GEOTECHNICAL REPORT
RECYCLED WATER SYSTEM IMPROVEMENTS
LAGUNA COUNTY SANITATION DISTRICT

Prepared for:
SANTA BARBARA COUNTY PUBLIC WORKS

November 14, 2008
November 14, 2008
Project No. 3190.042

Santa Barbara County Public Works Department
Resource Recovery & Waste Management
620 West Foster Road
Santa Maria, CA 93455

Attention: Mr. Martin Wilder

Subject: Geotechnical Report, Recycled Water System, Laguna County Sanitation District Wastewater Treatment Plant, Dutard Road, Santa Barbara County, California

Dear Mr. Wilder:

Fugro is pleased to submit this Geotechnical Report for the Laguna County Sanitation District Wastewater Treatment Plant’s Recycled Water System Improvements. The purpose of this report is to provide geotechnical recommendations for the design of the four 1 million gallon steel tanks, and for the clearwell and booster pump structure. This report was prepared in accordance with our proposal dated June 13, 2008. In addition, we provided sampling of the stockpile material near the site. Our services are being provided under the County’s MSA No. BC98-CN06987, Funding Memo dated June 18, 2008.

This report provides geotechnical recommendations for site preparation and grading, foundation design, buried structures, appurtenant pavements and utilities, and seismic data for use with the current building code and AWWA standard. Field and laboratory data obtained from the geotechnical evaluation are presented in this report.

We appreciate the opportunity to provide our services on this project. Please contact the undersigned if you have questions regarding this report, or require additional information.

Sincerely,
FUGRO WEST, INC.

Christopher L. Lovato, P.E. 60316
Project Engineer

Jonathan D. Blanchard, G.E. 23122
Principal Geotechnical Engineer

Copies: 4 – addressee (1 pdf on CD ROM)
1 – Mr. Steve Tanaka, Wallace Group
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1. SITE AND PROJECT DESCRIPTION

The project generally consists of constructing four new 1MG welded steel storage tanks for recycled water and associated improvements at the existing Laguna County Sanitation District’s wastewater treatment plant on Dutard Road near Orcutt, California. The location of the wastewater treatment plant site relative to nearby streets and geographic landmarks is shown on Plate 1, Vicinity Map. The layout of the proposed tanks and clearwell structure are shown on Plate 2, Field Exploration Plan.

1.1 EXISTING SITE

The existing wastewater treatment plant is a 3.7 million gallon per day (MGD) facility that serves the Orcutt area of Santa Barbara County. The wastewater treatment plant currently treats approximately 2.3 MGD of average daily flow. The western two tank sites are occupied by existing sludge drying beds that are no longer used by the plant. The eastern two tank sites are within an existing agricultural field that was fallow at the time of our field exploration. The existing sludge drying beds consist of 2 to 3 foot high concrete retaining walls with an earthen bottom. The clearwell structure is situated in a grassy area east of an existing one-story pump house building (see Plate 2).

The wastewater treatment plant site is situated within the northwest trending alluvial plain of Orcutt Creek. The topography at the site and within the treatment plant is relatively flat. The existing site grade ranges from about elevation (el.) 138 feet near the existing pump house to about 136 feet in the agricultural field (Wallace 2008).

1.2 PROJECT UNDERSTANDING

The recycled water system improvements are being designed by the Wallace Group (2007, 2008). The primary improvements will consist of four new 1MG tanks to store treated effluent for recycling, a new clearwell and booster pump station, and associated piping between the improvements and the existing effluent pump station (see layout on Plate 2). The District has stated that the tanks will likely be constructed in phases as demand for recycled water requires. The tanks will be welded steel and constructed above-grade. The tanks will be approximately 25 to 30 feet high, 80 feet in diameter, and be supported with a perimeter concrete ring wall footing. The average applied bearing pressure beneath the reservoir will be 2,000 pounds per square foot (psf) or less. The new clearwell will be approximately 25 feet by 25 feet and constructed to depths of approximately 12 feet below the existing ground surface. The capacity of the clearwell will be about 30,000 gallons. A new steel building will be constructed on top of the clearwell structure. Wallace anticipates that the building will be relatively lightly loaded; however, they could not provide specific loading information for the building at the time this report was being prepared for submittal.
2. WORK PERFORMED

2.1 PURPOSE

The purpose of this report is to provide geotechnical opinions and recommendations for the design and construction of the improvements. The main geotechnical considerations that we evaluated for the project are characterization of the soil and groundwater conditions, seismic considerations, site preparation and grading, settlement, and foundation design.

2.2 SCOPE

As a basis for providing the geotechnical recommendations presented in this report, we have performed the following scope of work:

- Performed site visits to observe the general site conditions and meet with the District to identify underground utilities;
- Reviewed selected published geologic maps, geotechnical data available from the District and our in-house files;
- Drilled two (2) borings at the wastewater treatment plant site to depths of approximately 30 and 50 feet;
- Advanced four (4) cone penetration test (CPT) soundings to depths of up to 67 feet;
- As an addition to our scope of work, we worked with the District to excavate three (3) test pits to depths of up to 6 feet within a soil stockpile located adjacent to the effluent storage reservoir north of the site;
- Performed laboratory tests on selected samples obtained from the borings and test pits; and
- Prepared this report summarizing the data obtained for the site and our opinions and recommendations regarding:
  - Soil and groundwater conditions encountered;
  - Geologic and seismic setting;
  - Potential for liquefaction and seismic settlement;
  - Site preparation and grading;
  - Requirements for on-site and imported fill materials;
  - Design of the 1MG above grade tanks:
    - Site preparation and grading;
    - Backfill and compaction requirements;
    - Suitability of on-site materials for use as fill below the tanks;
Ground motion parameters for seismic design for use with the current AWWA-D100/2007 California Building Code (CBC) and ASCE 7-05 standards for steel tank design, including plotted seismic response spectra for critical damping of 0.5 and 5 percent and extrapolated to a sloshing period of 15 seconds;

- Allowable foundation bearing pressures, footing depths and widths and estimated settlement;
- Settlement considerations;
- Passive pressure and friction coefficients to resist lateral loads;
- Corrosion data; and

- Design of pipelines:
  - Typical trench detail;
  - Foundation support and bedding thicknesses for the pipe;
  - Preparation and placement of backfill;
  - Thrust block soil parameters;
  - Suitability of on-site materials for use as bedding, pipe zone or trench backfill; and
  - Compaction requirements for fill materials.

- Design of below grade Clearwell Structure with Booster Pumps:
  - Foundation preparation and backfill requirements;
  - Need for subgrade stabilization and/or dewatering;
  - Passive pressure and friction coefficients to resist lateral loads; and
  - Lateral earth pressures (static and dynamic) for non-yielding buried walls.

- Construction considerations regarding groundwater conditions, need for temporary slopes or shoring, and excavation characteristics of the soils encountered.

2.3 FIELD EXPLORATION

The field exploration for this project consisted of advancing four cone penetration test (CPT) soundings on June 27, 2008 and two borings on July 26, 2008. The logs for the CPT soundings are presented in Appendix C. The logs for the borings are presented in Appendix A. The approximate locations of the explorations are shown on Plate 2 - Field Exploration Plan.
2.3.1 Drilling

The drilling subcontractor for the project was S/G Drilling Company of Lompoc, California. S/G drilled two (2) borings on July 26, 2008 to depths of approximately 30 and 50 feet below the existing ground surface. The approximate locations of the borings are shown on Plate 2, Field Exploration Plan.

Boring DH-1 was advanced using mud rotary techniques to drill a 5-inch diameter hole a depth of approximately 50 feet. Boring DH-2 was drilled to a depth of approximately 30 feet using 8-inch hollow stem augers. The borings were sampled using a 2-inch outside diameter standard penetration test (SPT) split-spoon sampler, a 3-inch outside diameter modified California split-spoon sampler, and a 3-inch outside diameter Shelby tube. Shelby tubes were thin-walled samplers that were pushed into the soil at the bottom of the hole using the rig’s hydraulics to obtain relatively undisturbed samples. The modified California sampler was equipped with 1-inch high brass rings. The SPT sampler was used without liners. The split-spoon samplers were driven into the materials at the bottom of the drill hole using a 140-pound automatic trip hammer with a 30-inch drop.

The blow count (N-value) shown on the boring logs is the number of blows from the hammer that were needed to drive the sampler 1 foot, after the sampler had been seated at least 6 inches into the material at the bottom of the hole. Bulk samples were collected from the drill cuttings retrieved from the auger flights. The borings were backfilled with the soil cuttings and tamped after drilling. The sample intervals, N-values, a description of the subsurface conditions encountered and other field and laboratory data are presented on the logs of the borings in Appendix A.

2.3.2 Cone Penetration Testing

CPT soundings were performed by Fugro Geosciences of Santa Fe Springs, California. Fugro advanced four (4) CPT soundings on June 27, 2008 to depths ranging between approximately 65 and 67 feet below the existing ground surface. The four CPT soundings were advanced to refusal. The approximate locations of the borings are shown on Plate 2, Field Exploration Plan.

The CPT soundings were performed using an electric cone penetrometer. The penetrometer was advanced into the ground using a hydraulic ram mounted in a truck having a weight of approximately 20 tons. The cone penetrometer has a diameter of approximately 1.4 inches. Cone tip resistance (q_t), sleeve friction (f_s), and penetration pore pressures measured behind the tip (u_2) were recorded on the penetrometer during penetration. Data was recorded at approximately 2-centimeter intervals using an on-board computer to provide a near-continuous profile of the soil conditions encountered. The friction ratio (FR) was computed for each value of q_t and f_s recorded. The data was retrieved electronically for use in subsequent geotechnical analyses. CPT data and soil behavior type classifications were used to evaluate the subsurface conditions encountered at the site. Plots of each CPT sounding are presented in Appendix C.
2.3.3 Exploratory Test Pits

During this work, the County asked about the possibility of using stockpiled soil materials they have stored for as fill for this project. We agreed to evaluate the stockpile as part of our evaluation for this report. Three exploratory test pits were excavated on September 9, 2008 on the easterly side of the County's stockpile that is located approximately 2,000 feet north of the site. The test pits were excavated by County personnel using a rubber-tired backhoe with a 2-foot wide bucket. The test pits were excavated with the backhoe to depths of approximately 7 feet below the ground surface. Bulk samples were collected from the test pits, packaged in the field, and transported to the laboratory for subsequent testing. Upon completion of each test pit, the excavations were backfilled with the excavated materials.

2.4 LABORATORY TESTING

Laboratory tests were performed on selected soil samples retrieved from each boring and test pit. The laboratory testing program for this project included tests for grain size distribution, plasticity (Atterberg limits), direct shear, compaction, consolidation, R-value, sand equivalent, triaxial compression, expansion, and corrosion. Corrosion tests were performed by LA Testing of Las Alamitos, California. The tests were performed in general accordance with the applicable standards of ASTM. The results of the tests are presented in Appendix B and on the boring logs in Appendix A.

2.5 PREVIOUS STUDIES


Staal, Gardner & Dunne (1985) performed a geotechnical investigation for the existing pump house and operations building. That report included a boring at the existing pump house and associated laboratory data. The field and laboratory data collected for that geotechnical investigation as well as others referenced in this report were used during development of this geotechnical study and report.

2.6 GENERAL CONDITIONS

Fugro prepared the conclusions and professional opinions presented in this report in accordance with generally accepted geotechnical engineering principals and practices at the time and location this report was prepared. This statement is in lieu of all warranties, expressed or implied.

This report has been prepared for Santa Barbara County and their authorized agents only. It may not contain sufficient information for the purposes of other parties or other uses. If any changes are made in the project as described in this report, the conclusions and recommendations contained in this report should not be considered valid unless Fugro reviews
the changes and modifies and approves, in writing, the conclusions and recommendations of this report. The report and drawings contained in this report are intended for design-input purposes; they are not intended to act as construction drawings or specifications.

Soil and rock deposits will vary in type, strength, and other geotechnical properties between points of observation and exploration. Additionally, groundwater and soil moisture conditions can also vary seasonally or for other reasons. Therefore, we do not and cannot have complete knowledge of the subsurface conditions underlying the site. The conclusions and recommendations presented in this report are based upon the findings at the points of exploration, and interpolation and extrapolation of information between and beyond the points of observation, and are subject to confirmation based on the conditions revealed during construction.

The scope of services did not include any environmental assessments for the presence or absence of hazardous/toxic materials in the soil, surface water, groundwater, or atmosphere. Any statements or absence of statements in this report or data presented herein regarding odors, unusual or suspicious items, or conditions observed are strictly for descriptive purposes and are not intended to convey engineering judgment regarding potential hazardous/toxic assessment.

3. SITE CONDITIONS

3.1 GEOLOGIC SETTING

The project site is located in the Santa Maria basin, a transitional area between the Coast Ranges geomorphic province to the north and the Transverse Ranges to the south. The onshore Santa Maria basin is a northwest oriented structural basin that could have been formed by a large tectonic depression originating during the Miocene as a result of extension related to the San Andreas Fault System (Richmond et al. 1981, Tennyson 1992). The site is likely at the easterly end of lacustrine plain formed within the former bed of the Guadalupe Lake. The lake is formed over the axis of the Santa Maria syncline.

