UPDATED GEOTECHNICAL INVESTIGATION REPORT
DENITRIFICATION AND MASTER PLAN IMPROVEMENTS PROJECT
WASTEWATER TREATMENT PLANT
DISCOVERY BAY, CALIFORNIA

BSK PROJECT NO.: G19-194-11L

PREPARED FOR:
TOWN OF DISCOVERY BAY CSD
1800 WILLOW LAKE ROAD
DISCOVERY BAY, CALIFORNIA 94505

March 31, 2021
March 31, 2021

Town of Discovery Bay CSD
1800 Willow Lake Road
Discovery Bay, California 94505

ATTENTION:  c/o Mr. Gregory Harris – HERWIT Engineering (gharris@herwit.com)

SUBJECT:  Updated Geotechnical Investigation Report
Denitrification and Master Plan Improvements Project
Wastewater Treatment Plant
Discovery Bay, California

Dear Mr. Harris:

BSK Associates (BSK) is pleased to submit our updated geotechnical engineering investigation report for the proposed improvements at the Discovery Bay Wastewater Treatment Plant (WWTP) in Discovery Bay, California. BSK had recently provided recommendations for improvements at Plant No. 1 and No. 2 of the WWTP in a report dated February 20, 2020. Since then, we have been informed that the Town of Discovery Bay would like to eliminate the originally planned improvements at Plant No. 1 (located at 2305 Cherry Hills Drive) and move the planned improvements to Plant No. 2 (located at 17501 Highway 4) as part of their overall Master Plan Improvements Project.

The enclosed updated report describes the supplemental geotechnical investigation performed at Plant No. 2 and includes our supplemental geotechnical findings, conclusions, and recommendations for foundations, below-grade walls, pavements, and earthwork for the project. Note that we have removed pertinent figures and discussions related to the originally planned improvements at Plant No. 1.

In summary, it is our opinion that the project site (Site) defined in the Project Description section of the updated report is suitable for the proposed improvements provided the recommendations presented in this report are incorporated in the design and construction of the project. The primary geotechnical concerns at the Site are the potential for strong ground shaking to affect the Site during a future significant seismic event (typical of the entire San Francisco Bay Area), the presence of shallow groundwater and associated hydrostatic and buoyancy pressures, and the presence of surficial soils with high organic content or soils containing peat within the planned excavation for the new oxidation ditch No. 4 and underneath new at grade ancillary structures. Ground shaking can be addressed by incorporating the

seismic design parameters presented herein and other seismically related aspects of the 2019 California Building Code (CBC) and ASCE 7-16 into the design of the proposed improvements. Additional information on our investigative methods and our specific recommendations for design and construction are contained in this report.

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

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

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

Sincerely,

BSK Associates, Inc.

Cristiano Melo PE, GE #2756
Livermore Branch Manager

Carrie L. Foulk, PE, GE #3016
Senior Geotechnical Engineer
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1. INTRODUCTION

This updated report presents the results of our geotechnical investigation for the planned Denitrification Master Plan Improvements Project for the Wastewater Treatment Plant (WWTP) in Discovery Bay, California. A Vicinity Map showing the location of the project site (Site) is presented on Figure 1. This report contains a description of our site investigation methods and findings at Plant No. 2, including field and laboratory data. Based on these findings, this report presents conclusions regarding the geotechnical concerns of the planned improvements. It also provides recommendations for the design of the planned structures and construction considerations. Note that we have removed pertinent figures and discussions related to the originally planned improvements at Plant No. 1.

1.1 Project Description

The Discovery Bay wastewater treatment plant (WWTP) has two general locations that are proximate to each other – Plant No. 1 and Plant No. 2. Plant No. 1 is located at 2305 Cherry Hills Drive and Plant No. 2 is located at 17501 Highway 4 in Discovery Bay. The planned improvements will be located at Plant No. 2 and will include the following:

- As shown on the Site Plan, Figure 2, two denitrification basins measuring approximately 100 ft by 50 ft and embedded about 15 to 20 feet below the existing ground surface (BGS) will be constructed north of the existing Oxidation Ditch No. 2 and No. 3. These basins will be supported on concrete mat foundations.
- New oxidation ditch (Oxidation Ditch No. 4) will be located immediately east of Oxidation Ditch No. 3 and will be approximately 8 feet in depth and approximately 350 feet in length and 120 feet wide. This structure includes a “gate box” and rotors. The sidewalls and bottom of the new oxidation ditch will consist of reinforced shotcrete.
- Three below-grade pump stations, approximately 8 feet deep each, will be constructed near the north side of Oxidation Ditches Nos. 2, 3 and 4.
- A third denitrification basin will be constructed immediately north of Oxidation Ditch No. 4 and will have overall dimensions similar to the other two basins discussed above.

Additional improvements for the project include installation of new underground utility lines to be connected to the new basins and oxidation ditch, rehabilitation of distressed pavement within Plant No. 2, ancillary at grade structures, and possibly new exterior concrete flatwork.

Overall site grades are expected to remain close to existing elevations and cuts and fills during construction are anticipated to be limited to less than about 5 feet in height. Excavations for the new basins, oxidation ditch, and associated underground utility lines are anticipated to be up to about 18 feet deep BGS.
If the actual project description differs significantly from that described above, especially the amount of grading, we should be notified so that we may review our recommendations presented herein for applicability.

1.2 Purpose and Scope of Services

The purpose of this our original and supplemental investigations was to explore and evaluate the subsurface conditions at the Site in order to provide geotechnical input for the design and construction of the planned improvements and the associated earthwork at Plant No. 2. The scope of services, as outlined in our August 9, 2019 and August 19, 2020 proposals (Proposal No. GL19-17862), consisted of field investigation, laboratory testing, engineering analysis, and preparation of this updated report.

Our investigations specifically exclude the assessment of site environmental characteristics, particularly those involving hazardous substances. Our scope of services did not include the evaluation of contaminants in the soil, water, or air.

1.3 Previous Investigations

Previous investigations were performed near the Site located at Plant No. 2 by Kleinfelder. These investigations were presented in the following documents:

1. Revised Geotechnical Services Report, Proposed Filter Complex, Plant 2, Discovery Bay WWTP, Discovery Bay, California, dated January 26, 2015 by Kleinfelder (File No. 20153350,001A),
2. Geotechnical Services Report, Discovery Bay West-Master Plan Improvements, Wastewater Treatment Plant, Discovery Bay, California, dated January 9, 2013, Kleinfelder (File No. 131113.G01),
3. Report, First Quarter 2006, Groundwater Monitoring, Town of Discovery Bay, Wastewater Treatment Plan No. 2, Discovery Bay, California, dated April 17, 2006 (File No. 51977.E01),
4. Supplemental Geotechnical Report, Wastewater Treatment Plant Improvements, Discovery Bay, California, dated September 12, 2003, Kleinfelder (File No. 34802.G01), and
5. Preliminary Geotechnical Investigation Report, Proposed Wastewater Treatment Plant, Discovery Bay, California, dated May 12, 1999, Kleinfelder (File No. 43-1149-01.GEO).

Pertinent information from these previous reports was considered in the preparation of this updated report. Logs of borings and test pits and well data from these previous investigations that are proximate to the Site located at Plant No. 2 are included in Appendix E. The approximate location of these previous exploration points are shown on Figure 2.
2. SITE INVESTIGATION

2.1 Field Investigation

Our original field investigation was performed on September 6th and October 31st, 2019 to evaluate the subsurface conditions at both Plant No. 1 and Plant No. 2 for the planned construction. The field investigation consisted of advancing two (2) Cone Penetration Tests (CPTs), labeled CPT-1 and CPT-2, to a depth of approximately 50 feet BGS each and drilling two (2) borings, labeled B-1 and B-2 to a depth of approximately 31½ feet BGS. CPT-1 and boring B-1 were located at Plane No. 1, while CPT-2 and boring B-2 were located at Plant No. 2 as approximately shown on Figure 2.

Our supplemental field investigation was performed on September 23rd and 24th, 2020, to evaluate the subsurface conditions at the planned locations for the third denitrification basin and Oxidation Ditch No. 4. The supplemental investigation consisted of drilling two borings, labeled B-20-1 and B-20-2, to a depth of approximately 31½ feet BGS and excavating four test pits, labeled TP-20-1 through TP-20-4, to a depth of approximately 15 feet BGS as shown on Figure 2.

Prior to the subsurface exploration, Underground Service Alert (USA) was notified to provide utility clearance and each exploration location was cleared for detectable underground utilities by GeoTech Utility Locating of Moraga, California. Drilling permits were obtained from the Contra Costa County Environmental Health Division (County). Upon completion of the field investigation, the borings and CPTs were backfilled with cement grout under the supervision of a County inspector and the upper 6 inches of boring B-2 (located in a paved area) was patched with Quikrete. The test pits were backfilled with excess soil cuttings and tamped down using the backhoe bucket. Excess soil cuttings generated by the borings during drilling were left in unimproved areas of the Site.

The locations of our exploration points were estimated by our field representative based on rough measurements from existing features at the Site. The elevations shown on the boring logs were estimated using the elevation information available on Sheets C-203 and C-204 of the grading plans entitled Discovery Bay Community Services District, Denitrification & Master Plan Upgrades Project, Plant No. 2, Paving & Grading – Plan, dated March 2021 prepared by HERWIT Engineering (Job No. 2019-135 T01). As such the elevations and locations of the exploration points should be considered approximate to the degree implied by the methods used.

2.1.1 Auger Borings

The borings were drilled using a truck-mounted drill rig to a depth of approximately 31½ feet BGS by Taber Drilling of West Sacramento, California. The borings were logged by a geologist of BSK Associates (BSK). Relatively undisturbed samples of the subsurface materials were obtained using a split spoon sampler with a 2.5-inch inside diameter (I.D.) and a 3-inch outside diameter (O.D.) fitted with stainless steel liners. In addition, a 1.4-inch I.D. Standard Penetration Test (SPT) sampler was driven at selected depths in general accordance with ASTM D1586 test procedures. The SPT sampler was used without liners. The
samplers were driven 18 inches using a 140-pound, automatic trip hammer falling 30 inches, and blow counts for successive 6-inch penetration intervals were recorded. The blow counts were reported on the final boring logs. After the sampler was withdrawn from the boreholes, the samples were removed, sealed to reduce moisture loss, labeled, and returned to our laboratory. Prior to sealing the samples, strength characteristics of the cohesive soil samples recovered were evaluated using a hand-held pocket penetrometer. The results of these tests are shown adjacent to the samples on the boring logs.

