geoservices Inc., Mobile B-53
Drilling Method: 4-inch tricone, rotary wash
Drive Weight: 140 lb
Hole Diameter: 4-inches
Drop: 30 in
Remarks: Surface Conditions: Agricultural Land



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LOG OF BORING NO. B-3

Project Name: **Manteca Water Quality Control Facility**
 Project Number: **G15-133-10L**
 Project Location: **Manteca, California**
 Logged by: **K. Johnston**
 Checked by: **C. Melo**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / Foot	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
		Surface El.: 25 feet Location: B-3										
	GRAVEL: fill		X									
	SILTY SAND (SM): olive to olive brown, slightly moist, fine grained sand, mica specs, (FILL)		█	1A	9							
	SILTY SAND (SM): olive to olive brown, slightly moist, medium dense, fine grained sand, mica specs		█	1B	10			106	4			
	SILTY SAND (SM): olive to olive brown, slightly moist, medium dense, fine grained sand, mica specs		█	1C	11							
5	loose		█	2A	3							
			█	2B	3							
			█	2C	3							
10			█	3A	3		41					
			█	3B	3		15					
			█	3C	4							
	▼ 12:47 am											
	▽ 12:12 am											
15	POORLY GRADED SAND (SP): gray to olive gray, wet, medium dense, fine to medium grained sand, some coarse grained sand		█	4A	5			104	20			
			█	4B	6							
			█	4C	10							
20	dense		█	5A	11							
			█	5B	20							
			█	5C	34							
		Boring terminated at approximately 21.5 feet. Boring backfilled with cement grout.										
25												
30												

GEO_TARGET G15-133-10L MANTECA WWTP.GPJ GEOTECHNICAL 08.GDT 1/6/16

Completion Depth: 21.5
Date Started: 7/30/15
Date Completed: 7/30/15
California Sampler: 2.5 inch inner diameter
SPT Sampler: 1.4 inch inner diameter

Drilling Equipment: Exploration Geoservices Inc., Mobile B-53
Drilling Method: 8.25-inch metal carbide combo, hollow stem auger
Drive Weight: 140 lb
Hole Diameter: 8-inches
Drop: 30 in
Remarks: Surface Conditions: Gravel access road



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LOG OF BORING NO. B-4

Project Name: **Manteca Water Quality Control Facility**
 Project Number: **G15-133-10L**
 Project Location: **Manteca, California**
 Logged by: **K. Johnston**
 Checked by: **C. Melo**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / Foot	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
		Surface El.: 25 feet Location: B-4										
	GRAVEL: fill		X									
	POORLY GRADED SAND WITH SILT (SP-SM): olive to olive brown, slightly moist, fine grained sand, mica specs (FILL)		█	1A	10		8					
			█	1B	8							
			█	1C	8							
5	loose		█	2A	4			104	9			
			█	2B	4							
			█	2C	6							
10	POORLY GRADED SAND (SP): gray to olive gray, moist, loose, fine to medium grained sand		█	3A	4							
			█	3B	6							
			█	3C	7							
	▼ 11:49 am											
	▽ 11:34 am											
15	wet, medium dense		█	4A	4			103	18			
			█	4B	9							
			█	4C	10							
20	LEAN CLAY (CL): light brownish gray, moist, firm, low to medium plasticity, mica specs		█	5A	7							
			█	5B	17							
			█	5C	17	2.0-3.0						
	Boring terminated at approximately 21.5 feet. Boring backfilled with cement grout.											

GEO_TARGET G15-133-10L MANTECA WWTP.GPJ GEOTECHNICAL 08.GDT 5/11/16

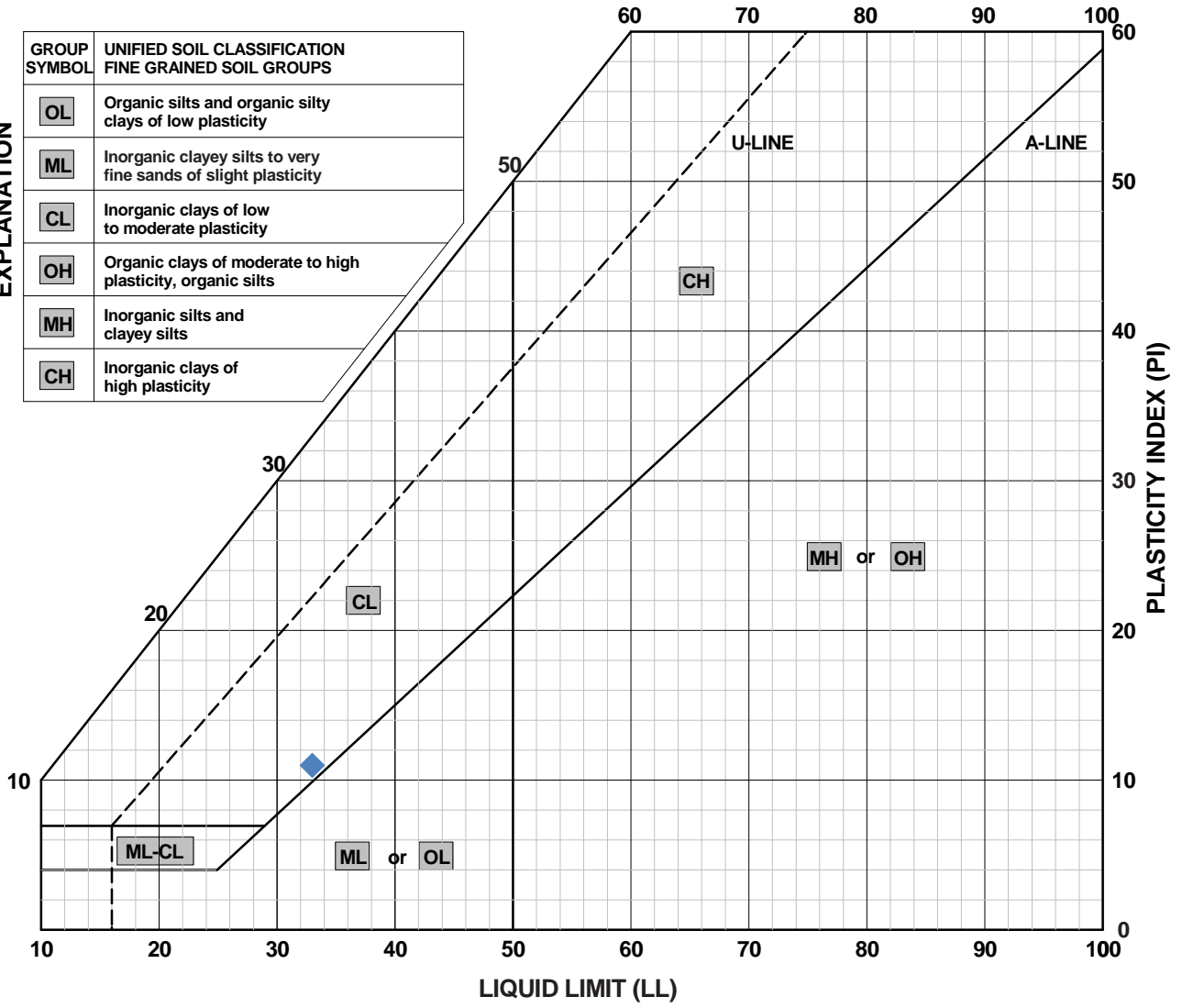
Completion Depth: 21.5 Date Started: 7/30/15 Date Completed: 7/30/15 California Sampler: 2.5 inch inner diameter SPT Sampler: 1.4 inch inner diameter	Drilling Equipment: Exploration Geoservices Inc., Mobile B-53 Drilling Method: 8.25-inch metal carbide combo, hollow stem auger Drive Weight: 140 lb Hole Diameter: 8-inches Drop: 30 in Remarks: Surface Conditions: Gravel access road
--	---

APPENDIX B

LABORATORY TEST RESULTS

EXPLANATION

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE GRAINED SOIL GROUPS
OL	Organic silts and organic silty clays of low plasticity
ML	Inorganic clayey silts to very fine sands of slight plasticity
CL	Inorganic clays of low to moderate plasticity
OH	Organic clays of moderate to high plasticity, organic silts
MH	Inorganic silts and clayey silts
CH	Inorganic clays of high plasticity



LEGEND:	SOURCE	DEPTH (ft)	LL	PL	PI	DESCRIPTION
◆	B-2	25.5	33	22	11	Lean Clay (CL)

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PROJECT NO. G15-133-10P
DRAWN: 12/28/15
DRAWN BY: B. Steen
CHECKED BY: C. Melo
FILE NAME: Lab.indd

<u>ATTERBERG LIMITS</u>
Manteca Water Quality Control Facility Improvements Manteca, California

PLATE
B-1

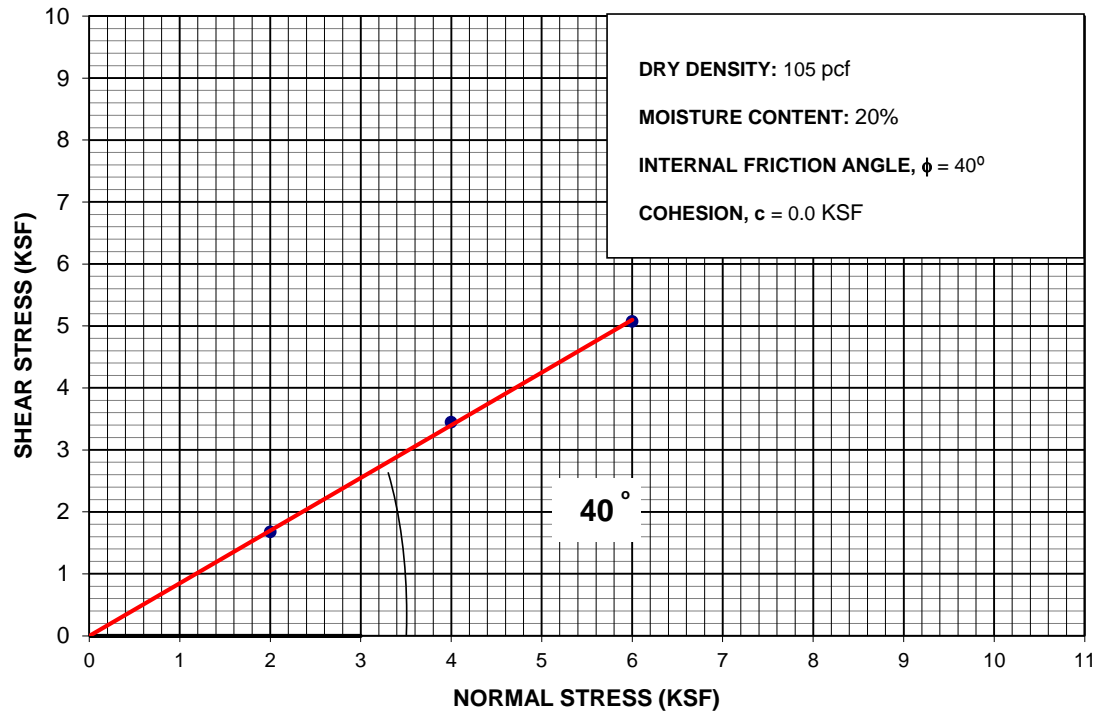
Direct Shear Test
ASTM D-3080

Project Name: Manteca WWTP Improvements
Project Number: G15-133-10L
Sample Location: B-1 @ 36 bgs
Sample Description: SP: Fine Sand; grey; moist