As mapped by Tennyson (1992), Dibblee (1994), and Worts (1951), the surface geology of the surrounding area consists of sediments of alluvium (Qa), stabilized dune sand deposits (Qs), and the Orcutt Formation (Qo). The near surface soil encountered within alluvium likely contains units of sediment and organic material that were deposited within the former limits of Guadalupe Lake. Based on our site explorations, data review, and the mapped surficial geology it appears that the site is underlain by a relatively thin layer of artificial fill that overlies alluvium which is then underlain by the Orcutt Formation.

3.2 SUBSURFACE CONDITIONS

The subsurface conditions encountered generally consisted of relatively thin units artificial fill materials (Af) overlying alluvium (Qal) and the Orcutt Formation (Qo). Two (2) borings and four (4) CPT soundings were performed for this study at locations shown on Plate 2. Three test pits were excavated in a soil stockpile located approximately 2000 feet north of
the site. A summary of our interpretation of the subsurface conditions is shown on Plate 3, Subsurface Profile. Discussions of the soil conditions encountered are provided below. The logs for the borings and CPT soundings are presented in Appendix A and C, respectively.

**Artificial Fill (Af).** Artificial fill materials were encountered in each of the explorations. Artificial fill materials were encountered from the ground surface to depths of approximately 1 to 5 feet below the ground surface. The artificial fill consisted of medium dense silty sand (SM), sand with gravel (SP), and soft silty clay (CL-ML) with varying amounts of gravel. Additionally, we observed gravel and aggregate base materials used for roadway construction, utilities, and concrete retaining walls associated with the abandoned sludge drying beds. Artificial fill materials encountered during our field exploration appear to be associated with grading activities associated with the wastewater treatment plant. Where encountered, the artificial fill materials were underlain by alluvium.

The results of laboratory tests performed on two samples of artificial fill had dry unit weights of approximately 96 and 103 pounds per cubic foot (pcf) and moisture contents of approximately 2 percent.

**Stockpile Artificial Fill (Af).** Artificial fill materials were encountered in each of the test pits excavated in the County soil stockpile area, just south of the effluent storage reservoir. Test pits were excavated approximately 7 feet into the stock pile to sample the artificial fill materials. The artificial fill materials consist of soil that was excavated during the construction of the adjacent effluent storage reservoir (K-C Geotechnical Associates 1991, 1992a, and 1992b). The artificial fill encountered predominately consisted of silty sand (SM) and clayey sand with (SC) with layers of sandy lean clay (CL).

**Alluvium (Qa1).** Alluvium was encountered below the artificial fill in each of the explorations. Alluvium was encountered from below the artificial fill materials to depths of approximately 23 to 29 feet below the existing ground surface. The alluvium was underlain by the Orcutt Formation. The alluvium encountered in the borings consisted of two predominant units that were characterized for our evaluation: sandy alluvium with interbedded clay and silt layers (Qa1) and clayey alluvium with interbedded sand and silt layers (Qa2). The estimated limits of the two units of alluvium are shown on Plate 3 - Subsurface Profile.

**Qa1.** This unit of alluvium was encountered below the artificial fill and above the Orcutt Formation and contains layers of the clayey alluvium unit (Qa2). The thickness of this unit ranged from approximately 2 to 14 feet. This unit of alluvium consisted of loose to very dense silty sand (SM), sand with silt (SP-SM), clayey sand (SC), clayey gravel (GC), and layers of gravel with relatively thin interbedded layers of clay and silt.

The results of laboratory tests performed on samples of the sandy alluvium had dry unit weights between approximately 89 and 112 pounds per cubic foot with moisture contents between approximately 10 and 22 percent.

**Qa2.** This unit of alluvium was encountered as fine-grained soil layers within the sandy alluvium unit (Qa1) (see Plate 3). The thickness of this unit ranged from approximately 4 to 13
feet. This unit of alluvium consisted of very soft to stiff clayey silt (ML), silty clay (CL-ML), lean clay (CL) and fat clay (CH) with varying amounts of sand and gravel. Although not classified as organic soils, the clayey alluvium contained organic material and voids left from root holes. CPT soundings suggest relatively thin layers of organic soils (such as peat or muck) may also be present within this unit. The organic materials may be associated with historic lake bed deposits.

The results of laboratory tests performed on samples of the clayey alluvium had dry unit weights ranging between approximately 86 and 103 pounds per cubic foot with moisture contents ranging between approximately 21 and 39 percent. Atterberg limits tests performed on two samples of the alluvium comprised of lean clay and fat clay had liquid limits of approximately 37 and 57 and plasticity indices of 23 and 37, respectively. Unconsolidated undrained triaxial tests performed on two samples of the clayey alluvium had shear strengths of approximately 680 and 1,140 pounds per square foot. Consolidation tests performed on two samples of the clayey alluvium materials consolidated between approximately 16 and 24 percent during testing, and had maximum past pressure ranging from approximately 1 to 2 times their existing overburden pressure.

Orcutt Formation (Qo). Orcutt Formation was encountered below the alluvium in each of the explorations. The Orcutt Formation was encountered to the maximum depths explored, approximately 67 feet below the existing ground surface. The Orcutt Formation consisted of interbedded firm to hard lean clay (CL), fat clay (CH) and medium dense to dense sandy silt (ML) and silty sand (SM). Refusal to penetration was encountered at the depth of termination in each of the four CPT soundings, approximately 65 to 67 feet below the existing ground surface.

The results of laboratory tests performed on samples of the Orcutt Formation had dry unit weights ranging between approximately 92 and 103 pounds per cubic foot with moisture contents ranging between approximately 23 and 32 percent. Atterberg limits tests for samples of the Orcutt Formation comprised of sandy lean clay, sandy silt, and lean clay had liquid limits ranging between approximately 28 and 43 and plasticity indices ranging between 8 and 23. Consolidation tests performed on samples of the Orcutt Formation consolidated between approximately 16 and 22 percent during testing, and had maximum past pressures ranging between approximately 2 to 4 times their existing overburden pressure.

3.3 GROUNDWATER CONDITIONS

Groundwater was encountered in the explorations at depths ranging between approximately 14 and 17 feet below the existing ground surface during our June and July 2008 field exploration program. Wet soil conditions were also encountered at depths as shallow as 7 to 8 feet below the existing ground surface (see Plate 3). The wet soil conditions appear to be associated with perched water above and within the fine grained alluvial soils. Previous studies report groundwater as shallow as 4 feet in the project vicinity (Staal, Gardner & Dunne, 1985) in some areas of the site. There is commonly surface water standing in areas around the plant during periods of wet weather. Groundwater depths and soil moisture conditions will vary seasonally depending on storm runoff, precipitation, irrigation, and other factors. A summary of groundwater depths reported in the vicinity of the proposed improvements follows:
## Summary of Groundwater Levels

<table>
<thead>
<tr>
<th>Well/Boring</th>
<th>Location</th>
<th>Date Recorded</th>
<th>Elevation (depth)</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Well No. 1</td>
<td>North of plant, adjacent to Orcutt Creek</td>
<td>6/17/1986</td>
<td>N/A (24 ft. bgs)</td>
<td>Hoover (1986)</td>
</tr>
<tr>
<td>B-9, CC-1</td>
<td>Dutard Road, Entrance to Plant</td>
<td>12/11/1987</td>
<td>No encountered to 10 ft. bgs</td>
<td>Hoover (1988a), Central Coast Labs (1971)</td>
</tr>
<tr>
<td>B-4</td>
<td>Chlorine Contact Basin, north of sludge drying beds</td>
<td>11/7/1973</td>
<td>N/A (17 ft. bgs)</td>
<td>Central Coast Labs (1973); Jenks &amp; Adamson (1974)</td>
</tr>
<tr>
<td>B-1</td>
<td>Near RO Filters</td>
<td>7/12-25/2001</td>
<td>N/A (13 ft. bgs)</td>
<td>Santa Barbara County (2001); Bengai (2001)</td>
</tr>
<tr>
<td>B-2</td>
<td>Tertiary Membrane Filters</td>
<td>7/12/2001</td>
<td>N/A (13 ft. bgs)</td>
<td></td>
</tr>
<tr>
<td>B-3</td>
<td>South of Pond C – Tertiary Holding Pond</td>
<td>7/12-25/2001</td>
<td>N/A (8 ft. bgs)</td>
<td></td>
</tr>
<tr>
<td>B-5</td>
<td>East of Plant</td>
<td>7/16/2001</td>
<td>N/A (13 ft. bgs)</td>
<td></td>
</tr>
<tr>
<td>DH-1</td>
<td>Proposed NE Tank Site</td>
<td>7/26/2008</td>
<td>el. 121 ft. (15 ft. bgs)</td>
<td>Current Study</td>
</tr>
<tr>
<td>DH-2</td>
<td>Proposed Clear Well</td>
<td>7/26/2008</td>
<td>el. 122 ft. (17 ft. bgs)</td>
<td></td>
</tr>
<tr>
<td>C-01</td>
<td>Proposed SW Tank Site</td>
<td>6/27/2008</td>
<td>el. 123 ft. (14 ft. bgs)</td>
<td></td>
</tr>
<tr>
<td>C-03</td>
<td>Proposed NE Tank Site</td>
<td>6/27/2008</td>
<td>el. 122 ft. (14 ft. bgs)</td>
<td></td>
</tr>
</tbody>
</table>

N/A – not available as reported in referenced report  
bgs – below ground surface

### 3.4 SEISMIC CONDITIONS

#### 3.4.1 Faulting

The site is located within a seismically active region of Central California that is prone to moderate to large earthquakes. The computer program FRISKSP (Blake, 2000) and the California Geologic Survey (CGS, 2002) fault database were first used to perform a search of potential earthquakes occurring on active or potentially active faults mapped within a 62-mile (100 km) radius of the site. The site location was estimated as −120.5033 degrees longitude and 34.8923 degrees latitude. Summarized below are 11 faults and fault segments that were considered the most capable of causing strong ground motion at the site. Additional information is provided in the CGS (2002) fault database.

## Summary of Predominant Faults

<table>
<thead>
<tr>
<th>Fault</th>
<th>Approximate Distance From the Site (miles)</th>
<th>Maximum Moment Magnitude (Mw)</th>
<th>Fault or Fault Segment Length (miles)</th>
<th>Slip Rate (mm/yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Casmalia (Orcutt Frontal Fault)</td>
<td>1.3</td>
<td>6.5</td>
<td>18</td>
<td>0.3 ± 0.2</td>
</tr>
<tr>
<td>Fault</td>
<td>Approximate Distance From the Site (miles)</td>
<td>Maximum Moment Magnitude (Mw)</td>
<td>Fault or Fault Segment Length (miles)</td>
<td>Slip Rate (mm/yr)</td>
</tr>
<tr>
<td>------------------------</td>
<td>--------------------------------------------</td>
<td>------------------------------</td>
<td>--------------------------------------</td>
<td>------------------</td>
</tr>
<tr>
<td>Lions Head</td>
<td>4.5</td>
<td>6.6</td>
<td>25</td>
<td>0.02 ± 0.02</td>
</tr>
<tr>
<td>San Luis Range (S. Margin)</td>
<td>8</td>
<td>7.2</td>
<td>40</td>
<td>0.2 ± 0.1</td>
</tr>
<tr>
<td>Los Alamos-W. Baseline</td>
<td>14</td>
<td>6.9</td>
<td>17</td>
<td>0.7 ± 0.07</td>
</tr>
<tr>
<td>Hosgri</td>
<td>14</td>
<td>7.3</td>
<td>107</td>
<td>2.5 ± 1.0</td>
</tr>
<tr>
<td>Los Osos</td>
<td>16</td>
<td>7.0</td>
<td>27</td>
<td>0.5 ± 0.4</td>
</tr>
<tr>
<td>Santa Ynez (west)</td>
<td>28</td>
<td>7.1</td>
<td>46</td>
<td>2.0 ± 1.0</td>
</tr>
<tr>
<td>Rinconada</td>
<td>29</td>
<td>7.5</td>
<td>117</td>
<td>1.0 ± 1.0</td>
</tr>
<tr>
<td>North Channel Slope</td>
<td>30</td>
<td>7.4</td>
<td>42</td>
<td>2.0 ± 2.0</td>
</tr>
<tr>
<td>San Juan</td>
<td>35</td>
<td>7.0</td>
<td>42</td>
<td>1.0 ± 1.0</td>
</tr>
<tr>
<td>San Andreas (Cholame)</td>
<td>46</td>
<td>7.3</td>
<td>38</td>
<td>34 ± 5</td>
</tr>
</tbody>
</table>

3.4.2 Probabilistic Ground Motions

A probabilistic seismic hazard evaluation was also performed for the site using FRISKSP. FRISKSP is based on FRISK (McGuire, 1978) and has been modified for the probabilistic estimations of seismic hazards using three-dimensional earthquake sources, such as those listed above. The ASCE (2005) design code and the California Building Code (CBC 2007) require structures to be designed for earthquake effects that are two-thirds ($\frac{2}{3}$) of corresponding Maximum Considered Earthquake (MCE) effects. Structural designs are based on the 0.2s and 1.0s period spectral accelerations corresponding to the MCE for a Site Class "B" (site class is defined per ASCE [2005], CBC [2007]) which are modified, if necessary, to account for different site class effects.