Soil determinations made in the field, based on visual/manual assessment of the auger cuttings and samples, were re-evaluated in the laboratory after further examination and testing. Where laboratory tests were performed, the results appear in the final boring logs, except for the corrosion test results, which are found only in Appendix B. Final soil classification was assessed through the judgement of a responsible Geotechnical Engineer supplemented with the laboratory testing at various intervals in general accordance with the ASTM Standard Practice for Classification of Soils for Engineering Purposes (D2487). A summary of the Unified Soil Classification System (USCS), adapted by ASTM D2487 and D2488 is presented in Appendix A, Figure A-1. The Soil Description Key and Log Key are presented on Figure A-2 and A-3. Sample classifications, blow counts recorded during sampling, and other related information are presented on the soil boring logs within Appendix A. Discussion of the subsurface conditions encountered at the Site is presented in the “Subsurface Conditions” section of this report.

2.1.2 Cone Penetration Tests

A cone penetration test probe was advanced to a depth of approximately 50 feet BGS. Taber Drilling of West Sacramento, California was subcontracted to provide CPT services. The CPT was performed using an integrated electronic cone system in accordance with ASTM D3441. The cone has a tip area of 10 square centimeters, a friction sleeve area of 150 square centimeters, and a ratio of end area friction sleeve to tip end area equal to 0.80. The cone bearing (Qc) and sleeve friction (Fs) were measured and recorded during the tests at 5 centimeters (about 2 inch) depth intervals.

The cone system was pushed using a 40,000-pound, all-wheel drive, CPT rig, having a down pressure capacity of approximately 20 tons. The information gathered from the CPTs was used for identifying potentially liquefiable and soft soils and for foundation design. The correlated CPT data collected from our CPT (cone resistance, friction ratio, pore pressure, and soil behavior type) versus penetration depth below the existing ground surface, generated using the CPT liquefaction assessment computer software CLiq2, is presented in Appendix B.

The stratigraphic interpretation of the CPT data was performed based on relationships between cone resistance (also known as tip resistance) and sleeve friction versus penetration depth. The friction ratio, which is sleeve friction divided by cone resistance, is a calculated parameter which is used to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone resistance and generate large excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone

2 CLiq v2.0 by Geoligmoidi
resistance and generate small excess pore water pressures. The interpretation of soil properties from the cone data has been carried out using correlations developed by Robertson et al, 1990\textsuperscript{3}, and Lunne, Robertson & Powell, 1997\textsuperscript{4}. It should be noted that it is not always possible to clearly identify a soil type based on cone resistance and sleeve friction. In these situations, experience and judgment and an assessment of the pore pressure dissipation data should be used to infer the soil behavior type.

2.1.3 Exploratory Test Pits

Four exploratory test pits were excavated using a backhoe equipped with a 36-inch bucket to a depth of approximately 15 feet BGS. The test pits were logged by a BSK field geologist and soil samples were obtained from the test pits at selected depths for laboratory testing. Strength characteristics of the cohesive soils were evaluated using a hand-held pocket penetrometer. The test pits were loosely backfilled with the excavated spoils and tamped down using the backhoe bucket. Sample classifications and other related information are presented on the tabulated test pit logs included at the end of Appendix A.

2.2 Laboratory Testing

Laboratory tests were performed on selected soil samples to evaluate their physical characteristics and engineering properties. The laboratory testing program included dry unit weight and moisture content, Atterberg limits, unconsolidated-undrained triaxial compression (TXUU), organic content, Resistance (R)-Value testing, and moisture-density relationship testing. Most of the laboratory test results are presented on the boring logs. The results of the Atterberg limits, TXUU, R-Value, and moisture-density relationship tests are presented graphically in Appendix C.

Analytical testing was performed as part of our original investigation on samples obtained from an approximate depth of 2½ feet BGS in boring B-2 to assist in evaluating the corrosion potential of the near-surface soils at Plant No. 2. No additional analytical testing was performed for the supplemental investigation. The corrosion results are presented at the end of Appendix C and were performed by CERCO Analytical of Concord, California using ASTM methods.


3. **SITE CONDITIONS**

3.1 **Site Description**

As previously discussed, the Discovery Bay WWTP is divided into Plant No. 1 and Plant No. 2 with Highway 4 dividing the two plants. Both plants are located in the southeastern end of the Town of Discovery Bay. The main office and administration facilities for the WWTP are located at Plant No. 2, which is located at 17501 Highway 4, just south of and adjacent to State Route 4.

The area for the planned improvements at Plant No. 2 is relatively flat with elevations ranging from approximately 89 to 94 feet\(^5\) according to the 2021 grading plans previously referenced. The project area is currently occupied by asphalt concrete (AC) pavement, gravel, grass, and weeds. The area for the planned Oxidation Ditch No. 4 is currently occupied by an elongated mound about 3 to 5 feet high and consisting of temporary fill placed during the original grading for Oxidation Ditch No. 3 to pre-load the area and induce long-term settlement.

We performed a site walk throughout the paved areas of Plant No. 2 on September 19, 2019 with a member of HERWIT Engineering to observe the existing pavement condition. During our site walk, we noticed that driveways exposed to heavy truck traffic were in poor to failed condition. Pavement distress in these areas consisted primarily of medium to high severity alligator cracking, rutting, potholes, and depressions. The most significant pavement distress was observed along the L-shaped entrance driveway that extends from Highway 4 into the southern section of the WWTP, especially where there is a 90-degree turn in the driveway. Areas exposed to light vehicular traffic, such as parking areas, were in overall good condition and are experiencing moderate block cracking. Figures 3 and 4, Site Photographs, depict examples of the pavement distress observed during our site walk.

3.2 **Site Geology**

Geology mapping of the Site and surrounding has been published by the U.S. Geological Survey (USGS, 2000\(^6\)) and the California Geological Survey (CGS, 2018\(^7\)). Both the USGS and CGS have mapped the Site as Holocene alluvial fan deposits, fine facies (fine-grained alluvial fan and flood plain overbank deposits laid down in very gently sloping portions of the alluvial fan or valley floor). These deposits are dominated by clay and silt, with interbedded lobes of coarser alluvium (sand and occasional gravel) that act as potential conduits for groundwater flow. This mapping is corroborated well with our current and the previous investigations discussed above. The Site lies within the Woodward Island 7.5-minute quadrangle.

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\(^5\) Elevations presented throughout this report are based on a plant datum of 100 feet that is equivalent to an actual elevation of 0 feet.


\(^7\) California Geological Survey Staff (2018), Seismic Hazard Zone Report for the Woodward Island 7.5-Minute Quadrangle, Contra Costa County, California, Seismic Hazard Zone Report 121.
and are mapped inside a liquefaction hazard zone by the CGS. The USGS has designated a moderate liquefaction susceptibility for this unit based on shallow groundwater and the presence of lenses of fine sand and silt. The Site does not lie within a mapped Alquist-Priolo Earthquake Fault Zone (AP Zone). The potential for geologic hazards associated with the Site was evaluated and is discussed in the “Geologic and Seismic Hazards” section of this report.

### 3.3 Subsurface Conditions

Below is a general description of soil conditions encountered at the Site (i.e., Plant No. 2). For a more detailed description of the soils encountered, refer to the boring logs and test pit logs in Appendix A and the CPT log in Appendix B. It should be noted that subsurface conditions can deviate from those conditions encountered in the current and previous investigations. If significant variation in the subsurface conditions is encountered during construction, it may be necessary for BSK to review the recommendations presented herein and recommend adjustments, as necessary.

According to previous borings and test pits performed within Plant No. 2, organic clay and peat layers of varying thicknesses were encountered in the upper approximately 5 feet BGS at various locations throughout this section of the plant. Therefore, we performed organic content testing of the surficial soils encountered during our 2019 and supplemental investigations as described below. According to NAVFAC 7.01\(^8\), soils having an organic content by weight less than 5 percent are slightly organic, while soils having an organic content between 5 and 30 percent are organic soils. Soils having an organic content of over 30 percent are considered as highly organic and classified as peat.

#### 3.3.1 Supplemental Borings and Test Pits

According to our supplemental borings and test pits (see Appendix A) performed for the planned Oxidation Ditch No. 4 and the pump station and denitrification basin immediately to the north, the area is generally underlain by approximately 1 to 6 feet of fill consisting predominantly of firm to hard sandy lean clay, but also including loose clayey sand. The fill is underlain by a layer of peat and high organic content soils about 1 to 5 feet thick. According to the organic content testing we performed on surficial samples obtained from test pits TP-20-1, TP-20-3, and TP-20-4, and boring B-20-2 at depths of about 1½ to 6½ feet BGS, these soils have between approximately 15 and 42 percent organic content.

Below the layer of peat and high organic content soils, our supplemental borings and test pits encountered interbedded layers of soft to hard lean clay, sandy lean clay, elastic silt, and fat clay to the maximum depth of our exploration in this area (about 31½ BGS). Atterberg limits testing performed on samples obtained in the upper 10 feet from boring B-20-1 and B-20-2 and test pits TP-20-1 and TP-20-3 resulted in liquid limits (LL) ranging from 55 to 100 and plasticity index (PI) values ranging from 16 to 41. These values are indication of soils with moderate to very high soil expansion potential.

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\(^8\) Naval Facilities Engineering Command (NAVFAC), Design Manual 7.01, Revalidated by Change 1 September 1986.
3.3.2 2019 Boring and CPT

According to our 2019 boring (see boring B-2 in Appendix A), the proposed locations of the two western denitrification basins and pump stations are generally underlain by about 5 feet of firm to hard fat clay with sand. The pavement section encountered at boring B-2 consisted of approximately 2½ inches of asphalt concrete (AC) over 3 inches of aggregate base (AB). Atterberg limits testing performed on a sample collected at approximately 3 feet BGS at boring B-2 resulted in an LL of 67 and PI of 39. These values are indicative of soils with a high to very high soil expansion potential.