Sampled By: K J
Tested By: V. Simental
Lab Tracking ID: B15-269

Sample Date: 7/29/2015
Test Date: 8/10/2015
Report Date: 8/14/2015

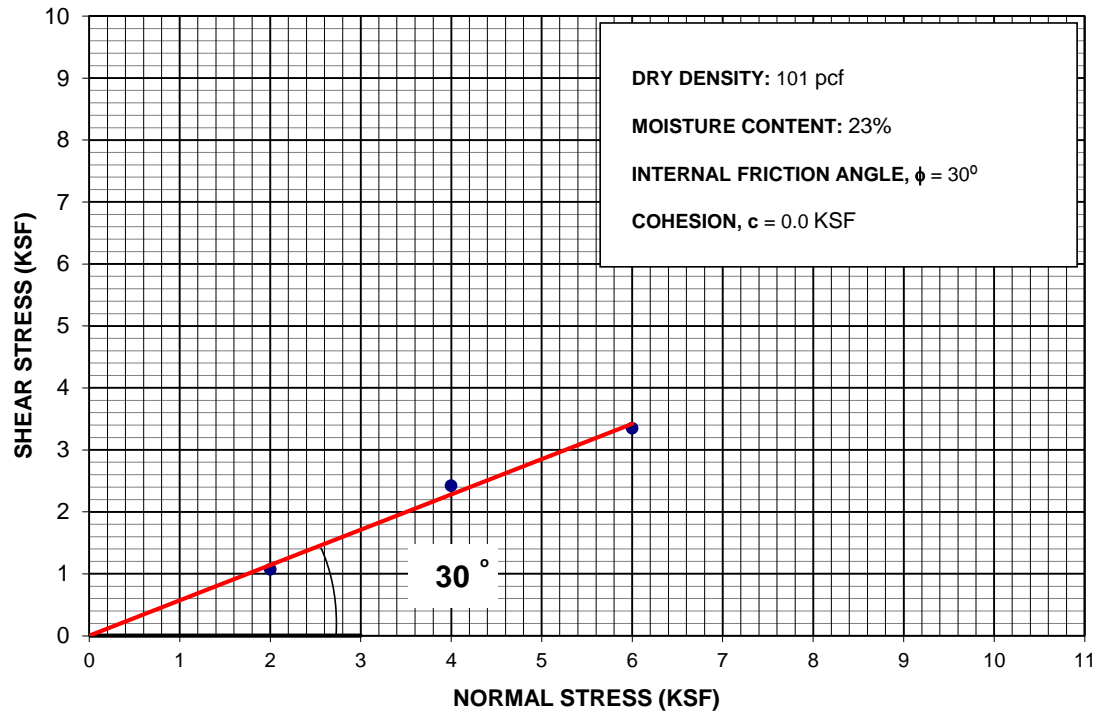
SHEAR STRENGTH DIAGRAM



Direct Shear Test
ASTM D-3080

Project Name:	<u>Manteca WWTP Improvements</u>	Sampled By:	<u>K J</u>	Sample Date:	<u>7/28/2015</u>
Project Number:	<u>G15-133-10L</u>	Tested By:	<u>J. Laybourn</u>	Test Date:	<u>8/11/2015</u>
Sample Location:	<u>B-2 @ 16 bgs</u>	Lab Tracking ID:	<u>B15-269</u>	Report Date:	<u>8/14/2015</u>
Sample Description:	<u>SP-SM: Poorly Graded Sand with Silt; light brown; f-m; moist</u>				

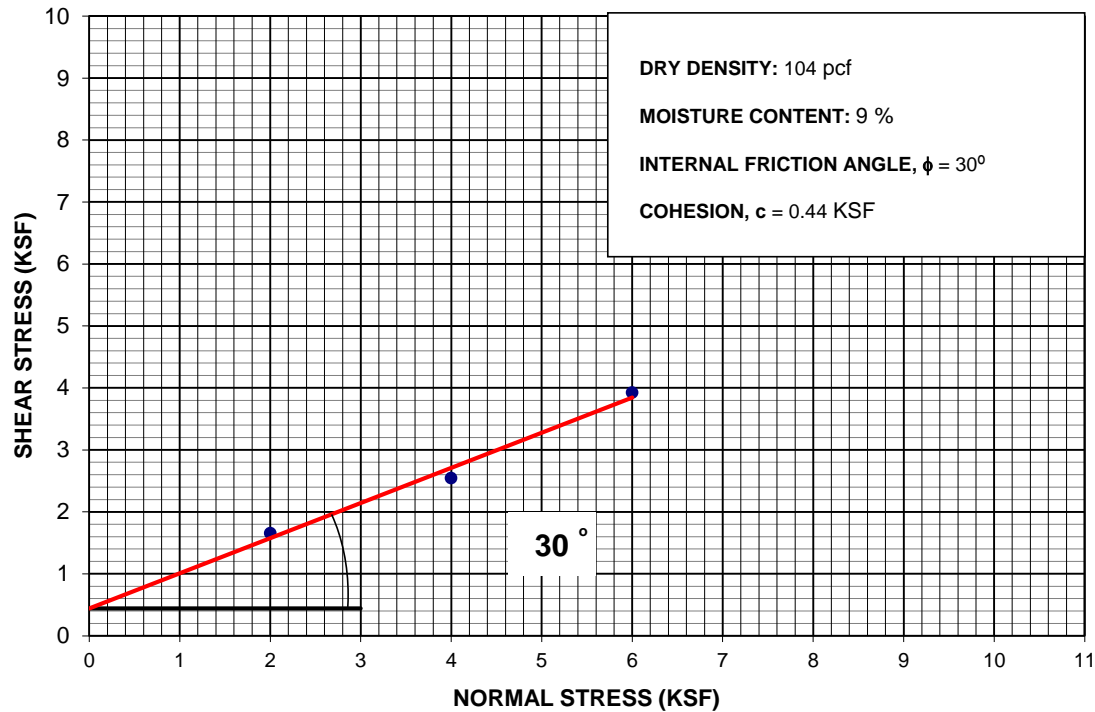
SHEAR STRENGTH DIAGRAM



Direct Shear Test
ASTM D-3080

Project Name:	<u>Manteca WWTP Improvements</u>	Sampled By:	<u>K J</u>	Sample Date:	<u>7/30/2015</u>
Project Number:	<u>G15-133-10L</u>	Tested By:	<u>J. Laybourn</u>	Test Date:	<u>8/11/2015</u>
Sample Location:	<u>B-4 @ 6 bgs</u>	Lab Tracking ID:	<u>B15-269</u>	Report Date:	<u>8/14/2015</u>
Sample Description:	<u>SP-SM: Poolry Graded Sand with Silt; brown; fine to medium; moist</u>				

SHEAR STRENGTH DIAGRAM



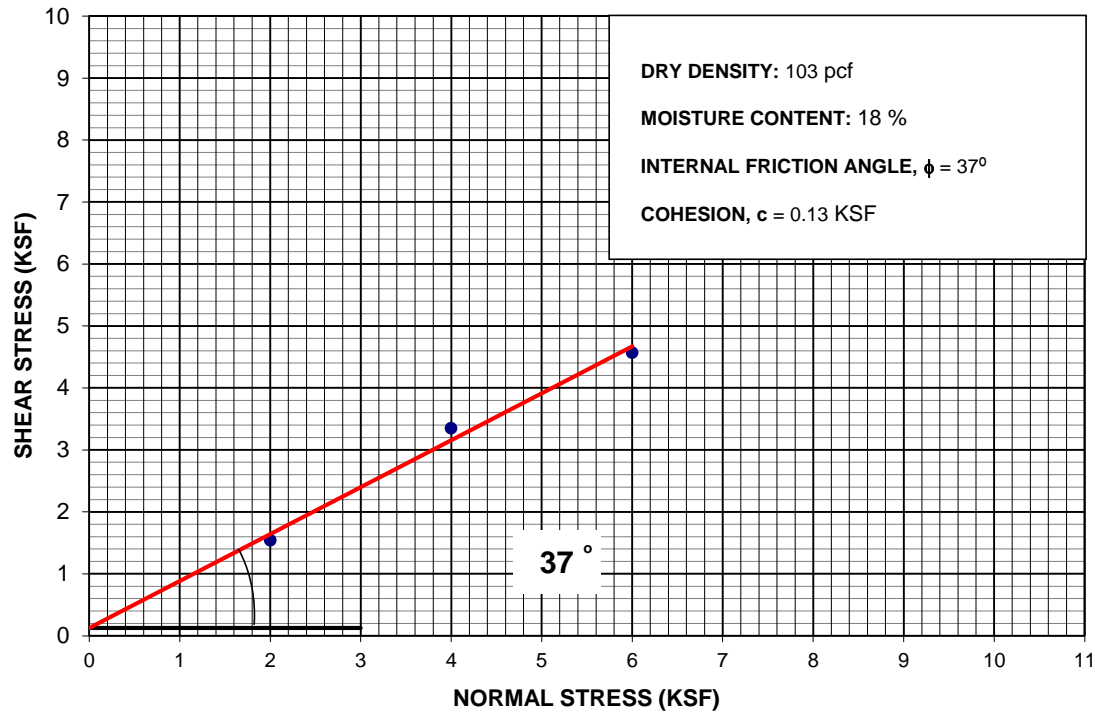
Direct Shear Test
ASTM D-3080

Project Name: Manteca WWTP Improvements
Project Number: G15-133-10L
Sample Location: B-4 @ 16 bgs
Sample Description: SP: Fine Sand; grey; wet