Estimated MCE ground motions are site class-modified spectral accelerations corresponding to earthquakes estimated to have a 2 percent chance of being exceeded in 50 years, or a return period of about 2,475 years. Design earthquake ground motions for liquefaction and other geotechnical analyses are defined as two-thirds ($\frac{2}{3}$) of the corresponding MCE ground motions. Two thirds ($\frac{2}{3}$) of the MCE was estimated to have a ground acceleration of 0.41g with a corresponding earthquake magnitude (M) of M6.5. As a result of statistical variations in the methods used to estimate strong ground motion, we expect that ground acceleration exceeding 2/3 the MCE could potentially occur if an earthquake were to occur near the site. Design response spectra for the site are presented with the Seismic Data in Section 4.3 of this report.

3.5 LIQUEFACTION AND SEISMIC SETTLEMENT

Liquefaction and seismic settlement hazards were evaluated for the site considering the design earthquake with a ground acceleration of 0.41g and a corresponding earthquake of M6.5. Liquefaction is a loss of soil strength due to a rapid increase in pore water pressures due to cyclic loading during a seismic event. The procedures described in the 1997 NCEER guidelines (Youd and Idriss, 2001) were used to evaluate liquefaction potential of the soils.
Liquefaction commonly occurs in loose to medium dense sandy soil that is below the groundwater table at the time of an earthquake. The potential and severity of liquefaction will depend on the intensity and duration of the strong ground motion. Seismically induced settlement can occur in association with liquefaction, and in soils not prone to liquefaction, which are loose or medium dense and weakly cemented. Settlement, lateral spread, collapse, and loss of bearing support are common manifestations of liquefaction.

Groundwater water was encountered at depths between 14 and 17 feet below the existing ground surface during our 2008 field exploration program. The fine grained units of alluvium (QaI2) and the Orcutt Formation (Qo) encountered in the explorations were predominately very soft to hard fine-grained clayey silt and clay and medium dense to dense sand that are not considered susceptible to liquefaction. The sand unit of the alluvium (QaI1) encountered consisted of mostly medium dense to dense sandy materials that are not considered susceptible to liquefaction. However, the sandy alluvium encountered also contained interbedded layers of loose to medium dense sand that are considered locally liquefiable and prone to moderate seismic settlement. Those layers prone to liquefaction and seismic settlement within QaI1 appear to be within relatively thin and/or discontinuous layers across the site.

Based on analysis of the CPT soundings, we estimate that approximately 1 to 2 inches of seismic settlement could occur within the sand unit of the alluvium (QaI1) in response to the design earthquake. Because the soil profile is variable, the estimated settlement would likely occur as differential settlement across the site and between tank locations. The largest estimated seismic settlement was in CPT-C-3. CPT C-3 encountered approximately 4 to 5 feet of sand at a depth of approximately 15 to 20 feet below the existing ground surface that could be potentially liquefiable.

4. CONCLUSIONS AND RECOMMENDATIONS

4.1 SUMMARY OF FINDINGS

- The site is underlain by a relatively thin layer of artificial fill overlying alluvium and the Orcutt Formation. The alluvium and Orcutt Formation generally consisted of interbedded sand and clay layers. Potentially compressible clay layers were encountered between approximately 5 to 12 feet, 18 to 20 feet, and 30 to 42 below the existing ground surface. Relatively dense and stiff to hard materials were encountered below approximately 42 feet below the existing ground surface to the maximum depth explored. The maximum depth explored was approximately 65 to 67 feet below the existing ground surface where refusal to penetration was encountered in each of the four CPT soundings.

- Groundwater was encountered at depths of approximately 14 to 17 feet below the existing ground surface in the explorations. Wet and saturated soil conditions were encountered at depths as shallow as 7 feet below the existing ground surface. The wet soils are likely a result of perching water above and within the clayey alluvium.
Layers of potentially compressible soft clay were encountered within the alluvium. We estimate that total static settlements of approximately 2 to 4 inches and differential settlements of up to 1 to 2 inches could occur across the footprints of the 80-foot diameter steel tanks. Recommendations have been provided in this report to reduce the estimated static settlements to less than about 1 inch. The settlement could be limited by either constructing a temporary soil surcharge to about 15 feet over the proposed tank sites, or by removing the compressible layers and replacing them with compacted fill to a depth of about 10 to 12 feet.

Layers of sand encountered within the alluvium are prone to liquefaction and seismic settlement. Those layers were encountered at varying depths and locations between explorations, and appear to be relatively thin and discontinuous across the site and tank locations. The impact of liquefaction to the new tanks should be limited to areal and differential settlement that could occur in response to an earthquake. Based on analysis of the CPT soundings, we estimate that seismic settlements of up to approximately 1 to 2 inches could occur in response to the design earthquake at the site.

Structures can be supported on shallow or mat type foundations as planned. Proper site preparation and grading should be performed to provide uniform support for structures and limit post construction settlements according to the recommendations of this report.

Based on the subsurface conditions encountered, the majority of the on-site soil should not be considered suitable for use as select materials (structural backfill, pipe bedding or pipe zone material). The materials excavated within the upper 4 to 5 feet of the site can likely be used for compacted fill or trench backfill material. However, portions of the clay material encountered below a depth of about 4 to 5 feet are predominantly fine-grained, wet, and may contain organic materials that are not considered suitable for use as compacted fill. Fine grained material can be sensitive to changes in moisture content and relatively difficult to compact.

Based on sampling of the County's soil stockpile located approximately 2,000 feet north of the project site, and review of the soil borings performed for the effluent storage reservoir from which these stockpile was derived (K-C Geotechnical Associates 1993), the majority of the soil stockpile consists of silty sand (SM) and clayey sand (SC) with layers of sandy lean clay (CL). The stockpile soil should not be considered suitable for use as select materials (structural backfill, pipe bedding or pipe zone material). The materials appear suitable for use as compacted fill, surcharge material, or trench backfill material. The fine grained material encountered in the stockpile can be sensitive to changes in moisture content and relatively difficult to compact.

Excavation for the clear well structure are expected to encounter approximately 5 to 6 feet of sandy material over very soft to soft wet clay material that may contain organics (see boring DH-2). The clay material is likely not suitable for use as
compacted fill. If the sandy material is to be used as fill material, the sandy material should be segregated from the clay material during excavation.

4.2 GRADING – GENERAL

4.2.1 Grading

Fill placement and grading operations should be performed according to the grading recommendations of this report. We recommend that, unless otherwise noted, fill and backfill materials be compacted to at least 90 percent relative compaction, as determined by the latest approved edition of ASTM Test Method D1557, unless a higher degree of compaction is otherwise recommended. Fill and backfill materials placed in foundation and pavement areas should be compacted to at least 95 percent relative compaction. Cut and fill slopes, if needed, should be designed to inclinations of 2h:1v or flatter.

4.2.2 Suggested Material Specifications

The following materials are referenced in various sections of this report. Additional recommendations for placement of trench backfill materials, and other components of the project, are presented in the sections that follow.

**Aggregate base** shall consist of imported material conforming to Caltrans Standard Specifications for Class 2 aggregate base, Section 26-1.02A. Class 3 material that incorporates reclaimed or recycled materials can also be used as aggregate base, provided the Class 3 material complies with the gradation and quality requirements for Class 2 material. Class 3 material shall not be placed below building areas.

**Asphalt concrete** shall conform to Caltrans Standard Specifications for Type A asphalt concrete, Section 39. Binder shall consist of PG 64/10 asphalt binder.

**Compacted fill material** shall consist of imported or on-site material free of organics, oversize rock (greater than 3 inches), trash, debris, corrosive, and other deleterious materials. Imported fill shall be reviewed by the geotechnical engineer prior to being brought to the site; however, imported fill materials shall comply with all specifications for material placed at the site. Fill materials shall comply with all specified material requirements for the area where the material is being placed. Fill materials used in tank areas shall have an Expansion Index of less than 20.

**Drainage material** shall conform to Caltrans Standard Specifications for Class 2 permeable material, Section 68-1.025. ASTM C-33 No. 8 coarse aggregate (pea gravel) can be used in lieu of Class 2 permeable material provided the materials are enclosed in a filter fabric. As an alternative, prefabricated geocomposite drainage panels can be placed behind retaining walls as recommended in this report.

**Geotextile for separation (filter fabric)** shall consist of geotextile that conforms to the requirements outlined in the Caltrans Standard Specifications for Filter Fabric-underdrains, Section 88-1.03.
Geotextile for subgrade stabilization shall conform to the requirements outlined in Caltrans Standard Specifications for Rock Slope Protection Fabric, Section 88-1.04.

Geocomposite drain shall consist of a manufactured plastic core not less than 8 millimeters thick with both sides covered with a layer of filter fabric that will provide a continuous drainage void in the horizontal and vertical directions. Geocomposite drain placed behind retaining walls shall have an impermeable backing. Geocomposite drain to be embedded in the ground shall be double-sided with filter fabric covering both sides of the drainage void.

The drain shall produce a flow rate through the drainage void of at least 10 gallons per minute per foot of width at a hydraulic gradient of 1.0 under a maximum externally applied pressure of 2,000 psf. The core materials and filter fabric shall be capable of maintaining the drainage void for the entire height of the geocomposite drain. Filter fabric shall be integrally bonded to the core materials with the drainage void. Core material manufactured from impermeable plastic sheets having non-connecting corrugations shall not be permitted.

The fabric shall overlap a minimum of 6 inches at all joints and wrap around the exterior edges of the drain a minimum of 6 inches beyond the edge. If additional fabric is needed to provide overlaps at joints and to wrap around the edges of core material, the added fabric shall overlap the fabric on the geocomposite drain at least 6 inches and be attached thereto.

Should the fabric on the geocomposite drain be torn or punctured: 1) the damaged section shall be replaced completely if damage is done to the core material, or 2) if the core material is not damaged than the repair can be performed by placing a piece of fabric that is large enough to cover the damaged area and provide a 1-foot overlap.

Pipe zone material shall consist of imported soil having a sand equivalent (SE per ASTM 2419) of at least 30 and conforming to Section 19-3.025B, Sand Bedding, of the Caltrans Standard Specifications.

Pipe bedding material shall consist of imported material having a sand equivalent of at least 30, and conforming to Section 19-3.025B, Sand Bedding, of the Caltrans Standard Specifications.

Pipe bedding material - gravel for trench bottom stabilization shall consist of material conforming to Caltrans Section 90-3.02, Coarse Aggregate Grading.

Retaining wall backfill material shall consist of either on-site or imported material conforming to Caltrans Standard Specifications for Structure Backfill, Section 19-3.06.

Subgrade stabilization material - gravel shall consist of material conforming to Caltrans Section 90-3.02, Coarse Aggregate Grading.

Trench backfill shall consist of imported or onsite material that is free of organics, debris, oversized material greater than 3 inches, and other deleterious materials. Trench backfill material shall have at least 50 percent of the material passing the U.S. Standard No. 4
sieve, and/or comply with the applicable requirements for the area where the trench backfill is being placed (such as the pavement structural section).

4.2.3 Use of On-site Soils

Based on the subsurface conditions encountered, the majority of the on-site soil should not be considered suitable for use as select materials (structural backfill, pipe bedding or pipe zone material). The materials excavated within the upper 4 to 5 feet of the site can likely be used for compacted fill or trench backfill material. However, the clayey materials encountered below a depth of about 4 to 5 feet are predominantly fine-grained, wet, and may contain organic materials that are not suitable for use as compacted fill. Fine grained material can be sensitive to changes in moisture content and relatively difficult to compact. During construction, segregation of the suitable sandy materials from the unsuitable clayey materials will need to be performed if the sand materials are to be used as compacted fill.

Based on sampling of the soil stockpile located approximately 2,000 feet north of the project site and review of the soil borings performed for the effluent storage reservoir (K-C Geotechnical Associates 1993), the majority of the soil stockpile consists of silty sand (SM) and clayey sand (SC) with layers of sandy lean clay (CL). The stockpile soil should not be considered suitable for use as select materials (structural backfill, pipe bedding or pipe zone material). The materials can likely be used for compacted fill, surcharge material, or trench backfill material. The fine grained material encountered in the stockpile can be sensitive to changes in moisture content and relatively difficult to compact.

4.2.4 Clearing and Grubbing

Prior to commencing grading operations in building or roadway areas that will receive compacted fill or structures, soil containing debris, organics, pavement, uncompacted fill, or other unsuitable materials, should be removed. Demolition areas should be cleared of old foundations, slabs, abandoned utilities, and soils disturbed during the demolition process. Depressions or disturbed areas left from the removal of such material should be replaced with compacted fill.

4.2.5 Fill Placement

Fill should be placed and compacted to at least the minimum relative compaction recommended in this report. The moisture content of the fill should be between 2 percent below to 2 percent above the optimum. Each layer should be spread evenly and should be thoroughly blade-mixed during the spreading to provide relative uniformity of material within each layer. Soft or yielding materials should be removed and be replaced with properly compacted fill material, prior to placing the next layer.