Below the surficial fat clay with sand layer, boring B-2 encountered mainly firm lean clay with varying amounts of sand interbedded with layers of loose silty sand and clayey sand to the maximum depth of our boring (approximately 31½ feet BGS). The soil layers encountered in our boring are consistent with the findings of our 2019 CPT (see CPT-2 in Appendix B).

No readily visible evidence of a peat layer was encountered in the continuous samples collected in the upper approximately 8 feet in boring B-2 for our 2019 investigation. According to the organic content testing we performed on surficial samples collected at depths of approximately 3½ to 5½ feet BGS at boring B-2, these soils have between approximately 3 and 7 percent organic content, which are indicative of slightly organic soils.

3.3.3 Previous Subsurface Data

As shown on Figure 2, various previous exploration points consisting of borings and test pits were performed by Kleinfelder between 1985 and 2013 proximate to the planned improvements at Plant No. 2: test pit TP-30 (1985), test pits TP-3 through TP-5 (2003), borings B-1 through B-2 (2012), and test pits TP-1 through TP-6 (2013). These exploration points extended to depths of approximately 4 to 51½ feet BGS. The logs for these exploration points are presented in Appendix E. Soil identified by Kleinfelder as organic clay and peat was encountered in the upper 2 to 5 feet BGS. Below the organic soil layer, predominantly firm lean clay was encountered to the maximum depth explored.

3.3.4 Groundwater

According to the California Geological Survey (2018), historically high groundwater depth at the Site is expected to be shallower than 10 feet BGS. Free groundwater was first observed at a depth of approximately 15 to 20 feet BGS in our supplemental and previous borings at Plant No. 2. At the time of the completion of the borings for our 2019 and supplemental investigations, free groundwater had risen as high 7½ feet BGS in boring B-2 (approximately elevation 83½ feet\(^9\)), 5 feet BGS in boring B-20-1 (approximately elevation 84 feet), and 10 feet BGS in boring B-20-2 (approximately elevation 83 feet). Initial free groundwater was observed in our supplemental test pits at depths of about 10½ to 14 feet BGS.

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\(^9\) Elevations presented throughout this report are based on a plant datum of 100 feet that is equivalent to an actual elevation of 0 feet.
Free groundwater was observed in borings B-2 (2012) and B-3 (2012) at depths of about 4½ feet BGS (approximately elevation 84½ feet) and about 5½ feet BGS (approximately elevation 84½ feet), respectively. According to the 2006 groundwater monitoring report prepared by Kleinfelder for Plant No. 2, groundwater elevations ranging from approximately 81½ to 85½ feet were measured in three wells (labeled MW-1, MW-2, and MW-3) between April 2005 and January 2006 (refer to Appendix E for the site plan showing the location of these wells and groundwater level measurements). It should be noted that groundwater levels can fluctuate several feet depending on factors such as seasonal rainfall, groundwater withdrawal, and construction activities on this or adjacent properties. Also, the initial and (when applicable) final free groundwater levels shown in the current and previous exploration points may not be representative of stabilized groundwater levels.
4. DISCUSSIONS AND CONCLUSIONS

Based on the results of our investigation, it is our opinion that the proposed improvements are feasible geotechnically. This conclusion is based on the assumption that the recommendations presented in this updated report will be incorporated into the design and construction of this project.

Additional discussions of the conclusions drawn from our 2019 and supplemental investigations, including general recommendations, are presented below. Specific recommendations regarding geotechnical design and construction aspects for the project are presented in the “Recommendations” section of this report.

4.1 Foundations

Based on the findings from our geotechnical investigation, the denitrification basins, the pump stations, Oxidation Ditch No. 4, the “gate box”, and rotors may be supported on mat foundations, while the at grade ancillary structures may be supported on shallow foundations consisting of mat foundations or spread footings. The primary geotechnical considerations for the proposed improvements are as follows:

1. The potential for strong ground shaking to affect the Site during a future significant seismic event (typical of the entire San Francisco Bay Area),

2. The presence of shallow groundwater that will impose hydrostatic pressures against the basins’ walls and buoyancy pressures that will cause uplift loads,

3. The need to temporarily dewater and shore the excavations for the basins and associated underground utility lines during construction, and

4. The presence of surficial soils with high organic content or soils containing peat within the planned excavation for the new oxidation ditch No. 4 and underneath new at grade ancillary structures.

New foundations should not be located above a 1H:1V (horizontal to vertical) imaginary plane projected upward from the bottom of the perimeter foundations for nearby structures or underground utility lines to avoid surcharging the adjacent structure and underground utilities with structure loads. Otherwise, the affected improvements should be evaluated to check if they can withstand surcharge loads imposed by the new foundations. Sometimes, it is also possible to use controlled low strength material (CLSM) as trench backfill for the portion of trenches extending below a 1H:1V line extending from the edge of nearby foundations to help protect new utility lines from surcharge loads. However, this alternative should be evaluated by the design team on a case-by-case basis prior to implementation during construction.

Specific recommendations regarding these considerations are provided in the “Recommendations” section of this report.
4.2 Shallow Groundwater

As discussed in the “Subsurface Conditions” of this report, free groundwater was observed at depths ranging from about 4½ to 14 feet BGS within current and previous borings and test pits performed at the Site. However, the actual depth at which groundwater may be encountered in trenches and excavations may vary. Therefore, excavations deeper than about 5 feet BGS will likely require dewatering (possibly continuous) during construction and the design of the proposed below-grade structures will need to consider buoyancy forces. Table 1 below presents estimated groundwater elevations at the Site.

<table>
<thead>
<tr>
<th>Location</th>
<th>Approximate Overall Existing Ground Surface Elevation (feet)¹</th>
<th>Estimated Groundwater Elevation for Design (feet)¹, ²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Western Plant No. 2 denitrification basin and pump station (located north of Oxidation Ditch No. 2)</td>
<td>91</td>
<td>83½</td>
</tr>
<tr>
<td>Eastern Plant No. 2 denitrification basin and pump station located north of Oxidation Ditch No. 4</td>
<td>89</td>
<td>84</td>
</tr>
<tr>
<td>Oxidation Ditch No. 4</td>
<td>93</td>
<td>83</td>
</tr>
</tbody>
</table>

Notes:
1. Ground surface elevations are based on plant datum and Sheets C-203 and C-204 of the March 2021 grading plans prepared by HERWIT Engineering for Plant No. 2.
2. Groundwater levels can fluctuate several feet depending on factors such as seasonal rainfall, groundwater withdrawal, and construction activities on this or adjacent properties. Also, the initial and (when applicable) final free groundwater levels shown in the current and previous exploration points may not be representative of stabilized groundwater levels.

4.3 Expansive Soils

The near-surface soils at the Site have a high to very high expansion potential when subjected to moisture content fluctuation. The presence of expansive soils can be addressed by using crushed drain rock and Class 2 aggregate base under slabs and proper moisture conditioning. These considerations are incorporated in the recommendations presented later in this report.

4.4 Anticipated Settlements

The subsections below present our estimated elastic and liquefaction-induced settlements for the project. For design purposes, these settlements should be assumed to be cumulative.
4.4.1 Elastic Settlement

Total elastic settlement is anticipated to be less than ½-inch for the denitrification basins, pump stations, Oxidation Ditch No.4, “gate box”, and rotors if they are supported on mat foundations. Differential elastic settlement is expected to be about half of the total elastic settlement over a horizontal distance of about 30 feet.

At grade ancillary structures for this project will be supported on shallow spread footings and mat foundations. Such structures could be located well away from the planned denitrification basins in areas containing surficial soils with high organic content or containing peat, which could undergo significant elastic settlement. Therefore, we recommend that potholing the upper approximately 5 feet BGS at these locations be performed at each of these structures prior to construction of their foundations. Where peat or soils containing more than 3 percent organic content by dry unit weight are encountered, the entire area within 5 feet laterally of the foundation perimeter, where feasible, should be overexcavated to a depth of 5 feet BGS and then be backfilled with properly compacted aggregate base or on-site soil containing less than 3 percent organic content by dry unit weight. Where this recommendation is followed, total settlement is anticipated to be less than ½-inch, while differential settlement is anticipated to be less than ¼-inch over a horizontal distance of about 30 feet.

Elastic settlement for poles supported on drilled piers is expected to be negligible.

4.4.2 Consolidation Settlement

Because the proposed below-grade structures (i.e., denitrification basins, pump stations, Oxidation Ditch No.4, “gate box”, and rotors) will be supported at a depth of approximately 8 to 20 feet BGS (and below the potential surficial peat layers that are present at the Site), we expect the bearing pressures imposed by these structures will not result in a significant increase to the current overburden pressures experienced by the supporting soils below these structures, especially when considering that buoyancy pressures will help counteract the loads imposed by these structures. Therefore, we conclude that the potential for the below-grade structures to induce consolidation settlement at the Site is negligible.

Provided peat or soils containing more than 3 percent organic content by dry unit weight are removed to at least 5 feet below at grade ancillary structures and replaced with properly compacted aggregate base or on-site soil containing less than 3 percent organic content by dry unit weight, we anticipated that total consolidation settlement would be less than ½-inch, while differential consolidation settlement is anticipated to be less than ¼-inch over a horizontal distance of about 30 feet.

Consolidation settlement for poles supported on drilled piers is expected to be less than ½-inch.
4.4.3 Liquefaction-Induced Settlement

Soil liquefaction is a condition where saturated, granular soils undergo a substantial loss of strength and deformation due to pore pressure increase resulting from cyclic stress application induced by earthquakes. In the process, the soil acquires mobility sufficient to permit both horizontal and vertical movements if the soil mass is not confined. Soils most susceptible to liquefaction are saturated, loose, clean, uniformly graded, and fine-grained sand deposits and some low plasticity clays. If liquefaction occurs, foundations resting above or within the liquefiable layer may undergo settlements and/or a loss of bearing capacity.