Sampled By: KJ
Tested By: J. Laybourn
Lab Tracking ID: B15-269

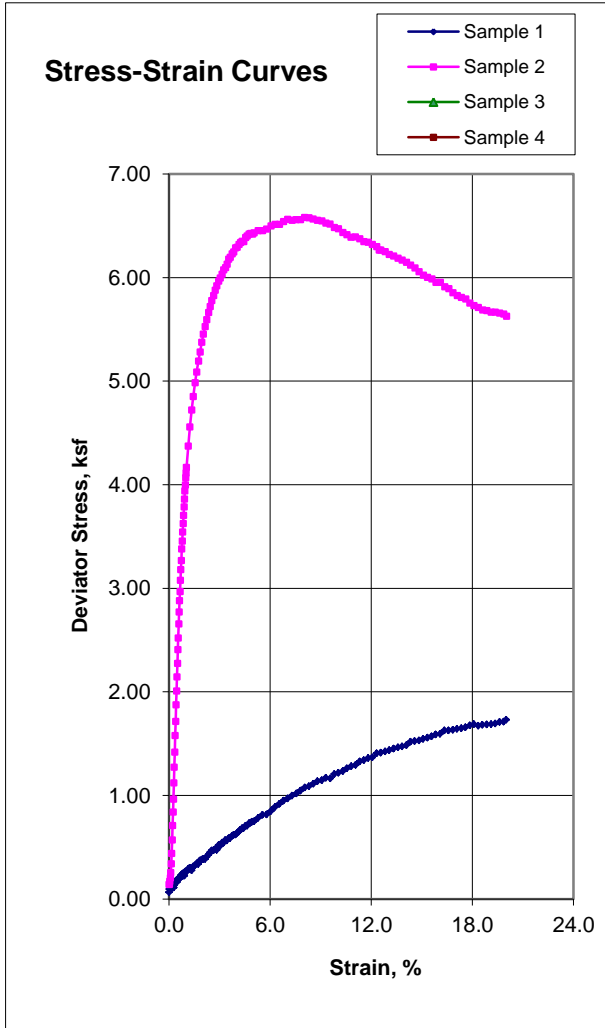
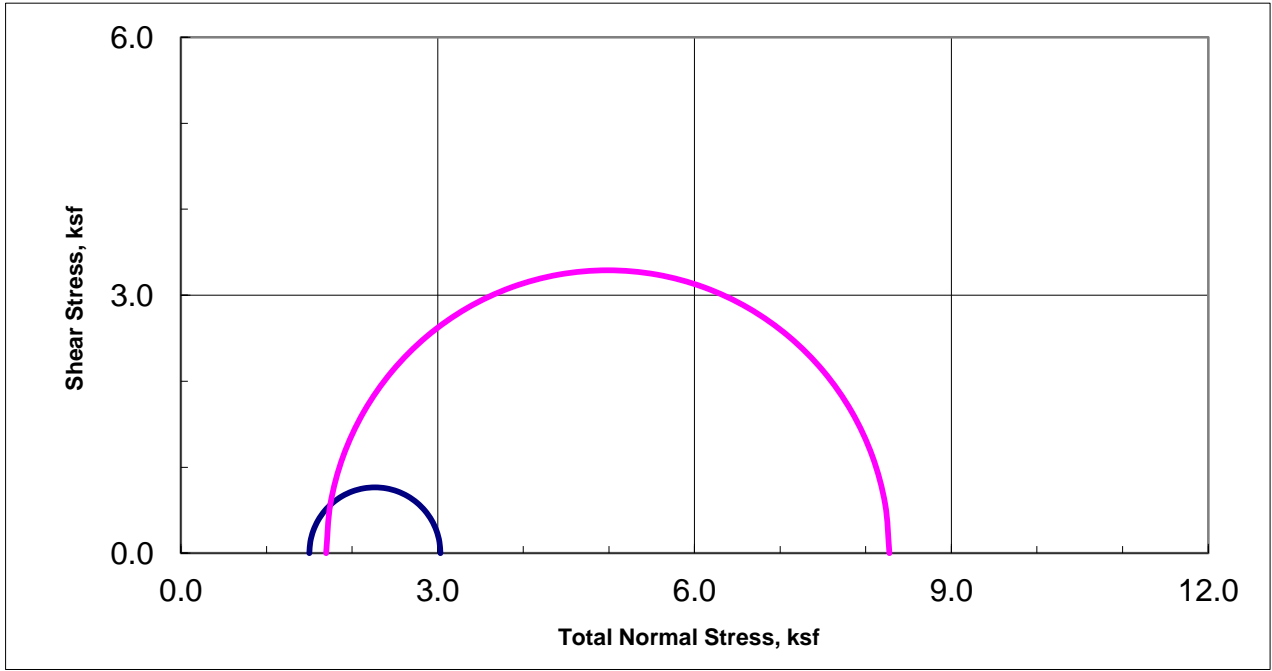
Sample Date: 7/30/2015
Test Date: 8/11/2015
Report Date: 8/14/2015

SHEAR STRENGTH DIAGRAM





Unconsolidated-Undrained Triaxial Test
 ASTM D2850



Sample Data				
	1	2	3	4
Moisture %	30.9	25.7		
Dry Den,pcf	93.7	100.5		
Void Ratio	0.865	0.739		
Saturation %	100.0	97.3		
Height in	4.99	5.00		
Diameter in	2.39	2.41		
Cell psi	10.4	11.8		
Strain %	15.00	8.05		
Deviator, ksf	1.531	6.580		
Rate %/min	1.00	1.00		
in/min	0.050	0.050		
Job No.:	664-048			
Client:	BSK Associates			
Project:	Manteca WWTP - G15-133-10L			
Boring:	B-2	B-2		
Sample:	5C	9C		
Depth ft:	26	46		
Visual Soil Description				
Sample #				
1	Grayish Brown CLAY, trace Sand			
2	Olive CLAY			
3				
4				
Remarks:				

Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.



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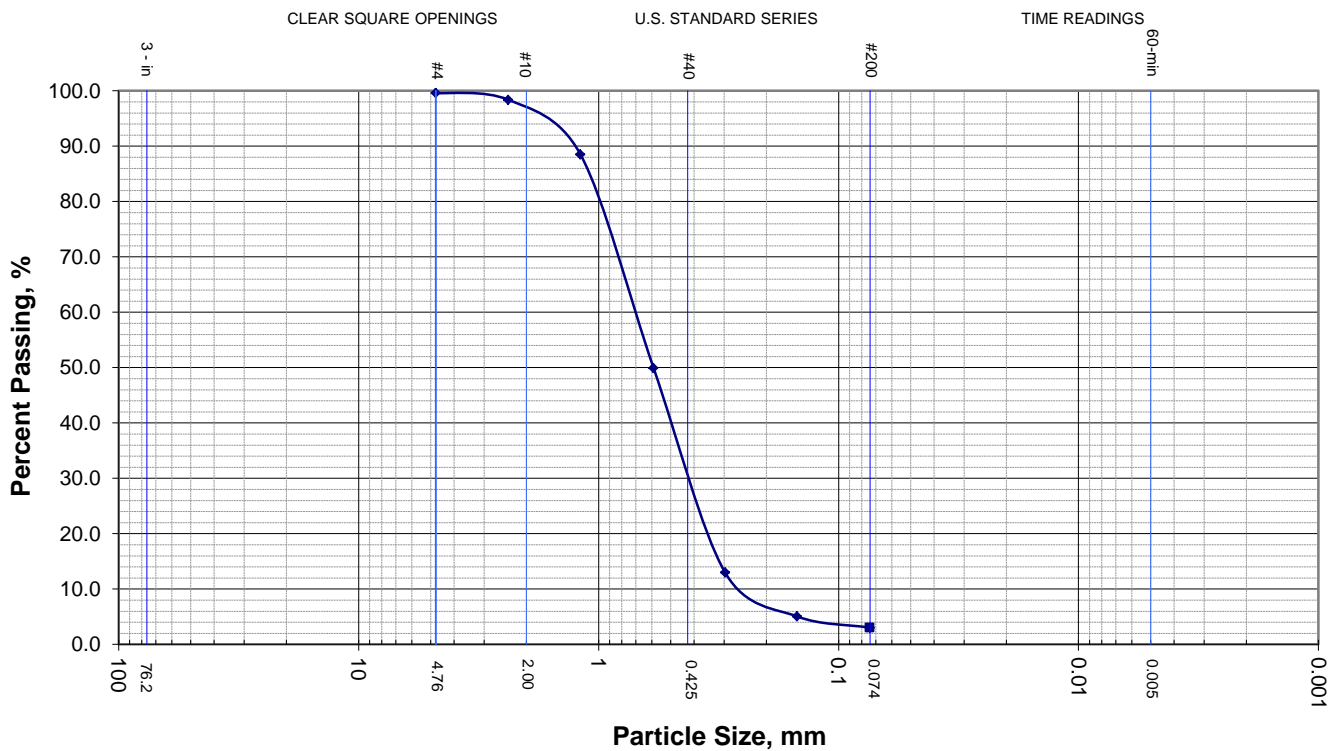
GRAIN SIZE ANALYSIS OF SOILS (ASTM D 422)

Client: HERWIT Eng.
 Project: Manteca WWTP
 Tested By: G. Minerales
 Reviewed By: J. Auser

Project No.: G15-133-10L
 Boring Number: B-1
 Sample ID: 2
 Sample Depth: 10'
 Date Tested: 8/10/2015

SIEVE ANALYSIS

HYDROMETER ANALYSIS



PARTICLE SIZE DISTRIBUTION

Cobbles	Gravel	Sand			Silt (non-plastic)	Clay (Plastic)
		Coarse	Medium	Fine		
-		4	66	27		

Specific Gravity, G_s - 2.65

Description of Soil: Poorly Graded Sand (SP)



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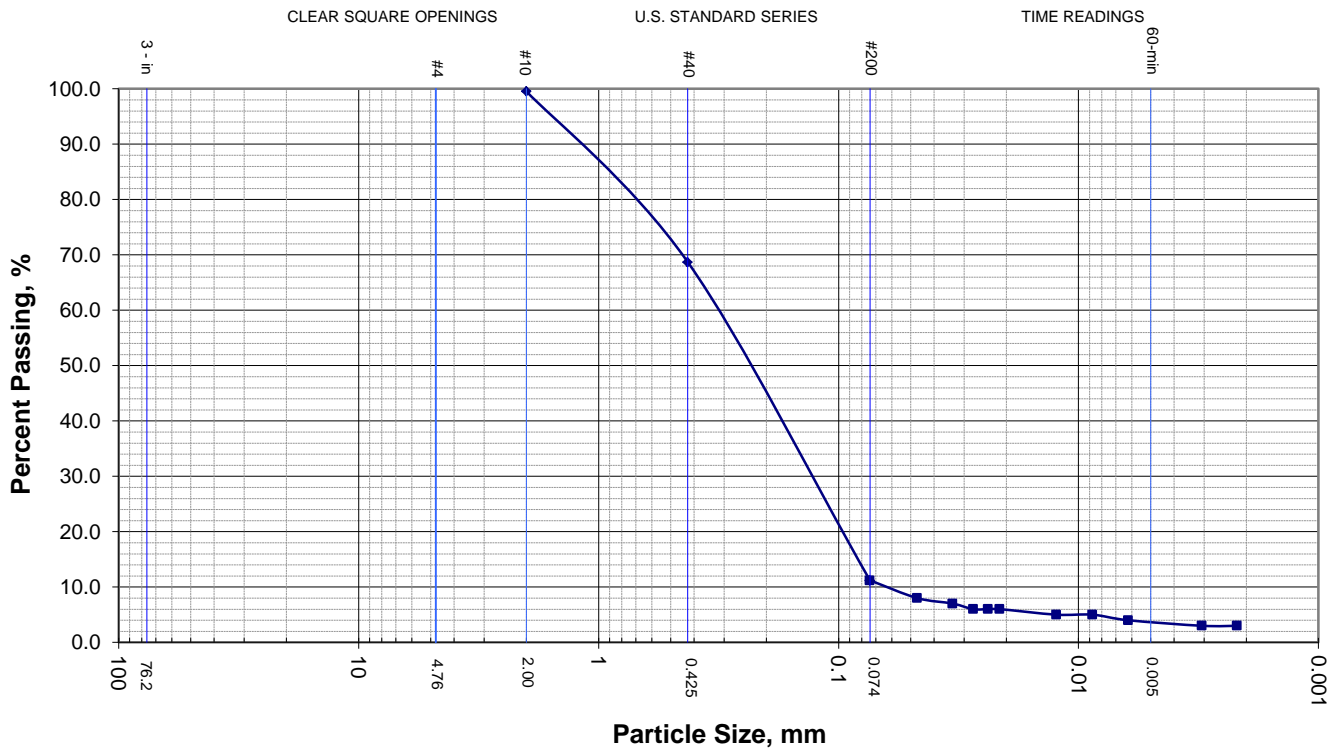
GRAIN SIZE ANALYSIS OF SOILS
(ASTM D 422)