Rock, gravel and other oversized material, greater than 3 inches in diameter, should be removed from the fill material being placed. Rocks should not be nested and voids should be filled with compacted material.
When the moisture content of the fill material is below that sufficient to achieve the recommended compaction, water should be added to the fill. While water is being added, the fill should be bladed and mixed to provide relatively uniform moisture content throughout the material. When the moisture content of the fill material is excessive, the fill material should be aerated by blading or other methods. Fill should be spread in thin lifts, typically no thicker than approximately 8 inches prior to being compacted. Fill and backfill materials may need to be placed in thinner lifts to achieve the recommended compaction with the equipment and type of soil being used. Compaction using jetting or ponding should not be permitted.

4.2.6 Compaction

Fill placement and grading operations should be performed according to the grading recommendations of this report. Relative compaction should be assessed based on the latest approved edition of ASTM D1557. We recommend the minimum relative compaction for the locations indicated in the following table:

<table>
<thead>
<tr>
<th>Location</th>
<th>Recommended Minimum Relative Compaction</th>
</tr>
</thead>
<tbody>
<tr>
<td>General</td>
<td>90 % U.O.N.</td>
</tr>
<tr>
<td>Utility trench bedding and pipe zone, and backfill materials</td>
<td>90 % U.O.N.</td>
</tr>
<tr>
<td>Backfill in non-pavement areas</td>
<td>90 % U.O.N.</td>
</tr>
<tr>
<td>Fill or backfill placed within 3 feet of finished grade in pavement areas</td>
<td>95 %</td>
</tr>
<tr>
<td>Asphalt concrete, aggregate base or subbase</td>
<td>95 %</td>
</tr>
<tr>
<td>tank and foundation areas and areas within 5 feet horizontally of the building or structure footprint</td>
<td>95 %</td>
</tr>
<tr>
<td>Retaining wall, buried tank or basin, or basement backfill</td>
<td>90% U.O.N.</td>
</tr>
</tbody>
</table>

U.O.N. = unless otherwise noted

4.3 SEISMIC CONSIDERATIONS

4.3.1 Seismic Settlement and Liquefaction

The potential for seismic settlement and liquefaction to impact the project site was evaluated using the procedures and methods discussed in Section 3.5 of this report.

Based on the subsurface explorations various layers of sand encountered within the Qal1 unit of alluvium are prone to seismic settlement. Those layers were encountered at various depths and locations between explorations, and appear to be relatively thin and discontinuous across the site. Based on analysis of the CPT soundings, we estimate that seismic settlements of approximately 1 to 2 inches could occur in response to the design earthquake. Because the soil profile is variable, the estimated settlement would likely occur as differential settlement across the site and between tank locations. The largest estimated seismic settlement was in CPT C-3. C-3 encountered approximately 4 to 5 feet of sand at a depth of approximately 15 to 20 feet below the existing ground surface that could be potentially liquefiable.
4.3.2 Seismic Data

Structures should be designed to resist the lateral forces generated by earthquake shaking in accordance with the building code and local design practice. This section presents seismic design parameters for use with the 2007 California Building Code (CBC). The CBC is based on the 2006 International Building Code (IBC). The site coordinate and USGS interactive web page "Seismic Design Values for Buildings" (USGS 2008) was used to obtain seismic design criteria. Based on these criteria, the seismic data for use with code-based designs are:

Summary of Seismic Data

<table>
<thead>
<tr>
<th>California Building Code</th>
<th>Seismic Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site Coordinates</td>
<td>Latitude, degrees</td>
<td>34.8924</td>
</tr>
<tr>
<td></td>
<td>Longitude, degrees</td>
<td>-120.5033</td>
</tr>
<tr>
<td>Section 1613.5.1</td>
<td>Ss , Seismic Factor, Site Class B at 0.2 sec</td>
<td>1.326</td>
</tr>
<tr>
<td>Figuro 1613.5</td>
<td>Ss , Seismic Factor, Site Class B at 1 sec</td>
<td>0.453</td>
</tr>
<tr>
<td>Section 1613.5.3</td>
<td>Site Class</td>
<td>Sd, Stiff soil</td>
</tr>
<tr>
<td>Table 1613.5.3(1)</td>
<td>Fa, Site Coefficient for Site Class D</td>
<td>1.0</td>
</tr>
<tr>
<td>Section 1613.5.3</td>
<td>Fv, Site Coefficient for Site Class D</td>
<td>1.5</td>
</tr>
<tr>
<td>Table 1613.5.3(2)</td>
<td>S_{MS}, Site Specific Response Parameter for Site Class D at 0.2 sec.</td>
<td>1.362</td>
</tr>
<tr>
<td></td>
<td>S_{MI}, Site Specific Response Parameter for Site Class D at 1 sec.</td>
<td>0.700</td>
</tr>
<tr>
<td>Section 1614A</td>
<td>S_{DS} = 2/3 S_{MS}</td>
<td>0.844</td>
</tr>
<tr>
<td></td>
<td>S_{D1} = 2/3 S_{MI}</td>
<td>0.467</td>
</tr>
</tbody>
</table>

Based on the seismic design parameters calculated by the USGS interactive web page "Seismic Design Values for Buildings" (USGS 2008), and per 2007 CBC Section 1613.5.6, structures of Occupancy Category I, II, III, and IV (defined in 2007 CBC Table 1604.5) should be designed according to Seismic Design Category “D”.

Per the CBC and ASCE 7-05 Section 21.4, the parameters $S_{DS}$ and $S_{D1}$ can be obtained from the site-specific design spectrum provided below. The $S_{D1}$ value is calculated as the larger of: (1) spectral acceleration at 1 second and (2) two times the spectral acceleration at 2 seconds. The $S_{DS}$ value is calculated as the larger of: (1) spectral acceleration at 0.2 second, and (2) 90 percent of the largest spectral acceleration for any period greater than 0.2 second.
4.3.3 Site Specific Response Spectra

Site-specific ground motion procedures were used to develop the design spectrum corresponding to 2/3 of the estimated MCE in accordance with the CBC 2007 (Chapter 21 of ASCE 7-05) and American Water Works Association (AWWA) Standard for welded steel tanks for water storage. The data from FRISKSP was used to estimate a response spectrum for the MCE with the attenuation relationship proposed by Abrahamson and Silva (1997). Design Response Spectra are presented on Plates 4a and 4b. The response spectra and associated background spectra at 5 percent damping are presented on Plate 4a. The estimated values of spectral acceleration ($S_a$) for the Design Response Spectra at damping ratios of 5 percent, and the adjusted spectra to 0.5 percent using the procedure recommended by Abrahamson and Silva (2003), are presented on Plate 4b.

Empirical attenuation relationships such as the one developed by Abrahamson and Silva (1997) allow for the estimation of response spectral ordinates for periods up to 5 seconds. For tank design, spectral ordinates are extrapolated to higher sloshing periods of up to 15 seconds. The spectral values beyond a 5 second period were extrapolated assuming constant spectral displacement.

Based on the subsurface conditions encountered, a Deep Soil profile was used for both probabilistic and deterministic seismic hazard evaluations for the site. The MCE used to generate the spectra is defined both probabilistically and deterministically. The Recommended Design Spectrum shown on Plate 4a is calculated from the following comparisons of probabilistic MCE and deterministic MCE. The lesser of the probabilistic ground motion having a 2 percent probability of exceedance in 50 years (i.e. 2475 year return period MCE) calculated with 5 percent damping, the greater of 150 percent of the median deterministic ground motion calculated for 5 percent damping, and a deterministic lower bound spectrum calculated according to ASCE 7-05 Section 21.2.2. Additionally, the recommended Design Response Spectrum presented on Plates 4a is defined by ASCE 7-05 as the greater of the site-specific MCE calculated above, or 80 percent of the general response spectrum calculated according to ASCE 7-05 Section 11.4. The deterministic, probabilistic, and general spectra used for the comparison are shown on Plate 4a with the Recommended Design Spectra. The deterministic spectrum was calculated using an M6.5 earthquake occurring approximately 1½ miles from the project site, on the Casmalia-Orcutt Frontal Fault.

The site-specific MCE spectrum described in this report takes into account the variation of near-surface stratigraphy at the project site. Therefore, the spectrum is applicable directly at the foundation level.

4.4 SITE PREPARATION AND GRADING

The site is underlain by a relatively thin thickness of existing fill over alluvial deposits. The fill and alluvium within the anticipated depth of excavation consists of loose to medium dense sandy materials over soft to stiff wet clay material. The recommended minimum depths of excavation are based on the depth of the loose materials encountered in the explorations and to provide relatively uniform support for foundations. The excavations should remove the
materials, and extend to at least 5 feet beyond the structure footprint or perimeter footings and slabs. Schematics summarizing grading recommendations for tanks and other at grade structures and for the clear well structure are shown on Plates 5, and 6, respectively. A summary of the estimated depths of removal follows:

**SUMMARY OF GRADING RECOMMENDATIONS**

<table>
<thead>
<tr>
<th>Structure</th>
<th>Depth of Excavation</th>
<th>Subgrade Preparation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Equipment Slabs</td>
<td>At least 4 ft below the existing ground surface or 2 ft below the bottom of the footing (whichever is deeper).</td>
<td>Scarify to a depth of 9 inches, moisture condition, and compact in-place to at least 90 percent relative compaction.</td>
</tr>
<tr>
<td>Tank Foundation with surcharge option</td>
<td>At least 4 ft below the existing ground surface or 2 ft below the bottom of the footing (whichever is deeper).</td>
<td>Scarify to a depth of 9 inches, moisture condition, and compact in-place to at least 90 percent relative compaction.</td>
</tr>
<tr>
<td>Tank Foundation with remove and replace option</td>
<td>Approximately 10 to 12 ft below the existing ground surface (to remove soft clay).</td>
<td>Place 2 ft of gravel encased in a filter fabric on undisturbed subgrade.</td>
</tr>
<tr>
<td>Clear well structure</td>
<td>2 ft below the bottom of the footing.</td>
<td>Place 2 ft of gravel encased in a filter fabric on undisturbed subgrade.</td>
</tr>
</tbody>
</table>

The geotechnical engineer should review the bottom of excavation prior to placing fill materials to evaluate whether or not the artificial fill materials and other loose or unsuitable materials have been removed, and that the base of the excavation is suitable for placing compacted fill. The project specifications should provide for review of the excavation by the geotechnical engineer, and for increasing the depth of the excavation to remove additional loose soil or other unsuitable materials if needed.

The excavation for the clearwell structure and the alternative for a deep excavation below the tanks (removal and replace option) should be performed using backhoe(excavator type equipment that will not operate on or disturb the base of the excavation. Gravel for subgrade stabilization should consist of open-graded material conforming to the recommendations of this report and be fully encased in a filter fabric. The fabric and gravel should be placed on a relatively undisturbed subgrade. The fabric should be overlapped or spliced as needed in accordance with the manufacturer’s recommendations.

Following subgrade preparation compacted fill can then be placed to finished grade according to the fill placement recommendations of this report. Fill materials placed in structure areas should be compacted to at least 95 percent relative compaction.

**4.5 FOUNDATIONS DESIGN**

**4.5.1 Shallow Foundation Design**

The proposed structures can be supported on spread footing foundations bearing in compacted fill materials prepared in accordance with the recommendations of this report. Spread footing foundations founded in compacted fill can be designed using maximum
allowable bearing pressure of 2,500 pounds per square foot. Continuous footings should be
designed with a width of at least 1 foot. Isolated pad footings should be designed with a least
dimension of 1.5 feet. Spread footings should be embedded at least 2 feet below the lowest
adjacent exterior grade or finished slab elevation whichever is deeper. The recommended
bearing pressure can be increased by 500 pounds per square foot for each additional foot of
footing width, and by 1,000 pounds per square foot for each additional foot of embedment, to a
maximum of 4,000 pounds per square foot.

The maximum allowable bearing pressure can be increased by one-third when
considering short-term wind or seismic loads. For retaining wall or eccentrically loaded footings,
the toe pressure can exceed the recommended maximum allowable bearing pressure provided
the resultant force acts within the middle third of the footing.

Reinforcing of foundations should be designed by the structural engineer based on
loading conditions. Based on the expected soil conditions, we recommend that at least four
Number 4 reinforcing bars be placed in continuous footings, two near the top and two near the
bottom.

4.5.2 Mat Foundation Design

Mat foundations can be designed using a modulus of subgrade reaction of 9 pounds per
cubic inch. The modulus of subgrade reaction was estimated from settlement analysis of a 10-
foot by 10-foot mat foundation bearing at or near the ground surface considering an average
bearing pressure of 2,000 pounds per square foot. The modulus value is provided for the
design of various equipment pad and structure foundations that may be supported on shallow
mat type foundations. The recommended modulus can be increased by 1/3 when considering
seismic or other transient loading conditions.

4.5.3 Resistance to Lateral Loads

Resistance to lateral loading can be provided by sliding friction acting on the base of
spread footings or slabs combined with passive pressure acting on the sides of foundations or
grade beams. We recommend that a coefficient of friction of 0.3 be used to estimate the sliding
resistance along the bottoms of footings or slabs bearing in compacted soil. We recommend
that a passive resistance of 300 pounds per cubic foot, equivalent fluid weight, be used to
estimate the lateral resistance acting on the sides of footings or grade beams. The passive
resistance should be reduced by ½ when considering submerged soil conditions. Passive
resistance should not be used for the upper one foot of soil that is not constrained at the ground
surface by slab-on-grade or pavement. A one-third increase in the passive value can be used
when considering short-term wind or seismic loads.
4.6 SETTLEMENT CONSIDERATIONS

4.6.1 General

We estimate that settlements resulting from static foundation loads should generally be approximately 1-inch total and approximately 3/4-inch differential in 30 feet for foundation elements (other than the tanks) designed according to the recommendations of this report. A discussion of static settlements for the above grade tanks are provided in Section 4.6.2 below.