We ran liquefaction analyses for CPT-2 using the methods by Boulanger and Idriss (2014)\textsuperscript{10}. For our analyses, we assumed a design groundwater depth of 5 feet BGS, a peak ground acceleration of 0.541g based on the 2019 California Building Code (CBC) seismic parameters (presented in the “2019 CBC Seismic Design Parameters” section of this report), and an earthquake magnitude of M6.5 obtained via the USGS deaggregation website (http://geohazards.usgs.gov/deaggint/2008/).

Our liquefaction analyses estimate that a design level earthquake is capable of inducing liquefaction settlement of up to approximately $\frac{1}{2}$-inch below a depth of 18 feet BGS.

Based on Youd and Garris (1995)\textsuperscript{11}, we consider the overall potential for ground surface disruption (such as sand boils, ground fissures, etc.) to occur at the Site to be low due to relative thickness of the non-liquefiable layers overlying the liquefiable layers.

Differential liquefaction-induced settlement can be considered to be about two-thirds of the total liquefaction-induced settlement over a horizontal distance of about 30 feet.

4.4.4 Dynamic Compaction/Seismic Settlement

Another type of seismically induced ground failure, which can occur as a result of seismic shaking, is dynamic compaction, or seismic settlement. Such phenomena typically occur in unsaturated, loose granular material or uncompacted fill soils. Due to the composition, consistency, and apparent relative density of the soils above the water table within our borings and CPT, we conclude that the potential for dynamic compaction/seismic settlement to affect the Site during a seismic event is low.

\textsuperscript{10} Boulanger, R. W., and Idriss, I. M. (2014), CPT and SPT Based Liquefaction Triggering Procedures, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, College of Engineering, University of California at Davis, California Report No. UCD/CGM-14/01, April 2014.

4.5 Geologic and Seismic Hazards

4.5.1 Faulting and Seismic Shaking

The San Francisco Bay Area is seismically dominated by the active San Andreas Fault system. This fault system movement is distributed across a complex system of generally strike-slip, right-lateral parallel and sub-parallel faults including the Greenville Fault located approximately 12 miles southwest of the Site, Concord Fault located approximately 22 miles west of the Site, the Calaveras Fault located approximately 22 miles southwest of the Site, the Hayward Fault located approximately 30 miles southwest of the Site, and the San Andreas Fault located approximately 48 miles southwest of the Site.

The Site is not located within an Alquist-Priolo Earthquake Fault Zone and no mapped active fault traces are known to transverse the Site. Nevertheless, the Site is in a seismically active area of California. We expect the Site to be subjected to substantial ground shaking due to a major seismic event on the active faults in the Bay Area and surrounding regions during the design life of the project.

In 2015, scientists and engineers released a new earthquake forecast for the State of California\textsuperscript{12}. It updates the earthquake forecast made for the greater San Francisco Bay Area by the 2007 Working Group for California Earthquake Probabilities. According to this recent study, there is a 72 percent probability that one or more magnitude M6.7 or greater earthquakes will occur in the San Francisco Bay Area in the next 30 years (2014 to 2044).

As has been demonstrated recently by the 1989 (M6.9) Loma Prieta, the 1994 (M6.7) Northridge, and the 1995 (M6.9) Kobe earthquakes, earthquakes of this magnitude range can cause severe ground shaking and significant damage to modern urban environments. Therefore, the design of the planned structures should incorporate the seismic design parameters presented in the “2019 CBC Seismic Design Parameters” section of this report.

4.5.2 Liquefaction Potential

The Site’s liquefaction potential is discussed in the preceding “Liquefaction-Induced Settlement” section of this report.

4.5.3 Dynamic Compaction/Seismic Settlement Potential

The Site’s dynamic compaction settlement is discussed in the preceding “Dynamic Compaction/ Seismic Settlement” section of this report.

4.5.4 **Lateral Spread Potential**

An irrigation channel is located approximately 400 feet west of the denitrification basins at Plant No. 2. According to the elevations provided by Google Earth Pro, the bottom of the irrigation channel appears to lie at a depth of approximately 5 feet BGS. Because the depth of the potentially liquefiable layers encountered at CPT-2 are located below 18 feet BGS, we conclude that the potential for lateral spread to adversely affect the project is low.

4.5.5 **Flood Hazard**

According to the 2009 Federal Emergency Management Agency (FEMA) flood insurance rate map\(^{13}\), the Site is located in “Areas of 0.2% chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.” Note that 0.2% and 1% chance floods have recurrence intervals of 500 and 100 years, respectively.

4.6 **Existing Pavement**

According to our discussion with HERWIT Engineering, the original pavement section for the southern section of the WWTP was placed in 2002 and consisted on average of approximately 3 inches of AC over 8 inches of AB. As noted in the “Site Description” section of this report, driveways exposed to heavy truck traffic in this section of the WWTP are experiencing significant pavement distress consisting primarily of medium to high severity alligator cracking, rutting, potholes, and depressions. We understand that the near-surface peat layer encountered during previous investigations throughout this section of the WWTP was not removed under pavement areas. Therefore, the likely causes of the poor pavement conditions are a combination of heavy traffic loading and poor subgrade conditions. HERWIT Engineering mentioned that since construction of Plant No. 2 in the early 2000’s, several pavement segments exposed to heavy truck traffic have been reconstructed multiple times using a layer of Mirafi\textsuperscript{®} 600X geotextile fabric as reinforcement. However, these reconstructed sections eventually failed in areas exposed to heavy traffic loading.

Based on this information and our experience, we considered recommending that full depth reclamation (FDR) be used to rehabilitate the driveways experiencing significant pavement distress. However, after discussions with HERWIT Engineering, we concluded that the anticipated square footage of distressed pavement exposed to heavy traffic loading is not a large enough area to make FDR a feasible rehabilitation alternative for this project. Therefore, we are recommending full-depth pavement reconstruction as a long-term repair measure for such areas. In addition, we recommend that Portland Cement Concrete (PCC) pavement be used for the 90-degree turn of the L-shaped entrance driveway to the southern section.

\(^{13}\) Federal Emergency Management Agency (FEMA 2009), FEMA Flood Insurance Rate Map, Contra Costa County, California and Incorporated Areas, Panel 390 of 602, Map Number 06013C0390F, June 16, 2009.
of the WWTP. Areas exposed to light vehicular traffic, such as parking areas, can be rehabilitated via application of crack sealing followed by application of a seal coat.

Because this project is expected to be constructed over a period of two years, areas experiencing significant failures could be temporarily repaired in the interim via localized excavation, and scarification and recompaction of the excavation bottom and then repaved until the long-term rehabilitation measures recommended above can be implemented. Additional discussion of these and other pavement rehabilitation measures are provided in the “Recommendations” section of this report.

### 4.7 Re-Use of On-Site Soil

From a geotechnical standpoint only, the on-site (native) soils are suitable for re-use as general engineered fill\(^\text{14}\) and trench/excavation backfill during grading of the project provided vegetation, soil containing more than 3 percent organic content by dry unit weight, peat, and deleterious matter are removed. On-site soils are not considered suitable for use as trench fine grading and pipe bedding (refer to the “Trench Fine Grading and Pipe Bedding” section of this report). Stripping requirements for these areas are discussed in the “Earthwork” section of this report. A BSK representative should be present on-site during grading to visually confirm the suitability of the soil to be used as fill and backfill. Further discussion on the re-use of on-site soil as fill is presented in the “Earthwork” section of this report.

### 4.8 Excavations and Shoring

Care should be taken during construction to reduce the impact of trenching and excavations on adjacent structures and pavements. Excavations should be located so that no structures, foundations, and slabs, existing or new, are located above an imaginary plane projected 1H:1V upward from any point in an excavation unless the excavation is properly shored and excavated in stages. Furthermore, excavations should be set back a minimum of 10 feet horizontally measured from the outer edge of the perimeter foundation of existing structures to the excavation slope face. If structures are located within this 1H:1V project line and/or this minimum setback is not feasible, the shoring should be designed to handle the applicable vertical and/or horizontal surcharge loading from the adjacent structure and allow no horizontal movement of the excavation. Further discussion on excavations and shoring is presented in the “Excavation, Shoring, and Backfill” section of this report.

### 4.9 Overexcavation and Backfill of Test Pits

The test pits performed for our supplemental investigation were loosely backfilled using the backhoe’s bucket. Therefore, where new improvements are constructed over or within 5 feet laterally of the test

\(^{14}\) Fill that is properly compacted and moisture conditioned per the requirements of the “Earthwork” section of this report and that may consist of on-site soils instead of imported soil, such as aggregate base.
pits, we recommend that the location of the test pits be overexcavated and backfilled with compacted engineered fill as discussed in the “Earthwork” section of this report.
5. **RECOMMENDATIONS**

Presented below are recommendations for foundations, lateral earth pressures and passive resistance, seismic considerations, earthwork, pavement rehabilitation, and construction considerations for this project.

5.1 **Foundation Recommendations**

5.1.1 **Below-Grade Structures**

The loads from the proposed below-grade structures (denitrification basins, pump stations, Oxidation Ditch No. 4, the “gate box”, and rotors) may be supported on mat foundations. The mats may be designed for an allowable bearing pressure of 1,500 pounds per square foot (psf). This value is based on the assumption that the mats uniformly bear on re-compacted native soil or compacted engineered fill\(^{15}\). The allowable bearing pressure value may be increased by 1/3 for short term seismic and wind loads. The bearing capacity value includes a factor of safety of at least 3.

Provided temporary dewatering lowers the groundwater table within the excavation for the proposed below-grade structures to a minimum of 2 feet below the excavation bottom, the subgrade below the limits of the mat foundation should be compacted per the requirements of Appendix D. We recommend that the mat be underlain by at least 12 inches of compacted 1-inch crushed drain rock meeting the gradation criteria for coarse aggregate No. 57 per ASTM C33, latest edition with no more than 10 percent by weight passing the No. 4 sieve. The crushed drain rock layer should be wheel-rolled to help its particles properly consolidate and interlock. The new mat foundations should extend below a 1H:1V plane projected upward from the adjacent existing or new foundations, other below-grade structures, and bottom of the underground utility to avoid surcharging these improvements with building loads. Otherwise, the affected improvements should be evaluated to check if they can withstand surcharge loads imposed by the new foundations.