Client: HERWIT Eng.
 Project: Manteca WWTP
 Tested By: G. Minerales
 Reviewed By: J. Auser

Project No.: G15 - 133 - 10L
 Boring Number: B - 2
 Sample ID: 1B
 Sample Depth: 5.5'
 Date Tested: 8/31/2015

SIEVE ANALYSIS

HYDROMETER ANALYSIS



PARTICLE SIZE DISTRIBUTION

Cobbles	Gravel	Sand			Silt (non-plastic)	Clay (Plastic)
		Coarse	Medium	Fine		
-			31	58	7	4

Specific Gravity, G_s - 2.65

Description of Soil: Poorly Graded Sand with Silt (SP-SM)



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 Fax: (925) 315-3152

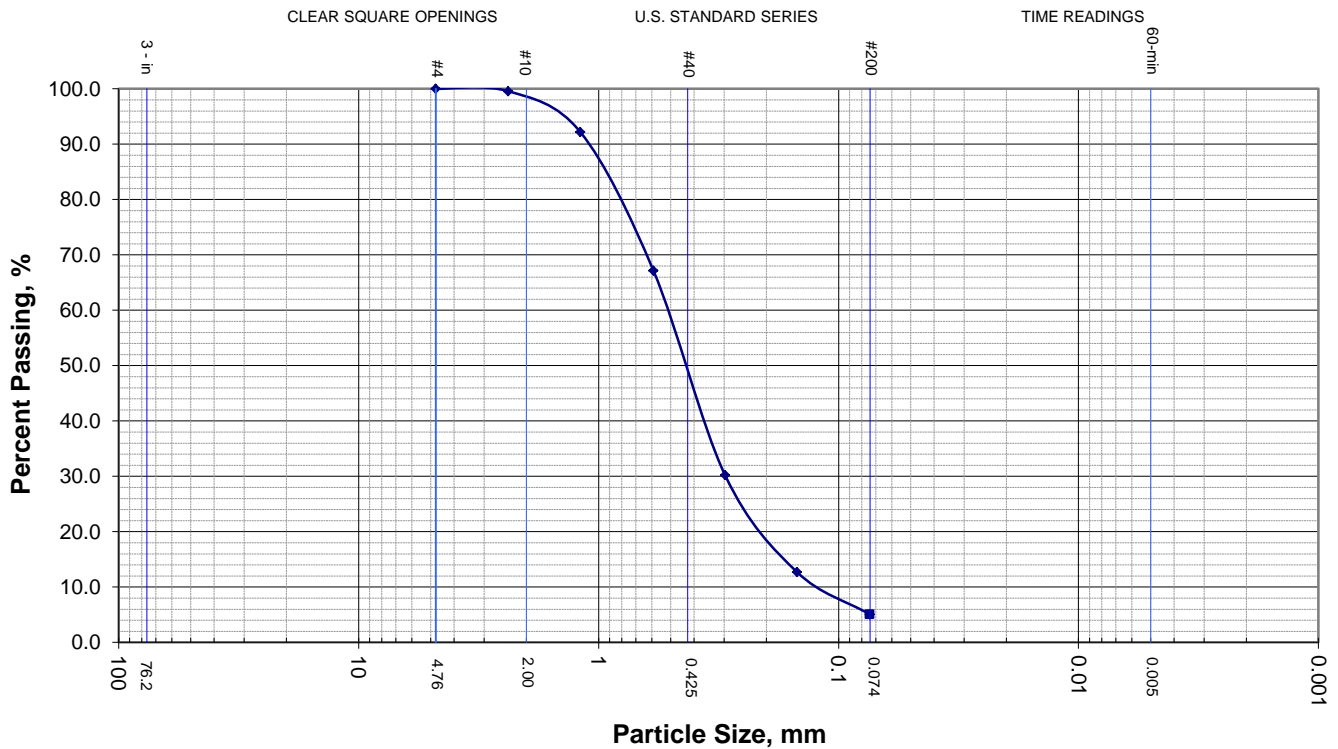
**GRAIN SIZE ANALYSIS OF SOILS
 (ASTM D 422)**

Client: HERWIT Eng.
 Project: Manteca WWTP
 Tested By: G. Minerales
 Reviewed By: J. Auser

Project No.: G15-133-10L
 Boring Number: B-2
 Sample ID: 4
 Sample Depth: 20'
 Date Tested: 8/10/2015

SIEVE ANALYSIS

HYDROMETER ANALYSIS



PARTICLE SIZE DISTRIBUTION

Cobbles	Gravel	Sand			Silt (non-plastic)	Clay (Plastic)
		Coarse	Medium	Fine		
-		2	48	45		

Specific Gravity, G_s - 2.65

Description of Soil: Poorly Graded Sand (SP)



324 Earhart Way
 Livermore, CA 94551
 Phone: (925) 315-3151
 Fax: (925) 315-3152

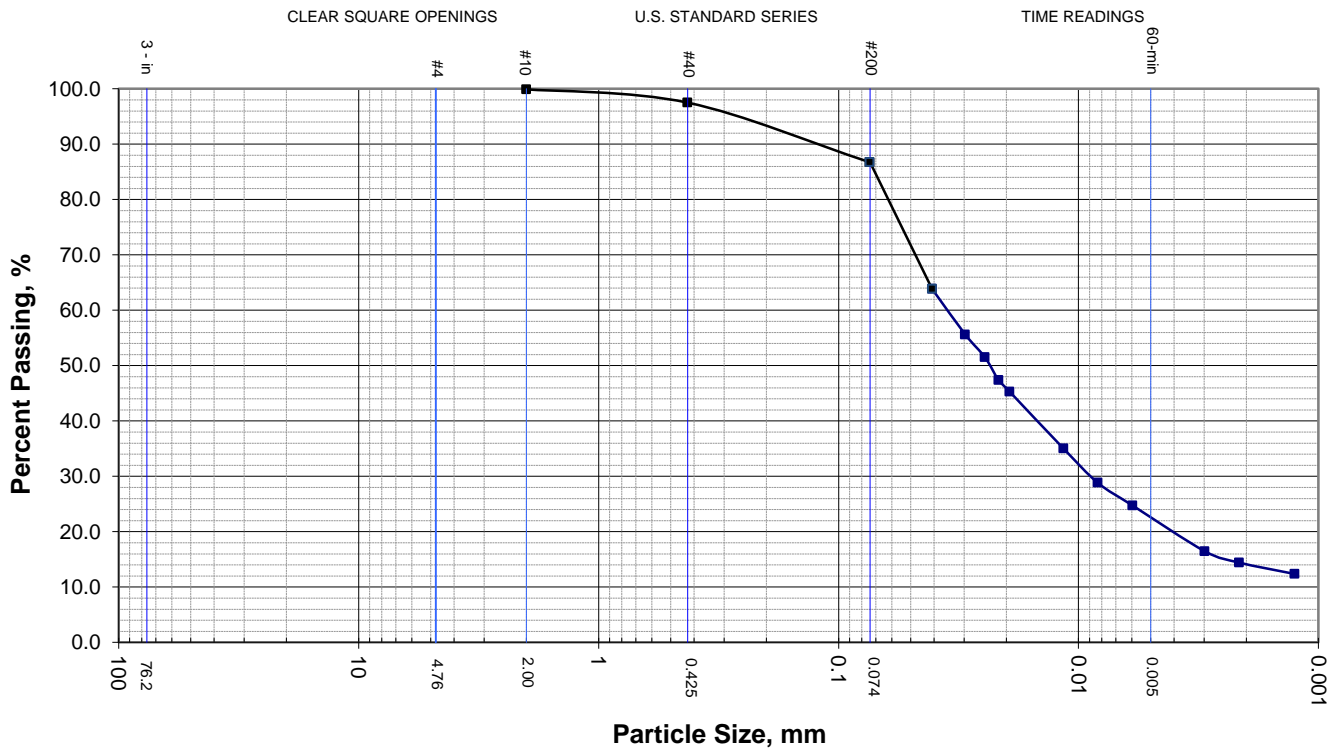
GRAIN SIZE ANALYSIS OF SOILS
(ASTM D 422)

Client: HERWIT Eng.
 Project: Manteca WWTP
 Tested By: G. Minerales
 Reviewed By: J. Auser

Project No.: G15 - 133 - 10L
 Boring Number: B - 2
 Sample ID: 5B
 Sample Depth: 25.5'
 Date Tested: 8/26/2015

SIEVE ANALYSIS

HYDROMETER ANALYSIS



PARTICLE SIZE DISTRIBUTION

Cobbles	Gravel	Sand			Silt (non-plastic)	Clay (Plastic)
		Coarse	Medium	Fine		
-			2	11	64	23

Specific Gravity, G_s - 2.65

Description of Soil: Lean Clay (CL)



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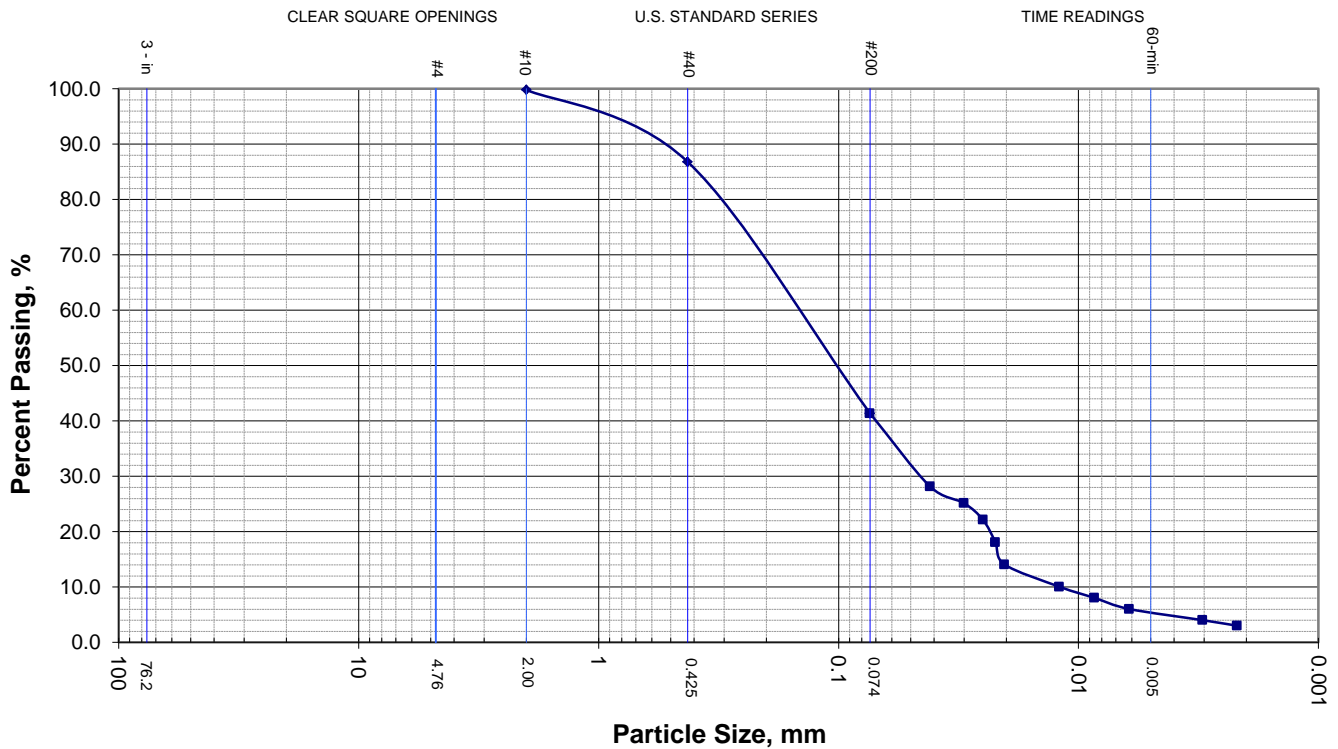
**GRAIN SIZE ANALYSIS OF SOILS
 (ASTM D 422)**