The proposed structures are in areas that may be prone to seismic settlement and/or liquefaction as previously discussed in this report. The seismic settlement is expected to occur as areal and differential settlement occurring over the structure areas. We recommend that the foundation elements be tied together wherever possible with either strip footings or grade beams to help limit differential settlement and distribute structural loads. Additionally, pipe or utility penetrations for buildings, tanks or equipment should be made flexible to help accommodate seismic and differential settlement.

The estimated seismic settlements for the design earthquake are estimated to range from approximately 1 to 2 inches total. The design of foundations should consider that approximately half of the estimated total seismic settlement could occur differentially within a structure area.

4.6.2 Tank Foundation Settlements

The proposed tank sites are underlain by a relatively thin layer of artificial fill overlying alluvium and the Orcutt Formation. The alluvium and Orcutt Formation generally consisted of interbedded sand and clay layers. Potentially compressible clay layers were encountered between depths of about 5 and 12 feet, 18 and 20 feet, and 30 and 42 feet below the existing ground surface. Based on our settlement analysis and assuming an average bearing pressure of about 2,000 psf below the tanks, we estimate that under the static weight of the tanks the foundation soil could settle approximately 2 to 4 inches due to the elastic settlement of the sandy alluvium and consolidation of the clay layers. Because of the variability of the soil profile, the settlement could occur differentially over the tank sites. We estimate that approximately 1 to 2 inches of differential settlement could occur across an individual tank. We estimate that the consolidation would occur relatively rapidly, over a period of approximately 15 to 30 days.

To help reduce the estimated post-construction static settlement to approximately 1 inch, we recommend that the upper 10 to 12 feet of the existing soil be excavated and replaced with compacted fill or that a soil surcharge and waiting period be provided in advance of the tanks construction.

4.6.2.1 Option for Excavation and Replacement

This option consists of removing the upper compressible clay layer to a depth of about 10 to 12 feet below the existing ground surface to reduce the estimated static settlement to approximately 1 inch or less. The approximate limits of the recommended removal are shown
on Plate 5. The excavation should extend to at least 5 feet beyond the footprint of the tanks foundations. The excavation should be performed using backhoe/excavator type equipment that will not operate on or disturb the base of the excavation. To provide a working mat for construction, at least 2 feet of gravel should be placed for subgrade stabilization, as previously recommended in the Site Preparation and Grading recommendations of this report. The gravel for subgrade stabilization should consist of open-graded material conforming to the recommendations of this report and be fully encased in a filter fabric. Following subgrade preparation compacted fill can then be placed to finished grade according to the fill placement recommendations of this report. No settlement period or monitoring would need to be performed for this alternative.

4.6.2.2 Option for Surcharge and Settlement Period

This option would consist of placing a temporary soil surcharge for a waiting period to reduce the estimated static settlement to approximately 1 inch or less. The surcharge would consist of a soil embankment placed to approximately 15 feet above existing site grades at each tank location. The approximate limits of the surcharge are shown on Plate 5. The horizontal limits of the top of the surcharge fill should extend to at least 5 feet beyond the tank footprint. The surcharge should remain in place for at least 30 days, the time needed for the settlement to occur should be confirmed by the settlement monitoring program described herein, and the settlement period should begin at the end of the fill placement. Surchage fill material should be compacted to at least 85 percent relative compaction. Temporary side slopes for the surcharge can be placed at 1.5h:1v or flatter.

If needed, the surcharge fill can be placed for the 4 proposed tanks in one operation or the surcharge can be staged to initially surcharge the two western tanks areas and then the fill can be moved over to cover the eastern tank areas. Surcharging only a single tank area within either the eastern or western site is not considered geotechnically feasible because future surcharging within that same area would likely impact the initially constructed tank. As a result of the overlapping stress distributions, the future surcharge of the adjacent single tank could cause the already constructed tank to settle differentially under the weight of the adjacent surcharge.

4.6.2.3 Instrumentation Program

We recommend that instrumentation for the settlement period be provided in association with the surcharge to observe that the estimated settlement occurs within the estimated settlement period. Settlement monitoring can be provided using a fluid level settlement platform constructed below the fill prior to placing the surcharge fill material. The settlement platform should be installed and monitored according to California Test Method 112 and under the direction of the Engineer. We recommend that at least two settlement platforms be installed within the footprint of each tank.

The contractor should be responsible for installing, maintaining, and monitoring the settlement platforms at least 2 times per week during the settlement period, and for at least every 5 feet of surcharge fill that is placed. The geotechnical engineer should review the details
of the settlement platform construction, and review the monitoring data during construction and the settlement period as it is acquired. The project specifications should provide for review of the data, and for modifying the length of the settlement period, if needed. We recommend that the contractor be required to replace or repair any settlement platforms that are damaged by construction operations.

4.7 DESIGN OF THE BELOW GRADE CLEARWELL STRUCTURE

The design of the below graded clearwell should consider the subsurface conditions that it will be embedded within. The soil conditions encountered at the clearwell generally consisted of approximately 6 to 7 feet of sand over very soft clay (see boring DH-2). Groundwater was encountered at approximately 14 to 17 feet below the existing ground surface during our June and July 2008 field exploration program. However, wet soil conditions were encountered at about 8 feet below the existing ground surface. Recommendations for site preparation and grading and foundation design are provided in Sections 4.3 and 4.5 of this report.

We understand that a steel building will be constructed on top of the clearwell structure. Loading information for the building is not available at this time. The addition of the building will increase the load on the clearwell foundation that should be considered in the foundation design and estimated settlement for the clearwell. We recommend that Fugro review the clearwell structure relative to our foundation design and support recommendations once loading information becomes available.

4.7.1 Site Resistance to Uplift Loads

The design of below grade structures whose foundations may extend into soils below the groundwater table or flood levels should consider buoyant forces that may act upward on the structure and cause uplift. If needed, uplift forces due to buoyancy can be resisted by the buoyant dead weight of the structure and by friction acting between the exterior walls of the structure and the surrounding soil. The maximum allowable frictional resistance between the soil and the buried concrete structure can be estimated as 0.2 times the effective overburden stress. The effective overburden stress, in psf, can be estimated using an effective buoyant unit weight for submerged soil of 60 pcf times the depth in feet. If filter fabric will be placed between the wall backfill and the ground in close proximity to the structure, the uplift resistance due to friction should be neglected.

4.7.2 Lateral Earth Pressures

Retaining walls and below grade structures should be designed to resist lateral earth pressures. The design of structures should consider the presence of groundwater and associated increases in lateral earth pressures. We have assumed that the clear well structure will be designed as restrained walls and therefore will be subject to at-rest lateral earth pressures. Active earth pressures are also provided for unbraced walls. Backfill material placed adjacent to subsurface structures should be compacted to at least 90 percent relative compaction, unless a higher degree of compaction is otherwise recommended for building or pavement areas.
Backfill material for retaining structures should consist of Retaining Wall Backfill material conforming to the suggested material specifications of this report. Our recommended equivalent fluid weights presented below are for conditions where the backfill material is placed level behind below grade walls. Recommended equivalent fluid weights are provided for imported Retaining Wall Backfill materials placed within a 1:1v line projected up from the base of the structure. The tabulated values presented below are based on a soil unit weight of 125 pounds per cubic foot. We recommend that the following lateral earth pressures (equivalent fluid weights) be used for the design of retaining structures where the backfill material is placed level behind the wall:

**Lateral Earth Pressures for Below Grade Walls**

<table>
<thead>
<tr>
<th>Wall Loading Condition</th>
<th>Backfill Condition</th>
<th>Lateral Earth Pressure Condition</th>
<th>Equivalent Fluid Weight (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Braced</td>
<td>Imported sand above groundwater level</td>
<td>At-rest, Drained and above groundwater level</td>
<td>65</td>
</tr>
<tr>
<td></td>
<td>Imported sand below groundwater level</td>
<td>At-rest, Undrained or below groundwater level</td>
<td>95*</td>
</tr>
<tr>
<td>Unbraced</td>
<td>Imported sand above groundwater level</td>
<td>Active, Drained and above groundwater level</td>
<td>40</td>
</tr>
</tbody>
</table>

*Includes unit weight of water at 62.4 pcf

Surcharges. The recommended equivalent fluid weights do not account for surcharge loads acting on the backfill. The surcharge from foundation loads can be neglected provided adjacent footings are setback behind a 1:1 line projected upward from the base of the wall. The lateral earth pressure from uniform surcharge loads can be estimated as 0.3 times the stress being applied at the ground surface. Traffic surcharges can be estimated as an additional 2 feet of soil cover, equal to a uniform pressure of 72 pounds per square foot. Fugro should provide additional recommendations if foundation loads act within the 1:1 line or other surcharges to retaining walls are anticipated.

Backfill and Drainage. For drained backfill conditions, drainage should be provided behind retaining walls to reduce the potential for the buildup of hydrostatic pressures. Retaining Wall Backfill Material should be placed between the wall and a 1h:1v backslope projected up from the heel of the retaining wall footing.

Retaining walls designed for drained loading conditions should be designed with weep holes or collector pipes to assist in the removal of water from the backfill, and to prevent the build up of hydrostatic pressures behind the wall. A continuous layer of granular drainage material consisting of either 1-foot of Drainage Material or Geocomposite Drain panels should be provided along the backside of retaining walls. The drainage material should be terminated 2 feet below the finished grade of the wall backfill, and be topped with on-site soil or topsoil. Retaining Wall Backfill and Drainage Materials should conform to the materials recommendations of this report.
4.7.2.1 Dynamic Earth Pressures

The dynamic earth pressures for below grade structures having braced non-yielding walls were estimated using procedures outlined in Kramer (1996). Using the design horizontal ground accelerations for the design earthquake as shown above, the additional force on the wall from earthquake loading is estimated to be about $29H^2$ (pounds per foot of wall). $H$ is the wall height in feet. The point of application of the seismic force can be assumed to act at approximately 0.63 $H$ above the base of the wall.

4.8 PAVEMENT DESIGN

Structural sections were estimated for asphalt concrete and Portland cement concrete pavements based on an R-value of 40 for the subgrade soils encountered at the site and traffic indices (TI) of 6 to 7.

4.8.1 Subgrade Preparation

The near surface soils encountered at the site may be loose and contain organic or other deleterious materials. Clearing and grubbing should be performed according to the recommendations of this report prior to beginning grading for pavement areas. As a minimum, we recommend that the existing soil be removed to a depth of at least 2 feet below the existing ground surface or 1 foot below the proposed structural section, whichever is deeper. The bottom of the excavation should then be scarified to a depth of at least 9 inches, moisture-conditioned, and compacted in-place to at least 95 percent relative compaction. Fill materials can then be placed to the base of the structural section according to the recommendations of this report. Fill materials placed within 3 feet of finished grade in pavement areas should be compacted to at least 95 percent relative compaction.

4.8.2 Asphalt Concrete Pavements

Structural sections for asphalt concrete pavements were estimated based on methods presented in the Caltrans Highway Design Manual. Structural section recommendations for flexible 2-layer pavements, asphalt concrete (AC) over aggregate base (AB), are provided in the table below.

<table>
<thead>
<tr>
<th>Traffic Index</th>
<th>Structural Section Thickness (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤6</td>
<td>3” AC over 5” AB</td>
</tr>
<tr>
<td>7</td>
<td>3.5” AC over 6” AB</td>
</tr>
</tbody>
</table>

The above structural section was calculated assuming sandy native soils having an R-value of at least 40. If areas are encountered that are underlain by clay or silty materials, we
recommend that 12 inches of sand meeting the requirement for Compacted Fill be provided below the structural section provided above.

Maintenance of asphalt concrete pavements should consist of periodic fog or slurry seals to reduce the potential for weathering. Deflection surveys can be performed to evaluate the structural capacity of pavements and the need for future overlays.

4.8.3 Portland Cement Concrete Pavements

Structural sections for Portland cement concrete pavements were estimated using the Portland Cement Association (PCA) design method for 3,000-psi strength Portland cement concrete for various estimates of the average daily truck traffic (ADTT). Portland cement concrete pavements should be designed with control joints, expansion joints, and load transfer provisions in accordance with PCA or other applicable guidelines.

<table>
<thead>
<tr>
<th>ADTT</th>
<th>Structural Section Recommendation (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤ 6</td>
<td>6&quot; PCC over 4&quot; AB</td>
</tr>
<tr>
<td>≤ 60</td>
<td>6.5&quot; PCC over 4&quot; AB</td>
</tr>
<tr>
<td>≤ 400</td>
<td>7&quot; PCC over 4&quot; AB</td>
</tr>
</tbody>
</table>

4.9 UTILITY TRENCHES AND PIPELINES

Material requirements for bedding material, pipe zone material and trench backfill are described in Section 4.2.2. Schematic trench details showing the cross sectional limits of the bedding, pipe zone, and trench backfill material are shown on Plates 7. Bedding, pipe zone, and trench backfill should be compacted to 90 percent relative compaction unless a higher degree of compaction is recommended for the area where the material is being placed. The upper 3 feet of trench backfill in pavement areas should be compacted to 95 percent relative compaction. Where pipes or utilities are to be constructed below structures or within building areas the bedding, pipe zone and trench backfill materials should be compacted to at least 95 percent relative compaction.