Resistance to lateral loads can be provided by a combination of friction between the foundation bottoms and the underlying crushed drain rock layer and by passive resistance acting against the vertical faces of the mats. An allowable friction coefficient of 0.30 between the foundation and supporting crushed drain rock layer may be used. For passive resistance, an allowable equivalent fluid pressures of 350 and 200 pounds per cubic foot (pcf) may be used against the sides of mats and the below-grade walls above and below, respectively, the design groundwater elevation. The friction coefficient and passive pressure values include factors of safety of about 1½. The passive pressure may be increased by one-third for seismic and wind loads. **Passive resistance in the upper foot of soil cover below finished grades should**

\(^{15}\) Fill that is properly compacted and moisture conditioned per the requirements of the “Earthwork” section of this report.
be neglected unless the ground surface is confined by concrete slabs, pavements, or other such positive protection.

5.1.2 At Grade Ancillary Structures

5.1.2.1 Mat Foundations

The loads from the proposed at grade ancillary structures may be supported on mat foundations. The mats may be designed for an allowable bearing pressure of 1,500 psf and should have a minimum depth at the edges of 18 inches. This value is based on the assumption that the mats uniformly bear on re-compacted native soil or compacted engineered fill. The allowable bearing pressure value may be increased by 1/3 for short term seismic and wind loads. The bearing capacity value includes a factor of safety of at least 3.

The mat foundations should be supported on a minimum of 12 inches of compacted aggregate base to provide enhanced slab support and to help mitigate seasonal movement of the expansive surficial soils. If moisture vapor through the slab is objectionable, the use of a vapor barrier at least 15 mils thick and capillary moisture break consisting of a minimum 6-inch thick layer of compacted 1-inch crushed drain rock meeting the gradation criteria for coarse aggregate No. 57 per ASTM C33, latest edition with no more than 10 percent by weight passing the No. 4 sieve should be considered by the designer. The new mat foundations should extend below a 1H:1V plane projected upward from the adjacent existing or new foundations, below-grade structures, and bottom of the underground utility to avoid surcharging these improvements with building loads. Otherwise, the affected improvements should be evaluated to check if they can withstand surcharge loads imposed by the new foundations.

Resistance to lateral loads for mat foundations can be provided by a combination of friction between the foundation bottoms and the underlying aggregate base or crushed drain rock layer and by passive resistance acting against the vertical faces of the mats. An allowable friction coefficient of 0.30 between the foundation and supporting aggregate base or crushed drain rock layer may be used. For passive resistance, an allowable equivalent fluid pressure of 350 and 200 pcf up to a maximum of 2,000 psf may be used against the sides of mats above and below, respectively, the design groundwater elevation. The friction coefficient and passive pressure values include factors of safety of about 1½. The passive pressure may be increased by one-third for seismic and wind loads.

Passive resistance in the upper foot of soil cover below finished grades should be neglected unless the ground surface is confined by concrete slabs, pavements, or other such positive protection. For foundations located on or proximate to sloping ground, the passive resistance should be neglected in the upper portion of the foundation until there is a horizontal distance of at least 10 feet between the slope face and the nearest edge of the foundation.
5.1.2.2 Spread Footings

The loads from the proposed at grade ancillary structures may also be supported on spread footings. Footings should be continuous around the perimeter of structures supported on spread footings to reduce the potential for moisture content fluctuations within the potentially expansive soils underlying the Site. The recommended allowable soil bearing pressure, depth of embedment, and width of footings are presented in Table 2 below.

<table>
<thead>
<tr>
<th>Footing Type</th>
<th>Allowable Bearing Pressure (psf)*</th>
<th>Minimum Embedment (in)**</th>
<th>Minimum Width (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exterior Continuous Footings</td>
<td>2,500</td>
<td>24</td>
<td>12</td>
</tr>
<tr>
<td>Isolated Interior Footings</td>
<td>2,500</td>
<td>24</td>
<td>18x18</td>
</tr>
</tbody>
</table>

* Pounds per square foot, dead plus live load. Includes factor of safety (FS) of about 3. The allowable bearing pressure may be increased by 1/3 for short-term seismic and wind loads.

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

It should be noted that the 24-inch embedment recommendation for footings reflects the expansive nature and susceptibility of the near surface soils to shrink/swell cycles associated with fluctuations in moisture content. The actual depth of embedment for bearing capacity requirements is 18 inches below the lowest adjacent grade. In lieu of extending the reinforcing steel and structural concrete the entire depth (if not needed for structural design), the portion of the footings in the excavation below 18 inches can be backfilled with CLSM meeting the project specifications that should be placed shortly following excavation. By placing CLSM shortly after excavation, the soil at the bottom of the excavation will not have time to dry out, thus avoiding higher expansion pressures upon wetting. The use of CLSM also eliminates disturbance of the soil exposed at the bottom of the excavation to weather and construction activities following excavation.

The new footings should extend below a 1H:1V plane projected upward from the adjacent existing or new foundations, below-grade structures, and bottom of the underground utility to avoid surcharging these improvements with building loads. Otherwise, the affected improvements should be evaluated to check if they can withstand surcharge loads imposed by the new foundations. Also, where utilities cross under the perimeter footing line and enter “interior” space, the trench backfill should consist of CLSM for a distance of 2 feet laterally on each side of the perimeter footing centerline to reduce the potential for the trench to act as a conduit to exterior surface water. In addition, where utilities cross through footings, flexible waterproof caulking should be provided between the sleeve and the pipe. Utility plans should be reviewed by BSK prior to trenching for conformance to these requirements.
Concrete for footings should be placed neat against undisturbed on-site soils or engineered fill. It is critical that footing excavations not be allowed to dry before placing concrete. If soft or loose soils are encountered at the bottom of the footing excavations, they should be overexcavated and replaced with lean concrete or engineered fill based on input from a BSK representative. The footing excavations should be monitored by a representative of BSK for compliance with appropriate moisture control, to confirm the adequacy of the bearing materials, and whether overexcavation of the footing bottoms is needed due to the presence of soft/loose soils.

Lateral loads for spread footings may be resisted by a combination of friction between the bottom of the footings and the supporting subgrade and by passive resistance acting against the footings. The frictional and passive resistance may be assumed in design to act concurrently and the passive resistance can be increased by one-third for wind and/or seismic loading. An allowable friction coefficient of 0.30 between the bottom of the foundations and supporting subgrade soils may be used. For passive resistance, an allowable equivalent fluid pressure (unit weight) of 350 and 200 pcf may be used above and below, respectively, the design groundwater elevation. The friction and passive values include factors of safety of about 1½.

Passive resistance in the upper foot of soil cover below finished grades should be neglected unless the ground surface is confined by concrete slabs, pavements, or other such positive protection. For foundations located on or proximate to sloping ground, the passive resistance should be neglected in the upper portion of the foundation until there is a horizontal distance of at least 10 feet between the slope face and the nearest edge of the foundation.

5.1.3 Pole Foundations

New poles may be supported on drilled piers and should derive their vertical load capacities through skin friction on the side of the piers. For resistance to uplift loads, the weight of the piers and the skin friction between the piers and supporting soils may be used. An allowable skin friction value of 400 psf may be used to resist downward loads. The allowable skin friction may be increased by 1/3 for short term seismic and wind loads. The dead plus live load friction resistance includes a safety factor of at least 2. Uplift loads for long-term conditions should not exceed 2/3 of the allowable downward skin friction. The piers should have a minimum depth of 5 feet below lowest adjacent grade, have a minimum diameter of 18 inches, and be spaced at least 3 diameters apart (center to center) or skin friction reductions may be necessary.

Resistance to lateral loads for drilled piers can be provided by passive resistance against the face of the piers using an allowable equivalent fluid pressure of 350 and 200 pcf up to a maximum of 2,000 psf acting against the piers above and below, respectively, the design groundwater elevation. These passive pressures may be increased to 500 and 300 psf up to a maximum of 3,000 psf acting against the piers above and below, respectively, the design groundwater elevation for poles that can accommodate up to ½-inch of lateral movement. The passive resistance may be applied to a width of twice the diameter of the piers. Piers should be spaced at least 6 diameters apart (center to center) in the direction of loading or lateral resistance capacity reductions may be necessary. The passive pressure value includes a factor
of safety of about 1½ to 2. The passive pressure may be increased by one-third for seismic and wind loads. Passive resistance in the upper foot of soil cover below finished grades should be neglected unless the ground surface is confined by concrete slabs, pavements, or other such positive protection. For foundations located on or proximate to sloping ground, the passive resistance should be neglected in the upper portion of the foundation until there is a horizontal distance of at least 10 feet between the slope face and the nearest edge of the foundation.

If the pier holes extend below the groundwater table or into sandy layers, the pier holes may be susceptible to caving and sloughing, which may necessitate the use of temporarily casing the pier holes during installation or drilling the piers using the slurry displacement method. If temporary casing is used during construction, it should consist of smooth-walled steel casing. Corrugated metal pipe (CMP) should not be permitted as casing because it results in excessive voids and/or disturbance of the surrounding soil during removal of the casing.

The bottom of the pier holes should be cleaned and/or tamped such that no more than 2 inches of loose soil remains in the holes prior to the placement of concrete. If more than 6 inches of standing water is present at the bottom of the pier holes during concrete placement, either the water needs to be pumped out or the concrete needs to be placed into the hole using tremie methods.

We recommend that steel reinforcement and concrete be placed within about 4 to 6 hours upon completion of each pier hole. As a minimum, the holes should be poured the same day they are drilled. The steel reinforcement should be centered in the pier holes. In order to develop the allowable skin friction value provided above, concrete used for pier construction should have a slump of 6 to 8 inches. A concrete mix with a low water/cement ratio should be used in the construction of the piers to reduce shrinkage of the concrete. To increase the fluidity of the mix for improved consolidation and bond with the reinforcing steel, increased slump may be desirable. If this is the case, the slump should be increased via use of a plasticizer, rather than by adding water to the mix, because a low water to cement ratio is desired for shrinkage control. Concrete used for pier construction should be discharged vertically into the pier holes to reduce aggregate segregation. Under no circumstances should concrete be allowed to free-fall against either the steel reinforcement or the sides of the pier hole during construction.