Client: HERWIT Eng.
 Project: Manteca WWTP
 Tested By: G. Minerales
 Reviewed By: J. Auser

Project No.: G15 - 133 - 10L
 Boring Number: B - 3
 Sample ID: 3B
 Sample Depth: 10.5'
 Date Tested: 8/31/2015

SIEVE ANALYSIS

HYDROMETER ANALYSIS



PARTICLE SIZE DISTRIBUTION

Cobbles	Gravel	Sand			Silt (non-plastic)	Clay (Plastic)
		Coarse	Medium	Fine		
-			13	45	35	6

Specific Gravity, G_s - 2.65

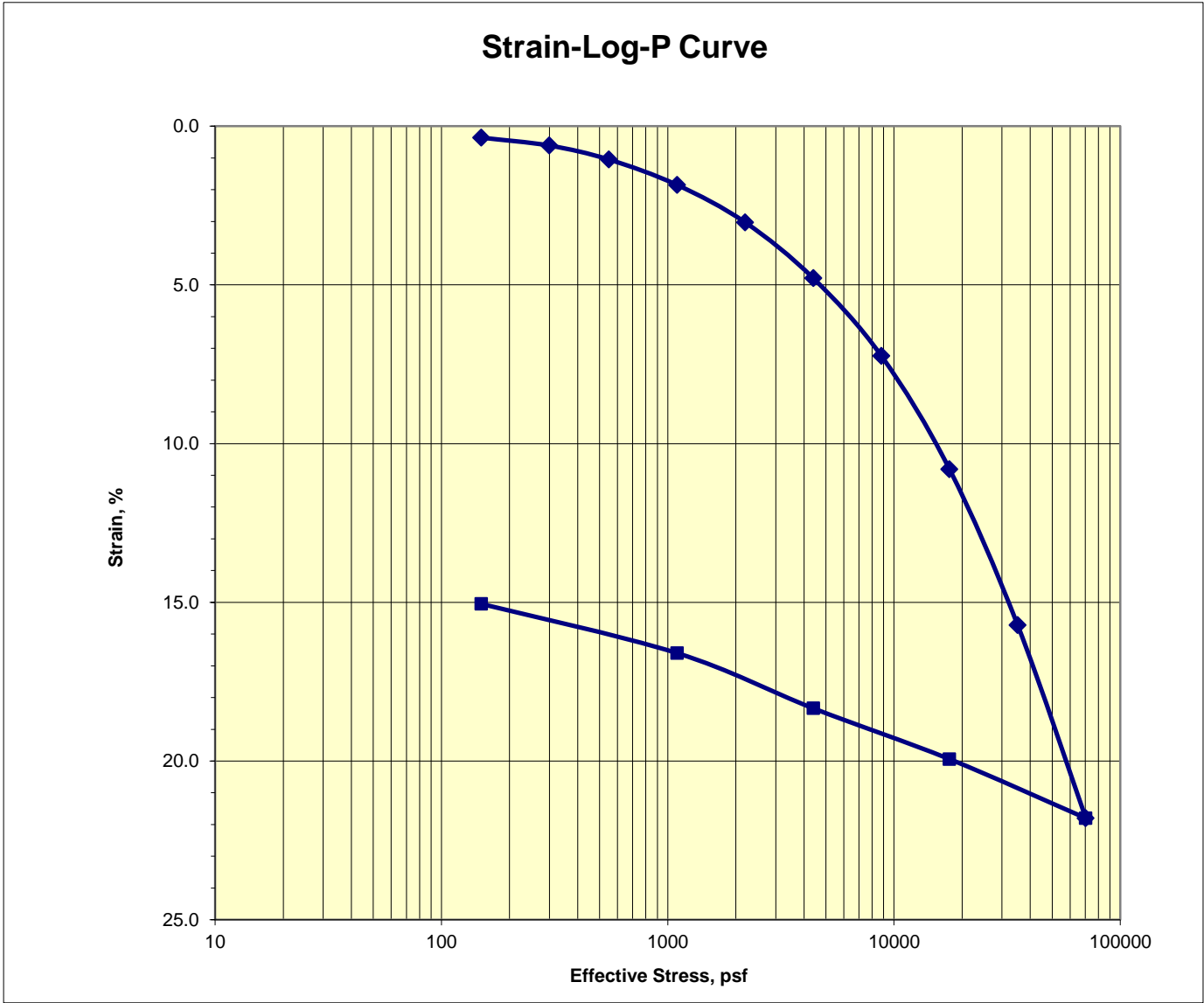
Description of Soil: Silty Sand (SM)



Consolidation Test

ASTM D2435

Job No.: 664-048	Boring: B-1	Run By: MD
Client: BSK Associates	Sample:	Reduced: PJ
Project: Maneca WWTP - G15-133-10L	Depth, ft.: 26	Checked: PJ/DC
Soil Type: Grayish Brown SILT w/ Sand (slightly plastic)		Date: 8/21/2015



Assumed Gs	2.75	Initial	Final
Moisture %:		36.8	27.6
Dry Density, pcf:		82.7	97.6
Void Ratio:		1.075	0.760
% Saturation:		94.2	100.0

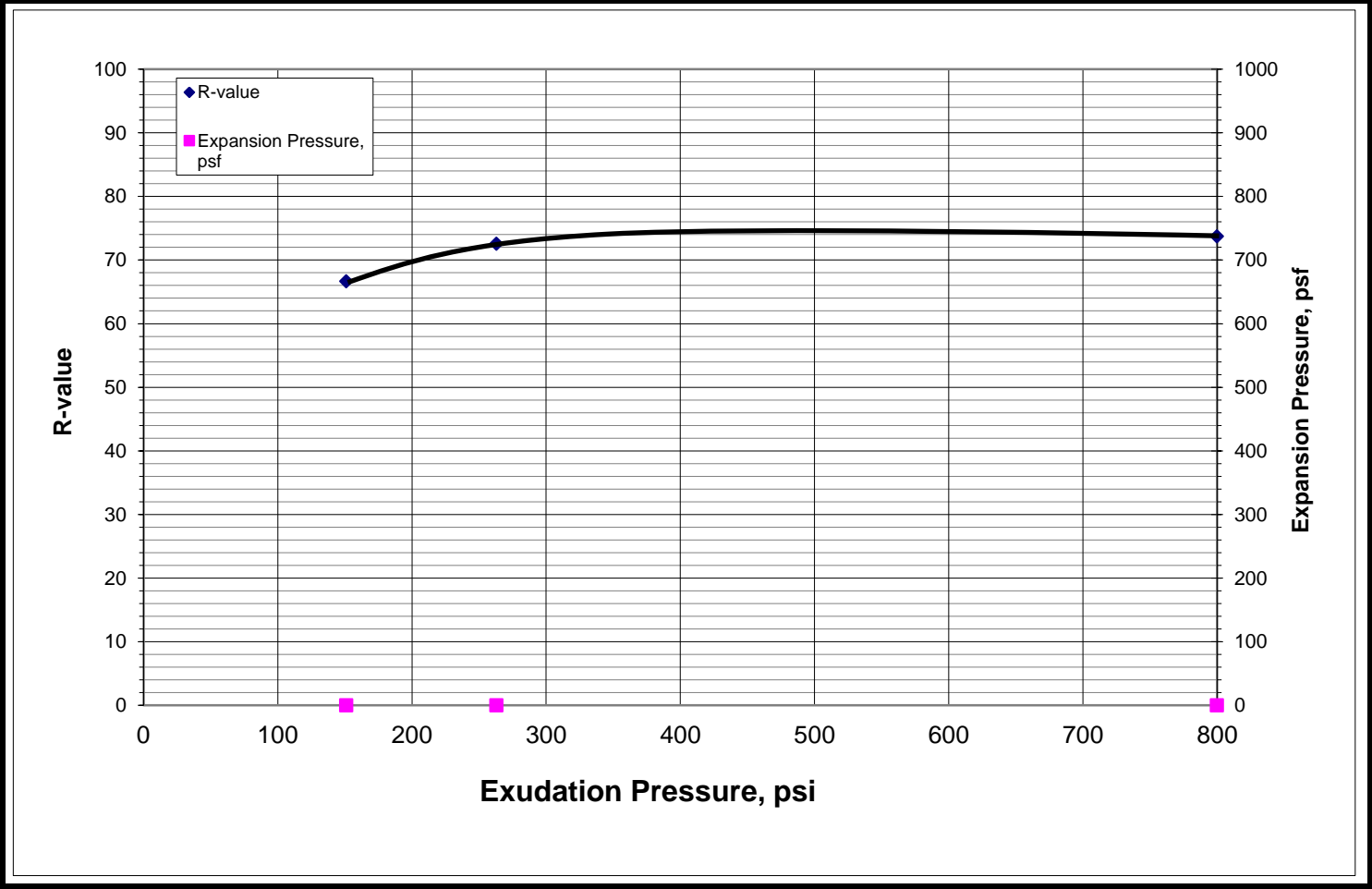
Remarks:



R-value Test Report (Caltrans 301)

Job No.:	664-048	Date:	08/13/15	Initial Moisture,	3.9%
Client:	BSK Associates	Tested	MD	R-value by	73
Project:	Manteca WWTP - G15-133-10L	Reduced	RU	Stabilometer	
Sample	B-2/B-4 @ 0-2'	Checked	DC	Expansion	0 psf
Soil Type:	Dark Brown Silty SAND				

Specimen Number	A	B	C	D	Remarks:
Exudation Pressure, psi	800	263	151		
Prepared Weight, grams	1200	1200	1200		
Final Water Added, grams/cc	61	81	91		
Weight of Soil & Mold, grams	3086	3146	3099		
Weight of Mold, grams	2092	2099	2114		
Height After Compaction, in.	2.41	2.52	2.43		
Moisture Content, %	9.2	10.9	11.8		
Dry Density, pcf	114.4	113.4	109.8		
Expansion Pressure, psf	0.0	0.0	0.0		
Stabilometer @ 1000					
Stabilometer @ 2000	26	28	33		
Turns Displacement	4.26	4.46	4.5		
R-value	74	73	67		





1100 Willow Pass Court, Suite A
Concord, CA 94520-1006

925 462 2771 Fax. 925 462 2775

www.cercoanalytical.com

12 August, 2015

Job No. 1508028
Cust. No.12667

Mr. Cristiano Melo
BSK Associates Engineers & Laboratories
324 Earhart Way
Livermore, CA 94551

Subject: Project No.: G15-133-10L
Project Name: Manteca WWTP
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Melo:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on August 05, 2015. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, Sample No.001 is classified as “moderately corrosive” and Sample No.002 is classified as “mildly 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 concentrations ranged from none detected to 15 mg/kg and are determined to be insufficient to attack steel embedded in a concrete mortar coating

The sulfate ion concentrations reflect none detected with a reporting limit of 15 mg/kg


The pH of the soils were 7.06 and 7.51, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials were 450-mV and 480-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 require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.