Bedding and Foundation Support. Utility pipes should be placed on properly prepared bedding. Bedding is select fill material placed between the trench subgrade and the bottom of the pipe. Where the soils exposed at the bottom of the utility trenches are soft and yielding, the foundation should be stabilized or be removed and replaced with properly compacted soil prior to placing the pipe. Gravel bedding can be used in lieu of sand bedding to assist in stabilizing the subgrade, if needed. At least 6 inches of bedding material should be provided below the pipe. Open graded materials, such as pea-gravel, should be encased in a filter fabric.
Pipe Zone Material. Pipe zone material is select fill material placed between the top of the bedding and at least 6 inches above the top of the pipe, and to at least 12 inches beyond the springline of the pipe. Compaction within the pipe zone should be performed such that the pipe is fully supported during compaction, and such that excessive deformation or damage to the pipe does not occur. Compaction above the springline or top of the pipe should not be performed until the fill placed below that elevation has been properly compacted.

Trench Backfill. Trench backfill is fill material placed above the pipe zone and to the finished grade or the base of other specified backfill materials (such as pavement structural sections). Trench backfill can consist of either on-site or imported fill material that complies with the recommendations of this report, and any other requirements for the area where the trench backfill is being placed.

4.9.1 Thrust Resistance

We understand from Wallace Group that some of the pipeline will be pressurized and have approximately 3 to 6 feet of soil cover on top of the pipe. Thrust blocks can be designed to resist lateral forces based on the passive resistance acting on the bearing side of the block, and the estimated frictional resistance acting along the base of the block. Thrust blocks should be designed with a minimum cover of 3 feet below finish grade. A passive resistance of 350 pounds per cubic foot, equivalent fluid weight, can be used for the design of thrust blocks located between 3 and 6 feet below the existing ground surface. We recommend that a coefficient of friction of 0.25 acting on the base of thrust blocks be used for design. For thrust blocks located below 6 feet below the existing ground surface, we recommend that a maximum allowable bearing pressure of 1000 pounds per square foot be used.

4.9.2 Existing Utilities

Existing buried utilities are likely present within the planned improvement areas. Existing utilities in the building or structure areas should be removed, relocated, and/or abandoned as part of site clearing and grubbing. Utilities that are present below the excavation for buildings or structures should be removed and replaced with compacted fill. If the utilities are not removed entirely, we recommend that they be filled with sand-cement slurry or concrete and that they be exposed and backfilled with properly compacted fill.

Conflicts commonly arise when utility lines or pipelines outside the building are located in close proximity to a structure. Generally, where a pipeline and backfill are located above a 1:1 line projected downward from the outside edge of a footing there is no conflict. Foundation construction and site preparation may also involve the construction of temporary slopes and shoring systems. The design of temporary slopes and shoring should consider support of adjacent foundations, utilities and pipelines. Particular attention should be made to pressurized lines that may rely on the lateral support of the ground to constrain the pipeline against movement.
4.9.2.1 Considerations for Foundations and Utilities

The proximity of foundations and utilities in relation to one another should consider: 1) the potential for the support of the foundation to be compromised by the presence of the utility trench backfill, and 2) the potential for the foundation to exert loads on the pipeline for which the pipe is not designed. Utility and service lines extending inside the building or tank footprint should be placed above the bearing level of the foundation. Utility lines that penetrate the building perimeter or tank footing should go through the footing stem wall or grade beam. The footing should be stepped down to encase the penetration into the building within the footing stem wall as recommended by the structural engineer.

These considerations regarding utilities and foundations are general. Poorly compacted backfill in pipeline trenches can result in settlement or impacts to adjacent structures that would normally not occur. Gravel bedding and granular backfill materials can serve as conduits that allow groundwater to infiltrate areas where it would not normally flow. The locations of existing and new utilities relative to foundations and grading should be considered in the foundations, structural, and grading plans for the project.

4.10 CORROSION CONSIDERATIONS

Corrosivity testing was performed on three selected sample obtained from the field exploration program by LA Testing of Los Alamitos, California. The results of the testing are presented in Appendix B. The corrosion tests were performed in accordance with Caltrans test methods.

Minimum resistivity values ranged between 2,000 and 12,000 ohm-cm. pH values ranged between 5.5 and 7.3. Chloride content tests were less than 42 parts per million (ppm). Soluble sulfate ranged from less that 42 ppm to 26 ppm. According to the Caltrans Corrosion Guidelines, a corrosive area is defined where "...the soil and/or water contains more than 500 ppm of chlorides, more than 2,000 ppm of sulfates, has a minimum resistivity of less than 1,000 ohm-cm, or has a pH less than 5.5." The sample of the clayey alluvium in DH-2 at 7.7 feet had a pH of 5.5 and is considered corrosive. The acidity could be due to the organics that we observed in the clayey alluvium.

4.11 DRAINAGE CONSIDERATIONS

Drainage should be provided such that surface water does not run over slopes or pond on pavements, slabs, or adjacent to foundations. Downspouts should be provided to collect roof drainage and direct the water to drainage pipes or areas away from the building. Concentrated flows and runoff should not be permitted to discharge on slopes. Down drains, solid pipes, or lined ditches should be provided to carry water to the base of the slope. Energy dissipation and erosion control devices should be provided at the outlet of drainage pipes and in areas of concentrated flow and runoff to reduce the potential for erosion. Landscaping and maintenance of slopes should be provided to assist vegetation to establish on slopes, graded areas, and reduce the potential for erosion.
4.12 CONSTRUCTION CONSIDERATIONS

4.12.1 Excavation

The soil materials encountered at the proposed water treatment plant and within the expected depths of excavation consist of a relatively thin thickness (1 to 5 feet) of artificial fill materials overlying clayey alluvium. The existing fill generally consists of silty sand. The alluvium generally consists of interbedded sand and very soft to soft wet clay that contains organics. The onsite soils can likely be excavated with typical construction equipment in good working order. These excavated materials may be wet and difficult to work with.

4.12.2 Groundwater Conditions

Groundwater was encountered at the proposed water treatment plant at depths ranging from approximately 14 to 17 feet below the existing ground surface during our June and July 2008 field exploration. Wet and saturated soils were encountered at depths of 7 to 8 feet below the existing ground surface and indicate possible perched groundwater conditions above and within the clayey alluvium (see Plate 3). Gravel layers were also encountered within the alluvium that can be highly pervious and relatively difficult to dewater.

The planned depth of excavation for the cleanwell structure is below the wet soils encountered and near the measured groundwater level. Groundwater and wet soil conditions will likely be encountered at this location. If encountered, dewatering should be provided to lower the groundwater below the depth of excavation prior to beginning the excavation. Dewatering should be performed such that water does not seep through sidewalls of the excavation, and is significantly below the excavation to allow for stabilization of the subgrade and proper compaction of fill material. The contractor should be responsible for both designing and maintaining the dewatering system for construction. If needed, gravel can be placed over the base of excavations to help stabilize wet subgrade conditions and provide a working mat for construction.

If needed, the contractor should submit a dewatering plan for review by the design consultant and geotechnical engineer. A qualified registered professional of California should prepare the dewatering plan. Dewatering facilities, such as sump pits, wells, and well points should be designed with filters such that sand and fine-grained materials are not removed from the soil during dewatering operations. Dewatering facilities should be installed prior to beginning excavation, and time should be allowed for lowering of the groundwater table before beginning excavation. Shoring systems, such as sheet piling, should be embedded adequately below the base of the excavation to cutoff groundwater and help stabilize the base of the excavation.

4.12.3 Temporary Slopes

Within the anticipated depths of excavation, the soil is anticipated to consist of loose to medium dense sand with varying amounts of silt and gravel, and soft to stiff clay and silt. The soils should not be considered capable of maintaining a stable vertical slope. Temporary slopes should be braced or sloped according to the requirements of OSHA. Based on review of OSHA
guidelines and the soil conditions encountered, the allowable temporary slope inclination will depend on the depth of the excavation. The soil conditions relative to the allowable temporary slope inclinations generally are weaker with depth and vary between locations.

As input to design, excavations without shoring that are shallower than 20 feet can likely be excavated at inclinations of 1.5H:1v or flatter for Type C soil conditions. Slopes should not be considered stable if seepage can daylight on the slope or groundwater is expected within the planned depths of excavation.

4.12.4 Shoring

Temporary slopes can be supported using trench shields, sheet pilings, or braced excavations. As discussed above, soil Type C will likely be encountered during construction activities.

"Dragging a shield" is a common method of providing worker safety during trenching and pipe construction. However, unless specific provisions exist to emplace the shield tight against the sidewalls, a shield provides no support for the trench sidewalls and should not be considered as an appropriate shoring system in pavement areas or adjacent to settlement sensitive structures or utilities.

According to OSHA, the lateral earth pressure acting on trench shoring can be estimated as a uniform soil pressure plus a surcharge for traffic loading.

For the Type C soil conditions, OSHA recommends that the active earth pressure acting on trench shoring be estimated as:

\[ \sigma_a = 80H + 72 \text{ psf for soil being retained above the water table} \]
\[ \sigma_a = 40H + u + 72 \text{ psf for soil being retained below the water table} \]

where:

\( \sigma_a \) is the uniform, active earth pressure acting on the shoring, in pounds per square foot (psf) with a level backslope

\( H \) is the height of the soil that is being retained in feet

\( u \) is the water pressure that increases at 62.4z, with \( z \) = depth in feet

"72 psf" is the traffic surcharge

Excavated material should generally be stockpiled away from excavations, or the shoring systems should be designed for the additional surcharge from the stockpiled material. The stock piled materials, or other surcharges, can be assumed to not influence the design of the shoring systems where the materials are located beyond a 1:1 line projected upward from the bottom edge of the trench.

5. CONTINUATION OF SERVICES

The geotechnical evaluation consists of an ongoing process involving the planning, design, and construction phases of the project. To provide this continued service, we recommend that the geotechnical engineer be provided the opportunity to review the project plans and specifications, and observe portions of the construction.
5.1 REVIEW OF CLEARWELL LOADING

A steel building is planned to be constructed on top of the clearwell structure. Loading information for the building is not available at this time. We recommend that Fugro review the clearwell structure relative to settlement considerations, foundation support, and grading once the loading information becomes available.

5.2 REVIEW OF PLANS AND SPECIFICATIONS

The geotechnical engineer should review the foundation and grading plans for the project. The purpose of the review is to evaluate if the plans and specifications were prepared in general accordance with the recommendations of this report.

5.3 GEOTECHNICAL OBSERVATION AND TESTING

Field exploration and site reconnaissance provides only a limited view of the geotechnical conditions of the site. Substantially more information will be revealed during the excavation and grading phases of the construction. Subsurface conditions, excavations and fill placement should be observed by the geotechnical professional during construction to evaluate if the materials encountered during construction are consistent with those assumed for this report.

6. REFERENCES


American Society of Civil Engineers (2005), ASCE Standard 7-05, Minimum Design Loads for Buildings and Other Structures.


Bengal Engineering (2001), “Foundation Redesign for Microfiltration and Membrane Filtration Facilities, Santa Barbara, California at Laguna County Sanitation District, Wastewater Treatment Plant, Santa Maria, Ca” undated.


County of Santa Barbara (2000), “Geotechnical Report, Laguna County, Dutard-Solomon Truck Sewer, County of Santa Barbara, Department of Public Works, Laguna County Sanitation District, Santa Barbara County, California”, May 16.


Santa Barbara County (2000a), Groundwater Assessment, County of Santa Barbara Department of Public Works, Laguna County Sanitation District, prepared by Santa Barbara County Public Works Department, Division of Solid Waste & Utilities, January.

Santa Barbara County (2000b), Geotechnical Report, Laguna County Dutard-Solomon Trunk Sewer, County of Santa Barbara, Department of Public Works, prepared by Santa Barbara County Public Works Department, Division of Transportation, dated June 2000.


Staal, Gardner & Dunne (1985), "Geotechnical Investigation, Laguna County Sanitation District, Wastewater Treatment Plant Modifications, Santa Barbara County, California", unpublished consultant report prepared for Kennedy/Jenks Engineer, dated November.


BASE MAP SOURCE: USGS Santa Maria, California 7.5' Quadrangle, revised 1982 and USGS Guadalupe, California 7.5' Quadrangle, revised 1982.

VICINITY MAP
Recycled Water System Improvements
Laguna County Sanitation District, California

PLATE 1
DESIGN RESPONSE SPECTRA
Recycled Water System Improvements
Laguna County Sanitation District, California

PLATE 4a
Response Spectra

Spectral Accel., Sa (g)

Period, T (sec)

Damping (%)
- 5
- 0.5

DESIGN RESPONSE SPECTRA
Recycled Water System Improvements
Laguna County Sanitation District, California

PLATE 4b
**Legend**

- Aggregate base Compacted to at least 95% relative compaction.

- Compacted Fill: Onsite or imported material placed below above or below grade structures compacted to at least 95% relative compaction (ASTM D1557).