5.1.4 Modulus of Subgrade Reaction

Due to the presence of shallow groundwater at the Site, a modulus of subgrade reaction, \( K_{V1} \), of 60 pounds per square inch per inch (pci) of deflection (based on published data for a one square foot bearing plate) is considered applicable for mat foundations for below-grade structures. New at-grade slabs (i.e., at the ground surface) may be designed using a \( K_{V1} \) value of 120 pci. These modulus values are typically reduced for foundation or lab sizes larger than 1 square foot. For various foundation and slab sizes, the subgrade modulus may be calculated using the following formula:
\[ K_R = (K_{V1}) \times \left(\frac{m+0.5}{1.5 \times m}\right) \]

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

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

### 5.1.5 Construction Monitoring

A BSK representative should observe the excavations for mat foundations, spread footings, and drilled piers to confirm that subsurface conditions are similar to those encountered in our borings, test pits, and CPT and to check if the contractor is properly dewatering the excavation for the proposed below-grade structures and is properly casing or using slurry to drill the pier holes if caving/sloughing conditions are present.

### 5.2 Proximity to Below-Grade Structures and Underground Utilities

Where new foundations are located adjacent to below-grade structures or near underground utilities, the new foundations should be located such that they extend below a 1H:1V imaginary plane projected upward from the bottom of the adjacent below-grade structure's foundation perimeter or bottom of the underground utility to avoid surcharging the below-grade structure and underground utility with structure loads. Where this is not feasible, the affected structure/pipeline should be evaluated to check that it can withstand the surcharge loads imposed by the new foundations. Sometimes, it is also possible to use CLSM as trench backfill for the portion of trenches extending below a 1H:1V line extending from the edge of nearby foundations to help protect new utility lines from surcharge loads. However, this alternative should be evaluated by the design team on a case-by-case basis prior to implementation during construction.

### 5.3 Uplift Loading Due to Buoyancy

Below-grade structures should be designed to resist a buoyancy force based on the recommended design groundwater elevations presented in Table 1 of this report. The weight of the structures (assume empty case) may be used to resist this uplift pressure as well as friction between the below-grade walls and the surrounding backfill. An allowable friction coefficient of 0.30 between the walls and surrounding backfill may be used. This value includes a factor of safety of about 1½. Normal pressures of 60D psf and 30D psf above and below the design groundwater elevation, respectively, where \( D \) is the depth in feet of the below-
grade walls below the ground surface, may be used to compute the normal force to be used with the allowable friction coefficient.

If the mat foundations for the basins extend beyond the outer basin wall limits to form a “lip”, the weight of the backfill above the lip plus a soil wedge extending upward at a 60-degree angle from the horizontal from the edge of the lip may also be used to resist uplift pressure in lieu of the wall friction discussed in the paragraph above. Effective soil unit weights of 120 and 58 pcf may be used above and below the design groundwater depth, respectively.

5.3.1 Hydraulic Relief Valves and Crushed Rock Layer

Due to the steep gradient of its sidewalls, the measures discussed above to counter buoyancy forces may not be feasible for the proposed Oxidation Ditch No. 4. If that is the case, we recommend installing one or more hydraulic relief valves at the bottom of the oxidation ditch. We also recommend that the reinforced shotcrete at the bottom of the oxidation ditch be underlain by a layer a minimum of 12 inches thick of compacted aggregate base or Class 2 Permeable Material\(^\text{16}\) as indicated on the project plans.

In addition to providing support under possible moist/wet conditions, this crushed drain rock layer should allow for better dissipation of hydrostatic pressures through the hydraulic relief valve(s) before they build up to the point of causing uplift of the oxidation ditch bottom.

5.4 Below-Grade Walls

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

The amount of outward displacement at the top of the wall designed for active earth pressures may be up to 0.004H to 0.04H, where H is the height of the wall. Below-grade walls may be designed using the lateral earth pressures provided in Table 3 below. A uniform surcharge pressure of 100 psf is typically applied over the upper 10 feet of below-grade walls to account for surcharge loading imposed by vehicular traffic, such as an H-20 live load.

\(^{16}\) Meeting the requirements of Section 68 of the 2015 Caltrans Standard Specifications.
<table>
<thead>
<tr>
<th>Earth Pressures</th>
<th>Equivalent Fluid Pressures (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Above Water*</td>
</tr>
<tr>
<td>Active (Flexible walls)</td>
<td>45</td>
</tr>
<tr>
<td>At-Rest (Rigid walls)</td>
<td>60</td>
</tr>
<tr>
<td>Seismic (Flexible walls)</td>
<td>20***</td>
</tr>
<tr>
<td>Seismic (Rigid walls)</td>
<td>35***</td>
</tr>
</tbody>
</table>

* See Table 1 for estimated groundwater elevations for design.
** Includes hydrostatic pressure
*** Section 1803.5.12 of the 2019 CBC requires that the design for foundation walls include seismic earth pressures and retaining walls supporting backfill heights greater than 6 feet include seismic earth pressures. These pressures are expressed as equivalent fluid pressures and should be added to the wall design in addition to the static active or at-rest pressures. The seismic earth pressure should be applied as a triangular distribution with the resultant force acting 1/3 times the wall height above the base of the wall. The seismic earth pressures presented herein are based on Agusti and Sitar (2013)\(^{17}\).

5.5 2019 CBC Mapped Seismic Design Parameters

The seismicity of the region surrounding the Site is discussed in the “Faulting and Seismic Shaking” section of this report. From that discussion, it is important to note that the Site is in a region of high seismic activity and will likely be subjected to significant shaking during the life of the project. As a result, the planned structures should be designed in accordance with applicable seismic provisions of the 2019 CBC.

Based on our analyses, the Site is susceptible to liquefaction during a design-level earthquake. Therefore, according to Table 20.3-1 of ASCE 7-16, the Site should be classified as Site Class F, which requires site-specific response analysis. However, Sections 11.4.8 and 20.3.1 of ASCE 7-16 state that for a short period (less than ½ second) structure on liquefiable soils, these factors may be based on the assessment of the site class assuming no liquefaction. Therefore, provided the planned improvements consist of structures with fundamental periods of less than about ½ second, we recommend using Site Class D (stiff soil) for design of the planned structures. Assuming this is the case, use of the 2019 CBC mapped seismic design criteria presented in Table 4 below would be considered appropriate for the design of structural improvements for this Site if the exceptions provided in Section 11.4.8 of ASCE 7-16 apply to the planned improvements. Otherwise, we should be consulted to evaluate whether either a site-specific ground motion hazards analysis or a site-specific response analysis is required for this project.

### TABLE 4
#### 2019 CBC MAPPED SEISMIC DESIGN PARAMETERS*

<table>
<thead>
<tr>
<th>Seismic Design Parameter</th>
<th>Value</th>
<th>Location</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site Class</td>
<td>D</td>
<td>D</td>
<td>Table 20.3-1, ASCE 7-16</td>
</tr>
<tr>
<td>MCE(_R) Mapped Spectral Acceleration (g)</td>
<td>S(_S) =1.172 S(_1) =0.419</td>
<td>D</td>
<td>USGS Mapped Values based on Figures 1613.2.1(1) and 1613.2.1(2), 2019 CBC</td>
</tr>
<tr>
<td>Site Coefficients</td>
<td>F(_a) =1.031 F(_v) =1.881</td>
<td>F(_a) =1.030 F(_v) =1.881</td>
<td>Tables 1613.2.3(1) and 1613.2.3(2), 2019 CBC</td>
</tr>
<tr>
<td>MCE(_R) Mapped Spectral Acceleration Adjusted for Site Class Effects (g)</td>
<td>S(<em>{MS}) =1.209 S(</em>{M1}) =0.788</td>
<td>S(<em>{MS}) =1.210 S(</em>{M1}) =0.788</td>
<td>Section 1613.2.3, 2019 CBC</td>
</tr>
<tr>
<td>Design Spectral Acceleration (g)</td>
<td>S(<em>{DS}) =0.806 S(</em>{D1}) =0.525</td>
<td>S(<em>{DS}) =0.807 S(</em>{D1}) =0.525</td>
<td>Section 1613.2.4, 2019 CBC</td>
</tr>
<tr>
<td>Seismic Design Category</td>
<td>D</td>
<td>D</td>
<td>Section 1613.2.5, 2019 CBC</td>
</tr>
<tr>
<td>MCE(_G) peak ground acceleration adjusted for Site Class effects (g)</td>
<td>PGA(_M) = 0.541</td>
<td>PGA(_M) = 0.541</td>
<td>Section 11.8.3, ASCE 7-16</td>
</tr>
</tbody>
</table>

*These seismic design parameters are based on the assumption that the proposed improvements have fundamental periods of less than about ½ second and that a site-specific ground motion hazard analysis is not required based on the exceptions provided in Section 11.4.8 of ASCE 7-16. Otherwise, either a site-specific ground motion hazard analysis or a site-specific response analysis may be required.

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### 5.6 Pipe Design

#### 5.6.1 Vertical Loads on Pipe

The pipe selected should be capable of supporting vertical loads due to the soil overburden (trench backfill) and surcharge, including traffic loads. An in-place density of 130 pounds per cubic foot may be assumed for the trench backfill, and Marston's Formula\(^\text{18}\) may be used. Table 5 below presents the vertical pressure on the pipe due to an H-20 live load as defined in the "American Iron and Steel Institute, Handbook of Steel Drainage and Highway Construction Products".

Additional surcharge loads on the pipe should be considered in the design if the loads are located above the pipe or within a 1H:1V plane projected upwards from the spring line of the pipe.