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

JDH/jdl
Enclosure

APPENDIX C

EXHIBIT 1 – SUMMARY OF COMPACTION RECOMMENDATIONS

EXHIBIT 1

SUMMARY OF COMPACTION RECOMMENDATIONS

Area	Compaction Recommendations (See Notes 1, 2, 3, 4)
Subgrade Preparation and Placement of General Engineered Fill, Including Imported Fill, Pads 1 through 4, Mat Foundations, Digester Control Building Pad, and Bottom of the Below-Grade Detention Ponds	Compact upper 12 inches of subgrade and entire depth of fill to a minimum of 90 percent compaction at near optimum moisture content for granular soils and to a minimum of 90 percent compaction at a minimum of 2 percent over optimum moisture content for clayey soils.
Trenches and Excavations ^{5, 6}	Compact trench backfill to a minimum of 90 percent compaction at near optimum moisture content for granular soils and to a minimum of 90 percent compaction at a minimum of 2 percent over optimum moisture content for clayey soils.
Pavements, including mat foundations exposed to vehicular/truck traffic	Compact upper 12 inches of subgrade to at least 95 percent compaction at near optimum moisture content. Compact aggregate baserock to at least 95 percent compaction at near optimum moisture content.

Notes:

- (1) Depths are below finished subgrade elevation.
- (2) All compaction requirements refer to relative compaction as a percentage of the laboratory standard described by ASTM D 1557.
- (3) Fill material should be compacted in lifts not exceeding 8 inches in loose thickness.
- (4) All subgrades should be firm and stable.
- (5) In landscaping areas only, the percent compaction in excavations may be reduced to 85 percent.
- (6) Where backfills are greater than 7 feet in depth below finish grade, the entire backfill should be compacted to a minimum of 95 percent compaction.

APPENDIX D

SUBSURFACE DATA FROM PREVIOUS INVESTIGATIONS

- Neil O. Anderson Associates (2014), Preliminary Geotechnical Investigation, Proposed Solar PV Arrays, City of Manteca Wastewater Quality Control Facility, 2450 W. Yosemite Avenue, Manteca California, dated December 29, 2014 (File No. LGE140037).
- Kleinfelder (2005), Geotechnical Services Report, Schedule D Project, Manteca Wastewater Treatment Plant, Manteca, California, dated May 12, 2005 (File No. 50876.G01).

PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED SOLAR PV ARRAYS
CITY OF MANTECA WASTEWATER QUALITY CONTROL FACILITY
2450 W. YOSEMITE AVENUE
MANTECA, CALIFORNIA

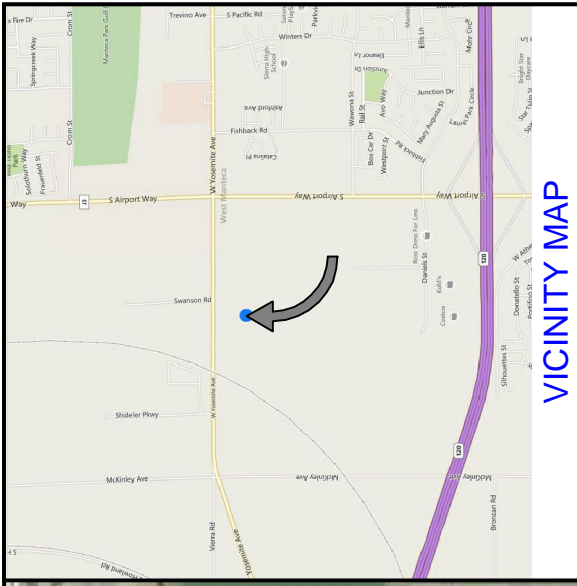
REPORT PREPARED FOR:
CITY OF MANTECA

OUR PROJECT NUMBER: LGE140037

September 12, 2014
Revised December 29, 2014

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VICINITY MAP




PROPOSED EXTENTS OF SOLAR ARRAYS

W. YOSEMITE AVENUE

B-2

B-1

LEGEND

 Approximate Boring Location

DATE:	09/12/14
JOB NUMBER:	LGE140037
SCALE:	NONE
DRAWN BY:	E. NUÑEZ
CHECKED BY:	R. KING
PLATE:	PLATE 1

BORING LOCATION MAP
MANTECA WQCF SOLAR ARRAYS
 2450 W. YOSEMITE AVENUE
 MANTECA, CALIFORNIA

GEOTECHNICAL
 ENVIRONMENTAL
 GROUNDWATER
 INSPECTIONS & TESTING
 LABORATORY SERVICES
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 POST TENSION DESIGN
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NEIL O. ANDERSON
 A N D A S S O C I A T E S
 902 INDUSTRIAL WAY
 LODI, CALIFORNIA
 PHONE: (209) 367-3701
 FAX: (209) 333-8303

NOTE: BORING LOCATIONS ARE APPROXIMATE.
 BASE PLAN PROVIDED BY GOOGLE EARTH

BORING LOG NO. B-1

PROJECT: Manteca WQCF Solar PV Arrays

CLIENT: City of Manteca
Manteca, California

SITE: 2450 W. Yosemite Avenue
Manteca, CA

GRAPHIC LOG	LOCATION See PLATE 1	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)
	DEPTH						
●●●●●	SILTY SAND (SM) , fine grained, brown, loose	5		◆	6-6-8		
●●●●●		10	▽ ▽	X	4-4-4 N=8	20	103
●●●●●	13.0 POORLY GRADED SAND (SP) , fine to coarse grained, brown, medium dense	15		X	1-4-15 N=19	23	106
●●●●●	20.0	20		X	6-8-9 N=17	24	103
●●●●●	20.5 SILT (ML) , brown						
●●●●●	SILTY SAND (SM) , fine to coarse grained, brown, medium dense						
●●●●●	21.5 Boring Terminated at 21.5 Feet						

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic SPT Hammer

Advancement Method:
6 inch hollow stem auger

See report for description of field procedures, laboratory procedures, and additional data (if any).

Notes:

Abandonment Method:
Borings grouted upon completion

See PLATE 4 for explanation of symbols and abbreviations.

WATER LEVEL OBSERVATIONS
▽ While drilling
▽ At completion of drilling



Boring Started: 8/25/2014

Boring Completed: 8/25/2014

Drill Rig: CME-75

Driller: NOA

Project No.: LGE140037

PLATE 2

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL. LGE140037 MANTECA SOLAR ARRAY.GPJ. TEMPLATE UPDATE 3-31-14.GPJ. 9/12/14

BORING LOG NO. B-2

PROJECT: Manteca WQCF Solar PV Arrays


CLIENT: City of Manteca
Manteca, California

SITE: 2450 W. Yosemite Avenue
Manteca, CA

GRAPHIC LOG	LOCATION See PLATE 1	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)
	DEPTH						
	SILTY SAND (SM) , fine to medium grained, brown, loose			◆	4-4-3	8	99
		5		◆	4-4-5	9	98
			▽				
	10.0		▽				
	POORLY GRADED SAND (SP) , medium to coarse grained, brown, loose			◆	3-5-6	11	92
		10					
	15.0						
	Boring Terminated at 15 Feet	15					

Stratification lines are approximate. In-situ, the transition may be gradual.












Hammer Type: Automatic SPT Hammer

Advancement Method: 6 inch hollow stem auger	See report for description of field procedures, laboratory procedures, and additional data (if any).	Notes: Heaving at 15 feet depth. Pulled auger to dislodge plug.
Abandonment Method: Borings grouted upon completion	See PLATE 4 for explanation of symbols and abbreviations.	
WATER LEVEL OBSERVATIONS		
▽ While drilling		Boring Started: 8/21/2014
▽ At completion of drilling		Boring Completed: 8/21/2014
		Drill Rig: CME-75
		Driller: NOA
		Project No.: LGE140037
		PLATE 3

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL. LGE140037 MANTECA SOLAR ARRAY.GPJ. TEMPLATE UPDATE 3-31-14.GPJ. 9/12/14

GENERAL NOTES

DESCRIPTION OF SYMBOLS AND ABBREVIATIONS

SAMPLING			WATER LEVEL		Water Initially Encountered	FIELD TESTS	(HP) Hand Penetrometer
	Auger	Split Spoon			Water Level After a Specified Period of Time		(T) Torvane
					Water Level After a Specified Period of Time		(b/f) Standard Penetration Test (blows per foot)
	Modified Cal	Macro Core		Water levels indicated on the soil boring logs are the levels measured in the borehole at the times indicated. Groundwater level variations will occur over time. In low permeability soils, accurate determination of groundwater levels is not possible with short term water level observations.			(PID) Photo-Ionization Detector
							(OVA) Organic Vapor Analyzer
Ring Sampler	Rock Core						
							
Grab Sample	No Recovery						

DESCRIPTIVE SOIL CLASSIFICATION

Soil classification is based on the Unified Soil Classification System. Coarse Grained Soils have more than 50% of their dry weight retained on a #200 sieve; their principal descriptors are: boulders, cobbles, gravel or sand. Fine Grained Soils have less than 50% of their dry weight retained on a #200 sieve; they are principally described as clays if they are plastic, and silts if they are slightly plastic or non-plastic. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size. In addition to gradation, coarse-grained soils are defined on the basis of their in-place relative density and fine-grained soils on the basis of their consistency.

LOCATION AND ELEVATION NOTES

Unless otherwise noted, Latitude and Longitude are approximately determined using a hand-held GPS device. The accuracy of such devices is variable. Surface elevation data annotated with +/- indicates that no actual topographical survey was conducted to confirm the surface elevation. Instead, the surface elevation was approximately determined from topographic maps of the area.

STRENGTH TERMS	RELATIVE DENSITY OF COARSE-GRAINED SOILS <small>(More than 50% retained on No. 200 sieve.) Density determined by Standard Penetration Resistance Includes gravels, sands and silts.</small>			CONSISTENCY OF FINE-GRAINED SOILS <small>(50% or more passing the No. 200 sieve.) Consistency determined by laboratory shear strength testing, field visual-manual procedures or standard penetration resistance</small>			
	Descriptive Term (Density)	Standard Penetration or N-Value Blows/Ft.	Ring Sampler Blows/Ft.	Descriptive Term (Consistency)	Unconfined Compressive Strength, Qu, psf	Standard Penetration or N-Value Blows/Ft.	Ring Sampler Blows/Ft.
	Very Loose	0 - 3	0 - 6	Very Soft	less than 500	0 - 1	< 3
	Loose	4 - 9	7 - 18	Soft	500 to 1,000	2 - 4	3 - 4
	Medium Dense	10 - 29	19 - 58	Medium-Stiff	1,000 to 2,000	4 - 8	5 - 9
	Dense	30 - 50	59 - 98	Stiff	2,000 to 4,000	8 - 15	10 - 18
	Very Dense	> 50	≥ 99	Very Stiff	4,000 to 8,000	15 - 30	19 - 42
				Hard	> 8,000	> 30	> 42

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 15
With	15 - 29
Modifier	> 30

GRAIN SIZE TERMINOLOGY

Major Component of Sample	Particle Size
Boulders	Over 12 in. (300 mm)
Cobbles	12 in. to 3 in. (300mm to 75mm)
Gravel	3 in. to #4 sieve (75mm to 4.75 mm)
Sand	#4 to #200 sieve (4.75mm to 0.075mm)
Silt or Clay	Passing #200 sieve (0.075mm)