- Undisturbed Alluvium

- Limits of alternative to excavate and replace compressible clay layer with compacted fill. Compact to at least 95% relative compaction.

- Limits of alternative to construct a temporary surcharge fill to consolidate compressible clay layer in-place prior to constructing the tank. Compacted to 85% relative compaction.

See test for excavation, compaction, and surcharge recommendations.

**SUMMARY OF TANK GRADING RECOMMENDATIONS**

Recycled Water System Improvements
Laguna County Sanitation District, California

PLATE 5
Legend

- Drainage Material: Minimum 2 feet of drainage material conforming to the recommendations of this report encased in filter fabric and placed according to the recommendations of this report.
  (combined with dewatering sumps or wells if needed).
- Structure Backfill Material: Imported material conforming to Caltrans Standard Specifications for Structure Backfill, Section 19-3.06 and compacted to at least 90% relative compaction (ASTM D1557).
- Undisturbed Alluvium
- Soil Cover: On-site material compacted to at least 90% relative compaction (ASTM D1557).

Notes:
1. If shoring option is used, shoring should be adequately embedded to provide cutoff and reduce potential for native wet soils to enter the excavation or a slurry seal should be provided.
2. If temporary slopes are used, dewatering should consist of lowering groundwater levels prior to excavating and significantly below the bottom of the excavation.

SUMMARY OF CLEARWELL STRUCTURE GRADING RECOMMENDATIONS
Recycled Water System Improvements
Laguna County Sanitation District, California
NOT TO SCALE

See text for description of trench materials and compaction requirements.
### General Notes

**Soil Texture Symbol**
- Sloped line in symbol column indicates transitional boundary
- Sampled and sampler dimensions listed in report text

#### Symbol for:
- **1** SPT Sampler, driven
- **2** CA Liner Sampler, driven
- **3** CA Liner Sampler, disturbed
- **4** Thin-walled Tube, pushed
- **5** Bulk Bag Sample (from cuttings)
- **6** CA Liner Sampler, Bagged
- **7** Hand Auger Sample
- **8** CME Core Sample
- **9** Pitcher Sample
- **10** Lexan Sample
- **11** Vibracore Sample
- **12** No Sample Recovered
- **13** Sonic Soil Core Sample

#### Sampler Driving Resistance
- Number of blows with 140 lb. hammer, falling 30" to drive sampler 1 ft. after seating sampler 6"; for example:
  - **Blows/ft** Description
  - **25** 25 blows drove sampler 12" after initial 6" of seating
  - **88/11** After driving sampler the initial 6" and 36 blows drove sampler through second 6" interval, and 50 blows drove sampler 5" into the third interval
  - **50/6** 50 blows drove sampler 6" after initial 6" of seating
  - **Rel/3** 50 blows drove sampler 3" during initial 6" seating interval

#### Blow counts for California Liner Sampler shown in ( )
- **Length of sample symbol approximates recovery length**
- **Classification of Soils per ASTM D2487 or D2488**
- **Geologic Formation noted in bold font at the top of interpreted interval**
- **Strength Legend**
  - **Q** = Unconfined Compressibility
  - **u** = Unconsolidated Undrained Triaxial
  - **T** = Traction
  - **p** = Pocket Penetrometer
  - **m** = Miniature Vane
- **Water Level Symbols**
  - **=** Initial or perched water level
  - **=** Final ground water level
  - **=** Seepage encountered
- **Rock Quality Designation (RQD) is the sum of recovered core pieces greater than 4 inches divided by the length of the cored interval.**

### KEY TO TERMS & SYMBOLS USED ON LOGS

<table>
<thead>
<tr>
<th>ELEVATION &amp; DEPTH</th>
<th>MATERIAL SYMBOL</th>
<th>SAMPLE NO.</th>
<th>BLOW COUNT</th>
<th>RECORDED (REV.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>-12</td>
<td>1</td>
<td></td>
<td>25</td>
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<td>-14</td>
<td>2</td>
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<td>5</td>
<td></td>
<td>18'/30'</td>
<td></td>
</tr>
<tr>
<td>-22</td>
<td>6</td>
<td></td>
<td>18'/30'</td>
<td></td>
</tr>
<tr>
<td>-24</td>
<td>7</td>
<td></td>
<td>20'/24'</td>
<td></td>
</tr>
<tr>
<td>-26</td>
<td>8</td>
<td></td>
<td>20'/24'</td>
<td></td>
</tr>
<tr>
<td>-28</td>
<td>9</td>
<td></td>
<td>20'/24'</td>
<td></td>
</tr>
<tr>
<td>-30</td>
<td>10</td>
<td></td>
<td>20'/24'</td>
<td></td>
</tr>
<tr>
<td>-32</td>
<td>11</td>
<td></td>
<td>20'/24'</td>
<td></td>
</tr>
<tr>
<td>-34</td>
<td>12</td>
<td></td>
<td>20'/24'</td>
<td></td>
</tr>
<tr>
<td>-36</td>
<td>13</td>
<td></td>
<td>20'/24'</td>
<td></td>
</tr>
<tr>
<td>-38</td>
<td>14</td>
<td></td>
<td>20'/24'</td>
<td></td>
</tr>
<tr>
<td>-40</td>
<td>15</td>
<td></td>
<td>20'/24'</td>
<td></td>
</tr>
<tr>
<td>-42</td>
<td>16</td>
<td></td>
<td>20'/24'</td>
<td></td>
</tr>
<tr>
<td>-44</td>
<td>17</td>
<td></td>
<td>20'/24'</td>
<td></td>
</tr>
<tr>
<td>-46</td>
<td>18</td>
<td></td>
<td>20'/24'</td>
<td></td>
</tr>
<tr>
<td>-48</td>
<td>19</td>
<td></td>
<td>20'/24'</td>
<td></td>
</tr>
</tbody>
</table>

**MATERIAL DESCRIPTION**
- Well graded GRAVEL (GW)
- Poorly graded GRAVEL (GP)
- Well graded SAND (SW)
- Poorly graded SAND (SP)
- Silty SAND (SM)
- Clayey SAND (SC)
- Silty, Clayey SAND (SC-SP)
- Elastic SILT (MH)
- SILT (ML)
- Silty CLAY (CL-ML)
- Fat CLAY (CH)
- Lean CLAY (CL)
- CONGLOMERATE
- SANDSTONE
- SILTSTONE
- MUDSTONE
- CLAYSTONE
- BASALT
- ANDESITE BRECCIA
- Paving and/or Base Materials
<table>
<thead>
<tr>
<th>ELEVATION (ft)</th>
<th>MATERIAL SYMBOL</th>
<th>MATERIAL DESCRIPTION</th>
<th>SURFACE EL: 135.9 ft +/- (rel. MSL datum)</th>
</tr>
</thead>
<tbody>
<tr>
<td>134</td>
<td></td>
<td>ARTIFICIAL FILL (af)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Silty CLAY (CL-ML): soft, gray-brown, dry</td>
<td></td>
</tr>
<tr>
<td>132</td>
<td></td>
<td>ALLUVIUM (Qa)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Silty SAND (SM): loose, light brown, moist</td>
<td></td>
</tr>
<tr>
<td>130</td>
<td></td>
<td>Clayey SILT (ML): soft, brown, moist to wet</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>- interlayered SILT (ML) and silty SAND (SM), with decayed rootlets, spongy</td>
<td></td>
</tr>
<tr>
<td>128</td>
<td></td>
<td>Lean CLAY (CL): firm, black, moist, with 6&quot; interlayered wet clayey SAND (SC), with organic</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>- with fine sand seams</td>
<td></td>
</tr>
<tr>
<td>124</td>
<td></td>
<td>Sandy Lean CLAY (CL): stiff to very stiff, brownish-gray, moist</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Silty SAND (SM): loose, light brown, moist</td>
<td></td>
</tr>
<tr>
<td>122</td>
<td></td>
<td>- interlayered sandy SILT (ML): loose, light brown, wet, with reddish-brown motting</td>
<td></td>
</tr>
<tr>
<td>118</td>
<td></td>
<td>1-foot thick layer of lean CLAY (CL)</td>
<td></td>
</tr>
<tr>
<td>116</td>
<td></td>
<td>Silty SAND (SM): medium-dense to dense, light brown, wet</td>
<td></td>
</tr>
<tr>
<td>112</td>
<td></td>
<td>- interbedded gravel layers</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>- clayey GRAVEL (GC): medium dense, orange-brown, wet, subrounded to 2&quot; diameter</td>
<td></td>
</tr>
<tr>
<td>108</td>
<td></td>
<td>ORCUTT FORMATION (Qo)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sandy lean CLAY (CL): stiff, orange-brown, moist</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>- no recovery with Shelby tube sampler, grab sample with SPT sampler</td>
<td></td>
</tr>
<tr>
<td>100</td>
<td></td>
<td>Clayey SILT (ML): very stiff, orange-brown, moist, with interbedded fat CLAY (CH): very stiff, orange-brown, moist, with dark brown motting and staining</td>
<td></td>
</tr>
</tbody>
</table>

**LOG OF NO. DH-01**
Recycled Water System Improvements
Laguna County Sanitation District, California

**PLATE A-2a**
<table>
<thead>
<tr>
<th>ELEVATION, ft</th>
<th>DEPTH, ft</th>
<th>MATERIAL SYMBOL</th>
<th>SAMPLE NO</th>
<th>SAMPLER</th>
<th>BLOW/COUNT</th>
</tr>
</thead>
<tbody>
<tr>
<td>64</td>
<td>42</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>62</td>
<td>44</td>
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<td>10</td>
<td></td>
<td>30</td>
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<tr>
<td>60</td>
<td>46</td>
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<td></td>
<td></td>
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<tr>
<td>66</td>
<td>48</td>
<td></td>
<td>11</td>
<td></td>
<td>(78)</td>
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<td>50</td>
<td>50</td>
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<td>64</td>
<td>52</td>
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<td>62</td>
<td>54</td>
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<tr>
<td>60</td>
<td>56</td>
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<td>58</td>
<td>58</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>60</td>
<td>60</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>74</td>
<td>62</td>
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<td>72</td>
<td>64</td>
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<td>74</td>
<td>66</td>
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<td>72</td>
<td>68</td>
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<td>68</td>
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<td>70</td>
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<tr>
<td>66</td>
<td>72</td>
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<td>58</td>
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<tr>
<td>50</td>
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<td>68</td>
<td>76</td>
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</tr>
<tr>
<td>58</td>
<td>78</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**LOCATION:** Laguna CSD, Proposed NE Steel Tank, 5' south of CPT C-3

**SURFACE EL:** 135.9 ft +/- (rel. MSL datum)

**MATERIAL DESCRIPTION**
- Silty SAND (SM): dense, light brown, wet, with black inclusions/streaks
  - interbedded layers of clayey silt (ML)
- very dense

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

**COMPLETION DEPTH:** 49.5 ft
**DEPTH TO WATER:** 15.0 ft
**BACKFILLED WITH:** Cuttings/Bentonite
**DRILLING DATE:** July 28, 2008

**DRILLING METHOD:** 5-inch diameter mud rotary
**HAMMER TYPE:** Automatic Trip
**DRILLED BY:** S/G Drilling Company
**LOGGED BY:** C Lovato
**CHECKED BY:** J Blanchard

**LOG OF NO. DH-01**
Recycled Water System Improvements
Laguna County Sanitation District, California

PLATE A-2b
**LOCATION:** 30' east of existing Pump House Building

**SURFACE EL:** 138.6 ft +/- (rel. MSL datum)

**MATERIAL DESCRIPTION**

<table>
<thead>
<tr>
<th>ELEVATION</th>
<th>MATERIAL SYMBOL</th>
<th>MATERIAL DESCRIPTION</th>
<th>UNIT WET WEIGHT</th>
<th>UNIT DRY WEIGHT</th>
<th>WATER CONTENT</th>
<th>COMPACTION</th>
<th>PLASTICITY INDEX</th>
<th>UNDRAINED SHEAR STRENGTH</th>
</tr>
</thead>
<tbody>
<tr>
<td>138</td>
<td></td>
<td>ARTIFICIAL FILL (af)</td>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>138</td>
<td></td>
<td>Silty SAND (SM): medium dense, tan, dry</td>
<td>105 133 2</td>
<td>105 133 2</td>
<td>2</td>
<td>14</td>
<td></td>
<td></td>
</tr>
<tr>
<td>136</td>
<td></td>
<td>SAND with gravel (SP): medium-dense, tan, moist</td>
<td>102 89 14</td>
<td>102 89 14</td>
<td>7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>132</td>
<td></td>
<td>ALLUVIUM (Gaq)</td>
<td></td>
<td></td>
<td></td>
<td>30</td>
<td>57 37 u 0.7</td>
<td></td>
</tr>
<tr>
<td>130</td>
<td></td>
<td>SAND with silt (SP-SM): loose, light brown, moist</td>
<td>112 65 30</td>
<td>112 65 30</td>
<td>30</td>
<td>57 37 u 0.7</td>
<td></td>
<td>1.06</td>
</tr>
<tr>
<td>130</td>
<td></td>
<td>Fat CLAY (CH): very soft to soft, black, moist to wet, with organics and pinhole voids, iron oxide staining around voids</td>
<td>112 65 30</td>
<td>112 65 30</td>
<td>30</td>
<td>57 37 u 0.7</td>
<td></td>
<td>1.06</td>
</tr>
<tr>
<td>128</td>
<td></td>
<td>- soft to firm</td>
<td></td>
<td></td>
<td></td>
<td>57 37 u 0.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>128</td>
<td></td>
<td>Clayey SAND (SC): medium-dense, dark grayish brown, wet, faint mottles</td>
<td>128 112 14</td>
<td>128 112 14</td>
<td>30</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>124</td>
<td></td>
<td>SAND with silt (SP-SM): medium-dense, light brown, wet</td>
<td>124 65 30</td>
<td>124 65 30</td>
<td>30</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>- with gravel and flowing sands resulting in high blow counts</td>
<td></td>
<td></td>
<td></td>
<td>57 37 u 0.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>- interbedded layers of gravel</td>
<td></td>
<td></td>
<td></td>
<td>57 37 u 0.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>ORCUTT FORMATION (Qo)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Interbedded firm and hard lean CLAY (CL): orange-brown, moist</td>
<td>121 32 32</td>
<td>121 32 32</td>
<td>6</td>
<td></td>
<td>1.5</td>
<td></td>
</tr>
</tbody>
</table>