### 5.6.2 Pipe Foundation and Trench Stabilization

Where pipe subgrade soils are firm and stable, free of debris, loose soil or mud, and free-standing water, no pipe foundation material is needed. However, if loose sands, other soft or loose soils, free standing water, or fill debris are encountered at the proposed subgrade elevations for the pipeline and manholes, or where the subgrade soils are disturbed during construction and proper compaction of the exposed subgrade cannot be achieved by compactive effort, we recommend that the pipe foundation consist of a layer of permeable materials wrapped in filter fabric (Mirafi® 140N or equivalent) placed below the subgrade unless otherwise indicated by BSK. Before placing the rock ballast, the subgrade should be overexcavated a minimum of 9 inches below the trench fine grading section, followed by the placement of filter fabric.

The filter fabric should cover the entire width of the excavation at the locations where rock ballast is used, and its seams should be fixed to the excavation walls before the placement of the rock ballast material. Once these steps are accomplished, the overexcavated subgrade should be filled with rock ballast to achieve a firm and stable surface. The rock ballast material should meet the following criteria:

- Consist of Class 1, Type B Permeable Material meeting the requirements of Section 68 of the 2015 Caltrans Standard Specifications; and
- Consist of crushed stone, or gravel, durable and free from slaking, or decomposition under action of alternate wetting or drying.

Once the rock ballast is placed, the filter fabric should be detached from the walls of the excavation, and wrapped around the rock ballast, and should overlap a minimum of one foot. Likewise, the filter fabric seams for any consecutive sections of rock ballast placed longitudinally along the excavation should also overlap a minimum of one foot.
5.6.3 **Trench Fine Grading and Pipe Bedding**

Trench fine grading should be placed for a thickness of at least 4 inches below the bottom of pipes and should consist of fine-grained silty sand and/or poorly graded sand meeting the project specifications. The same material used as trench fine grading may also be used as pipe bedding (also known as envelope or shading), which is the material typically placed from the top of the trench fine grading to 12 inches above the top of the pipe. The on-site soils are not considered suitable for use as trench fine grading or pipe bedding.

Coarse-grained sand, gravel, and drain rock should be avoided as trench fine grading and/or pipe bedding. Otherwise, the native soil used for backfill above the pipe envelope section could potentially migrate into coarse grained or gap-graded material causing loss of ground and resulting in ground settlement. This could result in pipe joint movement and pavement distress.

5.6.4 **Trench Backfill**

Trench backfill is the material used to backfill the portion of trenches extending from the top of the trenches to the top of the pipe bedding section. From a geotechnical standpoint only, the on-site (native) soils are suitable for re-use as trench backfill during grading of the project provided vegetation, topsoil containing more than 3 percent organic content by dry unit weight, and deleterious matter are removed. The trench backfill material may also consist of aggregate base or imported general fill material meeting the requirements of the “Re-Use of On-site Soil and Imported General Fill Material” section of this report. A BSK representative should be present on-site during grading to visually confirm the suitability of the soil to be used as trench backfill. Further discussion on the re-use of on-site soil as fill/backfill is presented in the “Earthwork” section of this report.

5.6.5 **Modulus of Soil Reaction for Buried Flexible Pipes**

Based on Jeyapalan (2001)\(^\text{19}\), the modulus of soil reaction for buried flexible pipes can be calculated using the equation below.

\[
E' = (S_C) \times (E'_b)
\]

Where:

- \(E'\) is the soil modulus of soil reaction for a buried flexible pipe (in units of psi).
- \(S_C\) is the soil support combination factor obtained from Table 6 below (unitless).
- The parameters needed to read Table 5 are defined below.
- \(E'_n\) is the modulus of soil reaction for the native soil surrounding the trench (in units of psi).
- \(B_d\) is the trench width (in units of inches).

---

- D is the outside diameter of the pipe (in units of inches).
- E’b is the modulus of soil reaction for the pipe embedment material (in units of psi).

Based on our findings, we recommend using an E’n value of 750 psi for the native clay underlying the Site. Table 7 below provides recommended E’b values for various materials based on compaction effort applied to the embedment material during trench backfill.

### Table 6
SOIL SUPPORT COMBINING FACTOR, S_c

<table>
<thead>
<tr>
<th>E’n/E’b</th>
<th>B_d/D = 1.5</th>
<th>B_d/D = 2.0</th>
<th>B_d/D = 2.5</th>
<th>B_d/D = 3.0</th>
<th>B_d/D = 4.0</th>
<th>B_d/D = 5.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>0.15</td>
<td>0.30</td>
<td>0.60</td>
<td>0.80</td>
<td>0.90</td>
<td>1.00</td>
</tr>
<tr>
<td>0.2</td>
<td>0.30</td>
<td>0.45</td>
<td>0.70</td>
<td>0.85</td>
<td>0.92</td>
<td>1.00</td>
</tr>
<tr>
<td>0.4</td>
<td>0.50</td>
<td>0.60</td>
<td>0.80</td>
<td>0.90</td>
<td>0.95</td>
<td>1.00</td>
</tr>
<tr>
<td>0.6</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
<td>0.95</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>0.8</td>
<td>0.85</td>
<td>0.90</td>
<td>0.95</td>
<td>0.98</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>1.0</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>1.5</td>
<td>1.30</td>
<td>1.15</td>
<td>1.10</td>
<td>1.05</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>2.0</td>
<td>1.50</td>
<td>1.30</td>
<td>1.15</td>
<td>1.10</td>
<td>1.05</td>
<td>1.00</td>
</tr>
<tr>
<td>3.0</td>
<td>1.75</td>
<td>1.45</td>
<td>1.30</td>
<td>1.20</td>
<td>1.08</td>
<td>1.00</td>
</tr>
<tr>
<td>≥5.0</td>
<td>2.00</td>
<td>1.60</td>
<td>1.40</td>
<td>1.25</td>
<td>1.10</td>
<td>1.00</td>
</tr>
</tbody>
</table>

### Table 7
MODULUS OF SOIL REACTION FOR THE PIPE EMBEDMENT MATERIAL, E’b

<table>
<thead>
<tr>
<th>Embedment Material</th>
<th>Level of Compaction¹</th>
<th>90%</th>
<th>95%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Native soil</td>
<td>750 psi¹</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Caltrans Class 2 AB</td>
<td>1,500 psi</td>
<td>2,000 psi</td>
<td></td>
</tr>
<tr>
<td>Silty Sand/Poorly Graded Sand (SP/SM)</td>
<td>1,350 psi</td>
<td>1,750 psi</td>
<td></td>
</tr>
<tr>
<td>CLSM</td>
<td>2,300 psi²</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Notes:**
1. Using the Modified Proctor test (ASTM D1557).
2. Level of compaction is not applicable to this type of material.

### 5.7 Exterior Flatwork

New exterior concrete flatwork at grade will be constructed on soils subject to swell/shrink cycles. Some of the adverse effects of swelling and shrinking can be reduced with proper moisture treatment. The intent is to reduce the fluctuations in moisture content by moisture conditioning the soils, sealing the moisture in,
and controlling it. Near-surface soils to receive exterior concrete flatwork should be moisture conditioned according to the recommendations in “Earthwork” section of this report. In addition, all exterior flatwork should be supported on a minimum of 12 inches of Caltrans Class 2 aggregate base.

Due the presence of highly expansive soils near the site surface, flatwork should have control joints (i.e., weakened plane joints) spaced no more than 8 feet on centers. Prior to construction of the flatwork, the 12 inches of aggregate base should be moisture conditioned to near optimum moisture content. If the aggregate base is not covered within about 30 days after placement, the soils below this material will need to be checked to confirm that their moisture content is at least 2 percent over optimum. If the moisture is found to be below this level, the flatwork areas will need to be soaked until the proper moisture content is reached. Where flatwork is adjacent to curbs, reinforcing bars should be placed between the flatwork and the curbs. Expansion joint material should be used between flatwork and buildings.

5.8 Demolition

5.8.1 Existing Utilities

Active or inactive utilities within the construction area should be protected, relocated, or abandoned. Pipelines that are 2 inches in diameter or less may be left in place beneath the planned structures provided they are cut off and capped at the structure perimeters. Pipelines larger than 2 inches in diameter within the planned structure footprint should be removed or filled with CLSM meeting the project specifications. Active utilities to be reused should be carefully located and protected during demolition and during construction.

5.8.2 Excavation and Backfill of Existing Foundations and Below-Grade Structures

If applicable, all existing foundations and below-grade structures to be abandoned should be demolished and removed. The resulting excavations should then be properly backfilled with compacted engineered fill per the requirements of the “Earthwork” section of this report. A BSK representative should observe and test the compaction for earthwork activities during construction.

5.8.3 Reuse of On-site Concrete, Asphalt Concrete, and Aggregate Base

Where applicable, existing AC may be pulverized and mixed with the underlying gravel layer (i.e., aggregate base) for use as general engineered fill under new asphalt pavement only if it meets the gradation requirements discussed in the “Re-Use of On-site Soils and Imported General Fill Material” section of this report.

Consideration should also be given to processing the existing concrete, AC, and underlying aggregate base for re-use in the lower 6 inches of the aggregate base layer for new paved areas, existing paved areas to be fully-reconstructed, and exterior flatwork. If these materials are reused, the processing should be performed in such a manner that these materials meet the gradation, R-Value, durability index, and sand
equivalent requirements of Section 26 of the 2015 Caltrans Standard Specifications, unless otherwise indicated by the Geotechnical Engineer-of-Record during construction. Also, the contractor should exercise extreme care not to contaminate the existing concrete, AC, and AB with the underlying clayey subgrade soils during removal or this could result in rejection of a portion or all the removed materials for use as aggregate base under new pavements and exterior flatwork.

5.9 Earthwork

Earthwork at the Site will generally consist of excavation, removal, and backfill of existing underground utility lines and below-grade structures (if present), excavation for the construction of the below-grade walls and foundations for the planned structures, backfill of below-grade walls, excavation for shallow foundations, excavation and backfill for new underground utility lines, and preparation of original grade and subgrade (including full depth reclamation) and aggregate base placement for pavement rehabilitation and possibly new concrete flatwork. Although grading plans are not currently available for the project, we assume that site grades will remain close to existing elevations and that cuts and fills during construction will be limited to less than about 5 feet in height. Excavations during demolition of existing improvements and for the construction of the proposed below-grade structures and associated underground utility lines are anticipated to be up to about 10 to 18 feet deep BGS. BSK should review the final grading plans for conformance to our design recommendations prior to construction bidding. In addition, it is important that a representative of BSK observe and evaluate the competency of existing soils or new fill underlying structures. In general, soft/loose or unsuitable materials encountered should be overexcavated, removed, and replaced with compacted engineered fill material.