RELATIVE PROPORTIONS OF FINES

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 5
With	5 - 12
Modifier	> 12

PLASTICITY DESCRIPTION

Term	Plasticity Index
Non-plastic	0
Low	1 - 10
Medium	11 - 30
High	> 30

**GEOTECHNICAL SERVICES REPORT
SCHEDULE D PROJECT
MANTECA WASTEWATER TREATMENT PLANT
MANTECA, CALIFORNIA**

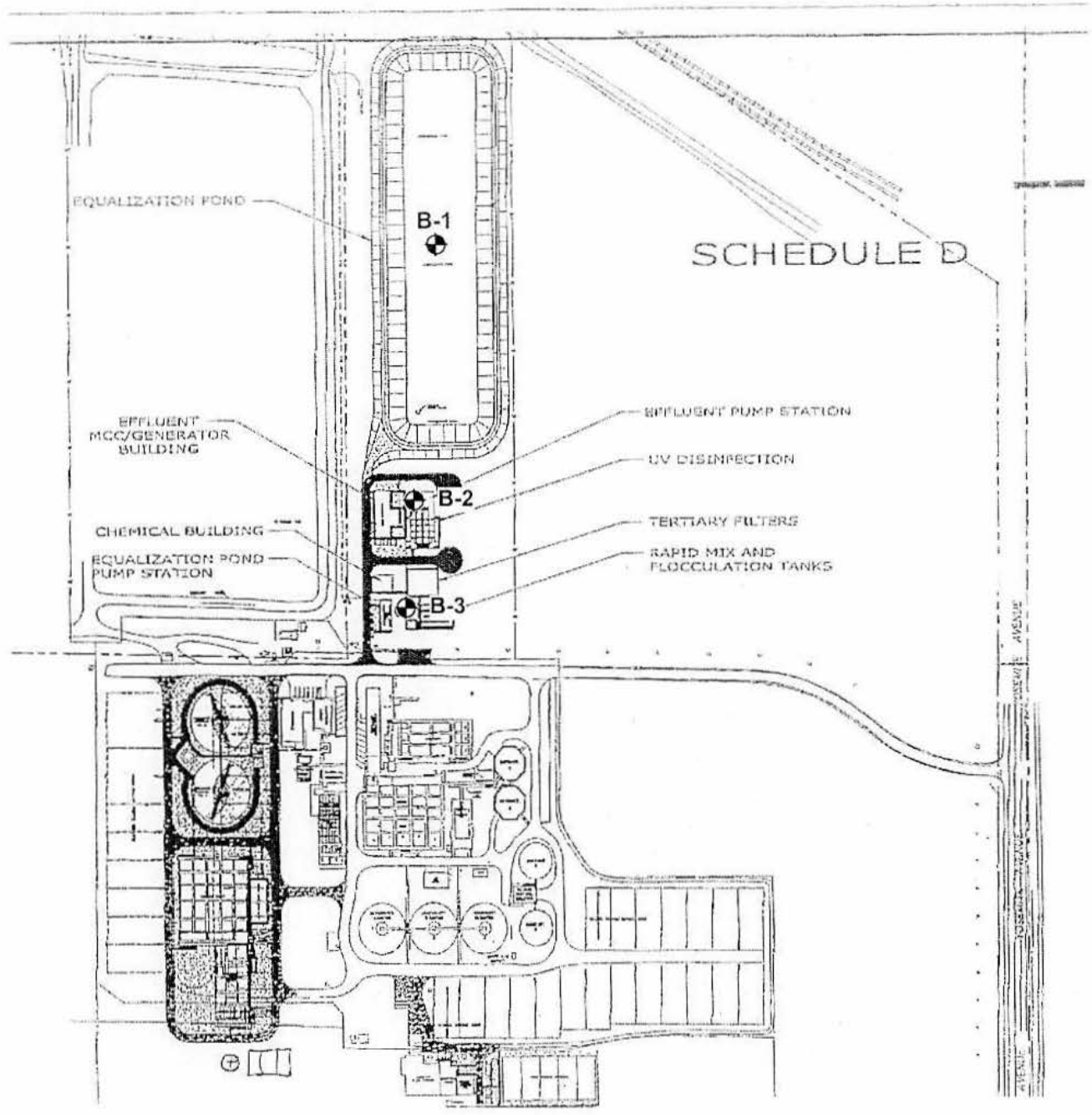
PREPARED FOR: NOLTE ASSOCIATES, INC.
1750 CREEKSIDE OAKS DRIVE, SUITE 200
SACRAMENTO, CA 95833

BY: KLEINFELDER, INC.
2825 E. MYRTLE STREET
STOCKTON, CA 95205

DATE: MAY 12, 2005

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 DENOTES NUMBERS AND APPROXIMATE LOCATIONS OF BORINGS DRILLED FOR THIS INVESTIGATION


 NOT TO SCALE

 KLEINFELDER

SITE PLAN AND VICINTIY MAP
 MANTECA WTP SCHEDULE D EXPANSION
 YOSEMITE AVENUE
 MANTECA, CALIFORNIA

PLATE No.

1

DATE PRODUCED: 11/18/2004	DATE REVISED:
PROJ. NO.: 50876.G01	FILENAME: STG4D901.FH9





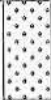










**APPENDIX
LOGS OF BORINGS AND
SUMMARY OF LABORATORY TESTS**

LIST OF ATTACHMENTS

The following plates are attached and complete this appendix.

	<u>Plate</u>
Unified Soil Classification System	A-1
Log Key	A-2
Logs of Borings B-1 through B-3	A-3, A-4, and A-5
Summary of Laboratory Tests	A-6
Direct Shear Test.....	A-7
Grain Size Distribution.....	A-8

UNIFIED SOIL CLASSIFICATION SYSTEM

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

K:\USC'S ST04G260.GPJ 11/19/04



UNIFIED SOIL CLASSIFICATION SYSTEM
 MANTECA WTP SCHEDULE D EXPANSION
 YOSEMITE AVENUE
 MANTECA, CALIFORNIA

PLATE

A-1

 Drafted By: GDG
 Date: 11/19/2004

 Project No.: 50876.G01
 File Number: ST04G260

LOG SYMBOLS

	BULK / BAG SAMPLE	-4	PERCENT FINER THAN THE NO. 4 SIEVE (ASTM Test Method C 136)
	MODIFIED CALIFORNIA SAMPLER (2-1/2 inch outside diameter)	-200	PERCENT FINER THAN THE NO. 200 SIEVE (ASTM Test Method C 117)
	CALIFORNIA SAMPLER (3 inch outside diameter)	LL	LIQUID LIMIT (ASTM Test Method D 4318)
	STANDARD PENETRATION SPLIT SPOON SAMPLER (2 inch outside diameter)	PI	PLASTICITY INDEX (ASTM Test Method D 4318)
	CONTINUOUS CORE	TXCU	CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION (EM 1110-1-1906)
	SHELBY TUBE	EI	EXPANSION INDEX (UBC STANDARD 18-2)
	ROCK CORE	COL	COLLAPSE POTENTIAL
	WATER LEVEL (level where first encountered)	UC	UNCONFINED COMPRESSION (ASTM Test Method D 2166)
	WATER LEVEL (level after completion)		
	SEEPAGE	MC	MOISTURE CONTENT (ASTM Test Method D 2216)

GENERAL NOTES

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



LOG KEY
 MANTECA WTP SCHEDULE D EXPANSION
 YOSEMITE AVENUE
 MANTECA, CALIFORNIA

PLATE

A-2

Drafted By: GDG
 Date: 11/19/2004

Project No.: 50876.G01
 File Number: STO4G260

Surface Conditions: Open disked field, relatively flat, north of storage pump

Groundwater: Groundwater encountered at a depth of approximately 10 feet below existing site grade.

Method: Hollowstem Auger

Equipment: CME 85

Date Completed: 10/27/2004

Logged By: BC

Total Depth: 16.5 feet

Boring Diameter: 8 Inches

Depth (feet)	Sample Type	Sample No.	FIELD				LABORATORY				Lithography	DESCRIPTION
			Blows/ft	Pocket Penetrometer (tsf)	Dry Density (pcf)	Moisture Content (%)	Liquid Limit	Plasticity Index	Passing #4 Sieve (%)	Passing #200 Sieve (%)		
5		1-2-1	6									(SM) SILTY SAND - Brown, moist, loose, slightly silty, fine grained
5-10		1-5-2	10			7						(SP/SM) SAND WITH SILT - Yellow-brown, moist, medium dense, fine grained
10		1-10-1	13			19						Grades brown, wet (SP) SAND - Brown, fine to coarse grained, medium dense, wet (SM) SILTY SAND - Dark brown, wet, fine grained
15		1-15-2	23	2.25								(SP) SAND - Gray, wet, medium dense, fine to coarse grained (ML) SANDY CLAYEY SILT - Gray, wet, very stiff, low plasticity, fine grained
16.5												Boring completed at a depth of approximately 16.5 feet below existing site grade.



LOG OF BORING B-1
 MANTECA WTP SCHEDULE D EXPANSION
 YOSEMITE AVENUE
 MANTECA, CALIFORNIA

PLATE
 1 of 1
 A-3

Drafted By: GDG
 Date: 12/1/2004
 Project No.: 50876.G01
 File Number: STO4G260

SAC:01044 ST04G260.GPJ 12/1/04

Surface Conditions: Relatively flat, open, disked field, north of storage pond

Date Completed: 10/27/2004

Groundwater: Groundwater encountered at a depth of approximately 12 feet below existing site grade.

Logged By: BC

Method: Hollowstem Auger

Total Depth: 21.5 feet

Equipment: CME 85

Boring Diameter: 8 Inches

Depth (feet)	FIELD				LABORATORY				Lithography	DESCRIPTION		
	Sample Type	Sample No.	Blows/ft	Pocket Penetrometer (tsf)	Dry Density (pcf)	Moisture Content (%)	Liquid Limit	Plasticity Index			Passing #4 Sieve (%)	Passing #200 Sieve (%)
5	2-2-1	7			10				12			(SM) SILTY SAND - Brown, moist, loose, fine grained
5-6	2-5-1	6										(SP/SM) SAND WITH SILT - Orange-brown, moist, loose, fine grained
10	2-10-1	10			37				12			(ML) SANDY SILT - Gray, moist, stiff, fine grained
10-11												(SP/SM) SAND WITH SILT - Gray-brown, moist, medium dense, fine to coarse grained
15	2-15-1	15						100	1	Gradation		(SP) SAND - Gray-brown, wet, medium dense, fine to coarse grained
15-16												(SM) SILTY SAND - Brown, wet, fine to medium grained
20	2-20-1	22										(SP) SAND - Gray, wet, loose, fine to coarse grained
20-21												(ML) SANDY CLAYEY SILT - Gray-brown, wet, very stiff, non-plastic, fine grained
21.5												Boring completed at a depth of approximately 21.5 feet below existing site grade.



LOG OF BORING B-2
 MANTECA WTP SCHEDULE D EXPANSION
 YOSEMITE AVENUE
 MANTECA, CALIFORNIA

PLATE
 1 of 1
 A-4

Drafted By: GDG
 Date: 12/1/2004

Project No.: 50576.G01
 File Number: ST04G260

S:\04\2104 ST04G260.GPJ 12/1/04

Quantity: 1 of 1

Surface Conditions: Relatively flat, disked field, north of storage pond

Groundwater: Groundwater encountered at a depth of approximately 18 feet below existing site grade.