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

**COMPLETION DEPTH:** 31.5 ft

**DEPTH TO WATER:** 17.0 ft

**BACKFILLED WITH:** Cuttings

**DRILLING DATE:** July 26, 2008

**DRILLING METHOD:** 8-inch diameter hollow stem auger

**HAMMER TYPE:** Automatic Trip

**DRILLED BY:** SG Drilling Company

**LOGGED BY:** C Lovato

**CHECKED BY:** J Blanchard

**LOG OF NO. DH-02**

Recycled Water System Improvements
Laguna County Sanitation District, California

PLATE A-3
GRAIN SIZE CURVES
Recycled Water System Improvements
Laguna County Sanitation District, California

PLATE B-1
**PLASTICITY CHART**

Recycled Water System Improvements
Laguna County Sanitation District, California

PLATE B-2
COHESION, ksf

ANGLE OF INTERNAL FRICTION, deg

LOCATION

DEPTH, ft

MOISTURE CONTENT, %

UNIT DRY WEIGHT, pcf

MATERIAL DESCRIPTION

SAMPLE CONDITION

DIRECT SHEAR TEST RESULTS

Recycled Water System Improvements
Laguna County Sanitation District, California

PLATE B-3
COMPACtion TEST RESULTS
Recycled Water System Improvements
Laguna County Sanitation District, California

LEGEND (location) depth, ft CLASSIFICATION MAXIMUM UNIT DRY WEIGHT, pcf OPTIMUM WATER CONTENT, %
O DH-01 2.0-5.0 Silty SAND (SM) 121.5 11.5
LOCATION
DEPT, ft
INITIAL MOISTURE CONTENT, %
UNIT DRY WEIGHT,pcf
MATERIAL DESCRIPTION
SAMPLE CONDITION

DH-01
10.5
22
100
Lean CLAY (CL)
Undisturbed

CONSOLIDATION TEST RESULTS
Recycled Water System Improvements
Laguna County Sanitation District, California

PLATE B-5a
CONsolidation test results
Recycled water system improvements
Laguna County Sanitation District, California

Location: DH-01
Depth: 29.5 ft
Initial moisture content (%): 23
Unit dry weight (pcf): 103
Material description: Sandy Lean Clay (CL)
Sample condition: Undisturbed
LOCATION
DEPTH, ft
INITIAL MOISTURE CONTENT, %
UNIT DRY WEIGHT,pcf
MATERIAL DESCRIPTION
SAMPLE CONDITION

DH-01
38
28
95
Clayey SILT (ML)
Undisturbed

CONSOLIDATION TEST RESULTS
Recycled Water System Improvements
Laguna County Sanitation District, California

PLATE B-5c
CONSOLIDATION TEST RESULTS
Recycled Water System Improvements
Laguna County Sanitation District, California

LOCATION
DEPTH, ft
INITIAL MOISTURE CONTENT, %
UNIT DRY WEIGHT,pcf
MATERIAL DESCRIPTION
SAMPLE CONDITION

DH-02
8.4
46
73
Fat CLAY (CH)
Undisturbed

PLATE B-5d
CONSOLIDATION TEST RESULTS
Recycled Water System Improvements
Laguna County Sanitation District, California

PLATE B-5e
LEGEND

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>DEPTH (ft)</th>
<th>CLASSIFICATION</th>
<th>DRY DENSITY (pcf)</th>
<th>% MOISTURE</th>
<th>S_u (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DH-01</td>
<td>10.0</td>
<td>Lean CLAY (CL)</td>
<td>103</td>
<td>21</td>
<td>1139</td>
</tr>
<tr>
<td>DH-02</td>
<td>7.5</td>
<td>Fat CLAY (CH)</td>
<td>86</td>
<td>30</td>
<td>677</td>
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</tbody>
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UNCONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS
Recycled Water System Improvements
Laguna County Sanitation District, California

PLATE B-6
### SAND EQUIVALENT TEST RESULTS

<table>
<thead>
<tr>
<th>Sample Number</th>
<th>Depth (ft)</th>
<th>Clay Readings (inches)</th>
<th>Sand Readings (inches)</th>
<th>Sand Equivalent (SE)</th>
<th>Soil Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>DH-1-A</td>
<td>2 - 5</td>
<td>12.9</td>
<td>2.4</td>
<td>18</td>
<td>Silty SAND (SM)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>12.7</td>
<td>2.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>TP-1-A</td>
<td>0</td>
<td>8.8</td>
<td>3.9</td>
<td>46</td>
<td>Poorly-graded SAND with silt (SP-SM)</td>
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<tr>
<td></td>
<td></td>
<td>7.7</td>
<td>3.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>TP-1-B</td>
<td>6</td>
<td>13.3</td>
<td>2.5</td>
<td>18</td>
<td>Silty SAND (SM)</td>
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<td></td>
<td></td>
<td>13.1</td>
<td>2.2</td>
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<td></td>
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<tr>
<td>TP-2-A</td>
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<td>13.3</td>
<td>1.7</td>
<td>13</td>
<td>Clayey SAND (SC)</td>
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<tr>
<td></td>
<td></td>
<td>12.7</td>
<td>1.7</td>
<td></td>
<td></td>
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<tr>
<td>TP-3-A</td>
<td>0</td>
<td>13.5</td>
<td>2.0</td>
<td>15</td>
<td>Silty SAND(SM)</td>
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<td></td>
<td></td>
<td>13.0</td>
<td>2.1</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Performed by: N. Lehr and A. Bajuniemi  
Test Method: ASTM D2419
SAMPLE NO.: DH-2-B
DEPTH: 1 to 5 Feet
DESCRIPTION: Silty SAND (SM) and SAND with gravel (SP)
DATE TESTED: 8/14/08
TEST METHOD: ASTM D2844

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>A</th>
<th>B</th>
<th>C</th>
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</thead>
<tbody>
<tr>
<td>INITIAL MOISTURE (%)</td>
<td>8</td>
<td>8</td>
<td>8</td>
</tr>
<tr>
<td>MOISTURE AT COMPACTION (%)</td>
<td>13.5</td>
<td>12.4</td>
<td>11.3</td>
</tr>
<tr>
<td>DRY UNIT WEIGHT (pcf)</td>
<td>107.9</td>
<td>108.9</td>
<td>109.2</td>
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<tr>
<td>EXUDATION PRESSURE (psi)</td>
<td>183</td>
<td>278</td>
<td>573</td>
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<tr>
<td>EXPANSION PRESSURE (psf)</td>
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<td>0</td>
<td>0</td>
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<tr>
<td>R-VALUE</td>
<td>49</td>
<td>57</td>
<td>70</td>
</tr>
<tr>
<td>R-VALUE BY EXPANSION (feet)</td>
<td>na</td>
<td></td>
<td></td>
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<tr>
<td>R-VALUE AT 300 psi EXUDATION PRESSURE</td>
<td>58</td>
<td></td>
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R-VALUE TEST RESULTS
Recycled Water System Upgrade
Laguna County Sanitation District, California

PLATE B-8
Fugro via CTM-643, CTM-417M, EPA 300.0M

<table>
<thead>
<tr>
<th>Sample ID</th>
<th>pH</th>
<th>Chloride (ppm)</th>
<th>Sulfate (wt %)</th>
<th>Resistivity (ohms-cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DH-2 #3 @ 7.7 ft</td>
<td>5.3</td>
<td>&lt;26</td>
<td>&lt;0.0026</td>
<td>2000</td>
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<tr>
<td>DH-2 #4B @ 1-5 ft</td>
<td>6.3</td>
<td>&lt;42</td>
<td>&lt;0.0042</td>
<td>12000</td>
</tr>
<tr>
<td>DH-1#5 @ 12 ft</td>
<td>7.3</td>
<td>&lt;97</td>
<td>&lt;0.0037</td>
<td>2000</td>
</tr>
</tbody>
</table>

Note: Sample received in acceptable condition unless otherwise noted. This report may not be reproduced except in full, without written approval by LA Testing. Unless otherwise noted, the results in this report have not been blank corrected.

JSL/MRH
Analyst

AIHA Accredited - Laboratory ID #101650
Page 1 of 1

CORROSION TEST RESULTS
Recycled Water System Improvements
Laguna County Sanitation District, California
PLATE B-9
PERCENT PASSING NO. 200 SIEVE TEST RESULTS

<table>
<thead>
<tr>
<th>Sample Number</th>
<th>Depth (ft)</th>
<th>Percent Passing No. 200 Sieve (%)</th>
<th>Soil Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP-1-A</td>
<td>0</td>
<td>16</td>
<td>Silty SAND (SM)</td>
</tr>
<tr>
<td>TP-2-A</td>
<td>0</td>
<td>31</td>
<td>Clayey SAND (SC)</td>
</tr>
<tr>
<td>TP-2-B</td>
<td>3</td>
<td>25</td>
<td>Silty SAND (SM)</td>
</tr>
<tr>
<td>TP-3-A</td>
<td>0</td>
<td>70</td>
<td>Sandy lean CLAY (CL)</td>
</tr>
</tbody>
</table>

Performed by: A. Bajuniemi
Test Method: ASTM D1140
Expansion Index Test Results

Sample: TP-2 Bulk "B"
Depth: 3 feet

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Water Content, %</td>
<td>11</td>
</tr>
<tr>
<td>Initial Dry Unit Weight, pcf</td>
<td>108.7</td>
</tr>
<tr>
<td>Assumed Specific Gravity, G_s</td>
<td>2.70</td>
</tr>
<tr>
<td>Degree of Saturation, %</td>
<td>54.2</td>
</tr>
<tr>
<td>Final Water Content, %</td>
<td>23.7</td>
</tr>
<tr>
<td>$E_{I_{measured}}$</td>
<td>74</td>
</tr>
<tr>
<td>$E_{I_{60}}$</td>
<td>78</td>
</tr>
<tr>
<td>ASTM Expansion Potential</td>
<td>Medium</td>
</tr>
<tr>
<td>Description:</td>
<td>Sandy lean CLAY(CL)</td>
</tr>
</tbody>
</table>
COLOR LEGEND FOR FRICTION RATIO TRACES

<table>
<thead>
<tr>
<th>Zone</th>
<th>Soil Behavior Type</th>
<th>U.S.C.S.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Sensitive Fine-grained</td>
<td>OL-CH</td>
</tr>
<tr>
<td>2</td>
<td>Organic Material</td>
<td>OL-OH</td>
</tr>
<tr>
<td>3</td>
<td>Clay</td>
<td>CH</td>
</tr>
<tr>
<td>4</td>
<td>Silty Clay to Clay</td>
<td>CL-CH</td>
</tr>
<tr>
<td>5</td>
<td>Clayey Silt to Silty Clay</td>
<td>MH-CL</td>
</tr>
<tr>
<td>6</td>
<td>Sandy Silt to Clayey Silt</td>
<td>ML-MH</td>
</tr>
<tr>
<td>7</td>
<td>Silty Sand to Sandy Silt</td>
<td>SM-ML</td>
</tr>
<tr>
<td>8</td>
<td>Sand to Silty Sand</td>
<td>SM-SP</td>
</tr>
<tr>
<td>9</td>
<td>Sand</td>
<td>SW-SP</td>
</tr>
<tr>
<td>10</td>
<td>Gravelly Sand to Sand</td>
<td>SW-GW</td>
</tr>
<tr>
<td>11</td>
<td>Very Stiff Fine-grained *</td>
<td>CH-CL</td>
</tr>
<tr>
<td>12</td>
<td>Sand to Clayey Sand *</td>
<td>SG-5M</td>
</tr>
</tbody>
</table>

* overconsolidated or cemented

CPT CORRELATION CHART
(Robertson and Campanella, 1984)

KEY TO CPT LOGS
Recycled Water System Improvements
Laguna County Sanitation District, California

PLATE C-1
LOG OF C-01
Recycled Water System Improvements
Laguna County Sanitation District, California
LOG OF C-02
Recycled Water System Improvements
Laguna County Sanitation District, California
LOG OF C-03
Recycled Water System Improvements
Laguna County Sanitation District, California
LOG OF C-04
Recycled Water System Improvements
Laguna County Sanitation District, California