Method: Hollowstem Auger

Equipment: CME 85

Date Completed: 10/27/2004

Logged By: BC

Total Depth: 26.5 feet

Boring Diameter: 8 Inches

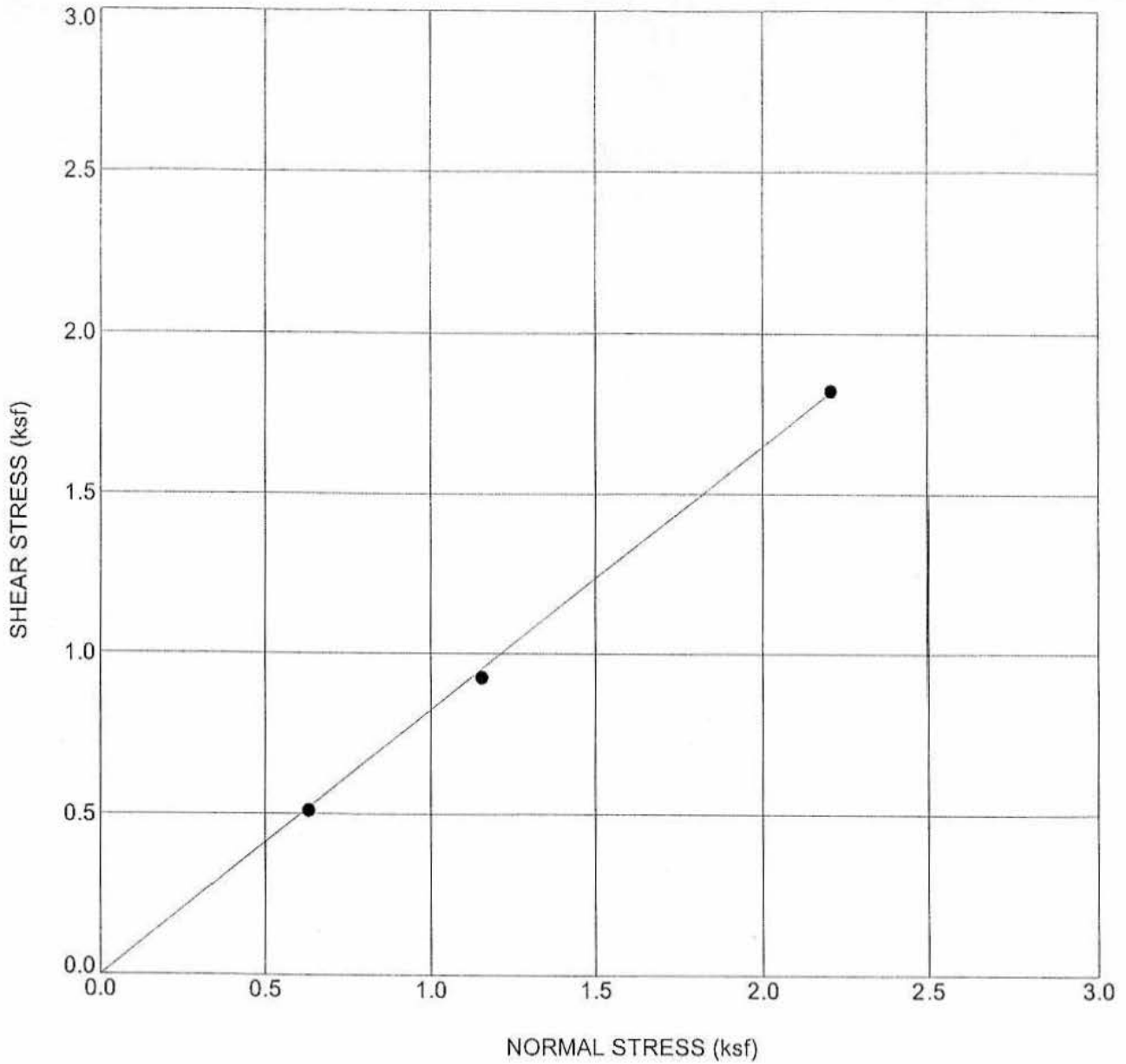
Depth (feet)	Sample Type	Sample No.	FIELD				LABORATORY				Lithography	DESCRIPTION
			Blows/ft	Pocket Penetrometer (tsf)	Dry Density (pcf)	Moisture Content (%)	Liquid Limit	Plasticity Index	Passing #4 Sieve (%)	Passing #200 Sieve (%)		
5		3-2-1	8									(SM) SILTY SAND - Dark brown, moist, loose, fine grained
5		3-5-1	3									Grades very loose
10		3-10-1	14			13						(SP/SM) SAND WITH SILT - Brown, loose, fine grained
15		3-15-1	13		101	18		100	2	Gradation Dia. 39 Deg.		(SM) SILTY SAND - Gray-yellow-brown, with silt layers, medium dense
15		3-15-1	13		101	18		100	2	Gradation Dia. 39 Deg.		(SP) SAND - Gray, fine to coarse grained, wet, loose
20		3-20-1	11			22						(ML) SANDY CLAYEY SILT - Gray-brown, wet, stiff, fine to medium grained
20		3-20-1	11			22						(SP) SAND - Gray, wet, loose, fine to medium grained
25		3-25-1	10									(ML) SANDY CLAYEY SILT - Gray-brown, moist, stiff, low plasticity, fine grained
25		3-25-1	10									(SP) SAND - Gray, wet, loose, fine to medium grained
25		3-25-1	10									(ML) SANDY CLAYEY SILT - Gray-brown, wet, stiff, low plasticity, fine grained
26.5												Boring completed at a depth of approximately 26.5 feet below existing site grade.



LOG OF BORING B-3
 MANTECA WTP SCHEDULE D EXPANSION
 YOSEMITE AVENUE
 MANTECA, CALIFORNIA

PLATE
 1 of 1
 A-5

SAC 2004 ST04G260.GPJ 12/1/04
 Drafted By: GEG
 Date: 12/1/2004
 Project No.: 50870.G01
 File Number: ST04G260



SOURCE: B-3
 DEPTH: 15 ft
 SOIL DESCRIPTION:

FRICTION ANGLE = 40 deg
 COHESION = 0 ksf

FINAL DRY DENSITY (pcf)	101.1	101.4	101.3
INITIAL WATER CONTENT (%)	17.1	18.4	17.8
FINAL WATER CONTENT (%)	22.2	22.5	21.6
NORMAL STRESS (ksf)	0.63	1.155	2.206
MAXIMUM SHEAR (ksf)	0.512	0.926	1.824

KA_DIRECT_SHEAR_STO4G260.GPJ 12/1/04



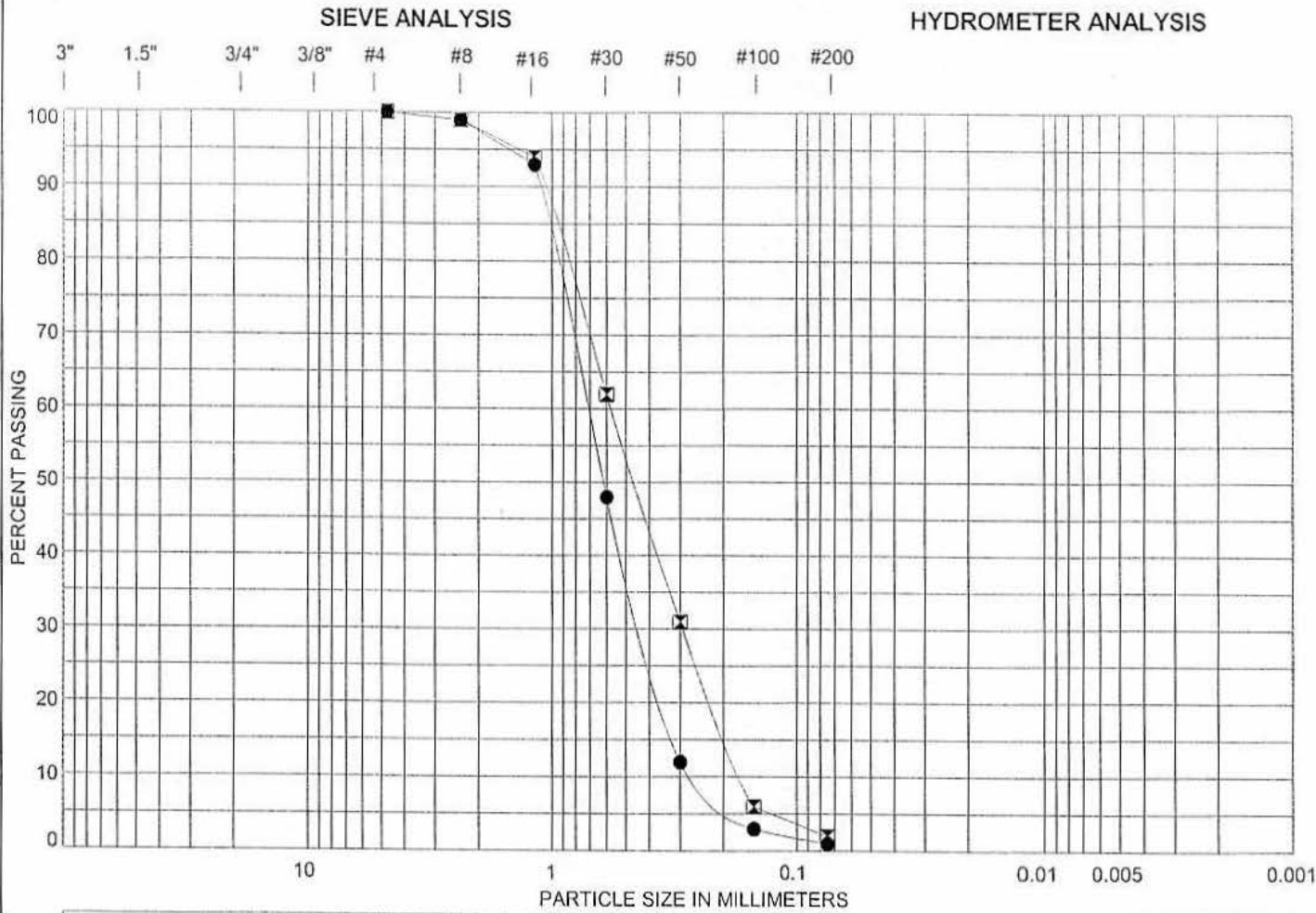
DIRECT SHEAR TEST
 MANTECA WTP SCHEDULE D EXPANSION
 YOSEMITE AVENUE
 MANTECA, CALIFORNIA

PLATE

A-7

Drafted By: GDG
 Date: 12/1/2004

Project No.: 50876.G01
 File Number: STO4G260



GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

LEGEND:	SOURCE	DEPTH (ft)	GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)	DESCRIPTION
●	B-2	15.0	0.0	99.0			POORLY GRADED SAND(SP)
☒	B-3	15.0	0.0	98.0			POORLY GRADED SAND(SP)

KA GRAIN SIZE ST04G260.GPJ 12/1/04



GRAIN SIZE ANALYSIS
 MANTECA WTP SCHEDULE D EXPANSION
 YOSEMITE AVENUE
 MANTECA, CALIFORNIA

PLATE
 1 of 1
A-8