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

5.9.1 Site Preparation and Grading

Prior to the start of grading and original grade preparation operations, the Site should first be cleared and stripped to remove all surface vegetation, deleterious materials, organic laden topsoil having more than 3 percent organic content by dry unit weight, and debris generated during the demolition of existing improvements located within the Site. Stripping to a minimum depth of 3 to 6 inches is expected to remove a majority of loose and organic laden surficial soils. If significant amounts of soil containing more than 3 percent organic content are encountered below this depth, additional stripping may be required. Stripping should extend a minimum of 5 feet laterally beyond planned structures and improvements, where feasible. Stripped topsoil having more than 3 percent organic content by dry unit weight may be

---

20 Ground elevation prior to grading.
21 Final soil elevation prior to placement of select fill, such as aggregate base course, and trench fine grading, or asphalt concrete, or concrete for foundations or other improvements.
stockpiled for later use in landscaping areas; however, this material should not be reused for engineered fill.

Any buried tree stumps, roots, or major root systems thicker than approximately 1-inch in diameter, and abandoned foundations uncovered during site stripping and/or grading activities should be removed.

Following stripping and removal of deleterious materials, the original grade for the planned structures and improvements should be scarified to a minimum depth of 12 inches, moisture conditioned, and recompressed as indicated in Appendix D except for the areas to undergo excavation for the proposed below-grade structures. Scarification should extend a minimum of 5 feet laterally beyond the planned structures and improvements, where feasible. All fills should be compacted in lifts of 8-inch maximum uncompacted thickness. A summary of compaction requirements for the project is presented in Appendix D. Laboratory maximum dry density and optimum moisture content relationships should be evaluated based on ASTM Test Designation D1557 (latest edition).

Proper moisture conditioning is very important. After subgrade soils are properly moisture conditioned, their moisture content should be maintained until they are covered by improvements. This may require periodic moisturizing of the subgrade soils.

Where access for compaction testing in deep excavations is limited due to trench stability, safety, and other access concerns, CLSM meeting the project specifications may be considered as an alternative to soil backfill. If this type of backfill material is used, the utility lines should be anchored to prevent the pipe from floating. The CLSM should be properly vibrated to allow backfilling under the spring line of the pipes affected.

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

5.9.2 Re-Use of On-site Soil and Imported General Fill Material

From a geotechnical standpoint only, the on-site soils are suitable for re-use as general engineered fill and backfill provided vegetation, topsoil containing more than 3 percent organic content by dry unit weight, and deleterious matter are removed. A BSK representative should be present on-site during grading to visually confirm the suitability of the soil to be used as fill and backfill. Particles larger than 3 inches within the on-site soils (if encountered), should either be removed and disposed off-site or broken down to 3 inches or less prior to using the soil as engineered fill. Nesting (i.e., concentration) of larger particles should be avoided to reduce the potential that this could create voids and allow future settlement in the overlaying fill/backfill.
Maximum particle size for fill material should be limited to 3 inches, with at least 90 percent by weight passing the 1-inch sieve. Proper granular bedding and embedment (i.e., shading/envelope) meeting the project specifications should be used beneath and around new utilities. On-site soils are not considered suitable for use as trench fine grading and pipe bedding (refer to the “Trench Fine Grading and Pipe Bedding” section of this report). Where imported general fill is required, it should be granular in nature, adhere to the above gradation recommendations, and conform to the minimum criteria presented in Table 8 below.

<table>
<thead>
<tr>
<th>TABLE 8 \IMPORTED GENERAL FILL CRITERIA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plasticity Index</td>
</tr>
<tr>
<td>Liquid Limit</td>
</tr>
<tr>
<td>% Passing #200 Sieve</td>
</tr>
<tr>
<td>Corrosivity</td>
</tr>
</tbody>
</table>

Highly pervious materials such as pea gravel or clean sands are not recommended as imported general fill because they permit transmission of water to the underlying soils and can allow migration of fines from the surrounding native soils, which could lead to loss of ground and resulting in ground settlement. Imported general fill material should not be any more corrosive than the on-site soils and should not be classified as being more corrosive than "moderately corrosive." Prior to transporting proposed imported general fill materials to the Site, the contractor should make representative samples of the material available to BSK at least 10 working days in advance to allow our representatives enough time to confirm the material meets the above requirements. All on-site or imported general fill material should be compacted to the recommendations provided for engineered fill in Appendix D.

5.9.3 Temporary Dewatering

We expect that cut and cover excavation methods and temporary shoring will be used to construct the proposed below-grade structures and install new underground utility lines. Some of these excavations are expected to extend to depths of about 15 to 20 feet BGS. We expect that excavations deeper than about 5 feet BGS (refer to Table 1 for estimated groundwater elevations for design for the Site) will need to be continuously dewatered during construction (depending on the time of year). The soils encountered within the maximum depth of our exploration (about 50 feet below the ground surface) consisted primarily of clay layers, but interbedded layers of sand were also encountered below groundwater in our borings and CPTs. Depending on their fines content (i.e., amount of material passing the No. 200 sieve), sand layers typically have much higher transmissivity rates than clay and silt layers. **Groundwater should be lowered and maintained at least 2 feet below the bottom of the planned excavations in order to maintain the undisturbed state of the supporting soils and to allow proper compaction of backfill after below-grade structures and utility lines are installed.** After completion of all below-grade structures and
utility lines, the dewatering operations may be terminated to allow the groundwater table to return to its natural level.

We anticipate that dewatering in the project area will be performed in stages and can be performed using deep wells, well points, sumps, drains, and open pumping. However, because sand layers were encountered in our borings and CPTs at depths ranging from about 14 to 23 feet BGS, slope stability and boiling at the bottom of the excavations could pose a problem. The contractor should be fully responsible for developing and implementing a dewatering program. This should include making any necessary adjustments to the dewatering program during construction based on actual field conditions encountered.

The successful implementation of the dewatering program for this project will be substantially determined by the experience and performance of the contractor retained to perform the dewatering. Therefore, we recommend that the general contractor for the project be required to retain the services of a specialty dewatering subcontractor to review the anticipated subsurface conditions, develop, and implement a proper dewatering program for the project. We recommend the use of a specialty dewatering subcontractor with a minimum of 5 to 10 years of continuous construction experience in similar subsurface conditions on projects of similar scope (i.e., depth of excavations, proximity to and type of existing structures and utility lines, etc.). The dewatering operation should also conform to the project specifications. The dewatering subcontractor selected should provide examples of dewatering for projects they have successfully completed in the past 5 years under similar subsurface conditions and similar scope to this project. The example projects should note instances when things went wrong during particular projects and how they were successfully remediated during construction.

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

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

5.9.4 Weather/Moisture Considerations

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

5.9.5 Excavation, Shoring, and Backfill

We anticipate that excavations can be made with standard earthwork equipment, such as excavators, dozers, backhoes, and trenchers. Where trenches or other excavations are extended deeper than 5 feet BGS, the excavation may become unstable and should be evaluated to monitor stability prior to personnel entering the excavations.

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

Where the stability of adjoining structures could be endangered by excavation operations or in areas where existing site features or constraints prevent the use of sloped excavations, support systems such as shoring, bracing, or underpinning may be required to provide structural stability and to protect personnel working within the excavation. Shoring should be removed as the excavations are backfilled. Shoring should be designed to resist earth pressures exerted by the retained soil plus any applicable surcharge loading, such as construction equipment and stockpiles. Table 9 below presents recommended equivalent fluid lateral earth pressures for use in the shoring design of solid sheeting and isolated soldier piles. Figure 6 presents pressure distributions for braced loads. Note that these distributions do not include surcharge loads due to stockpiled soils, construction equipment, or vehicular traffic and are based on the assumption that shored excavations will be properly dewatered during construction. A uniform surcharge pressure of 100 psf is typically applied over the upper 10 feet of an excavation to account for surcharge loading imposed by construction equipment and vehicular traffic.
TABLE 9
LATERAL EARTH PressURES FOR SHORING DESIGN

<table>
<thead>
<tr>
<th>Earth Pressures</th>
<th>Equivalent Fluid Pressures (pcf)¹</th>
</tr>
</thead>
<tbody>
<tr>
<td>Active</td>
<td>45</td>
</tr>
<tr>
<td>At-Rest</td>
<td>60</td>
</tr>
<tr>
<td>Passive</td>
<td>250²</td>
</tr>
</tbody>
</table>

**Notes:**
1. The lateral earth pressures presented herein are based on the assumption that the shoring will retain a level ground surface behind it, the retained soils have an effective soil unit weight of approximately 120 pcf, and a fully-drained soil condition will be maintained while the shoring is in place.
2. The passive pressure may be applied to a width of twice the diameter of the soldier piles. Note that the passive pressure provided herein is intentionally lower than what was provided in the “Resistance to Lateral Loads” section of this report because the portion of the shoring to mobilize passive pressure will likely extend into predominantly sand instead of clay according to our borings and CPTs.

The shoring designer should provide for a uniform and timely mobilization of soil pressures as the shoring system(s) are installed to reduce the amount of lateral movement of the shoring. Interior bracing should be loaded to the design loads prior to excavation of the underlying soil, so that load induced strains in the retaining system will not result in the system moving toward the excavation. If voids are created behind the shoring system by overexcavating, soil sloughing, etc., the voids should be filled by grout to reduce potential strains. **The design and installation of shoring, bracing, or underpinning required for the project should be the responsibility of the contractor and should be designed by a professional engineer registered in the State of California and should be based on the project specification requirements.** We recommend that the proposed shoring, bracing, and underpinning system design be submitted (along with the appropriate design calculations) in advance for review by the design team. The purpose of the review would be to evaluate whether proper soil parameters have been used and to confirm whether the anticipated deflections are within the tolerance established by the owner or its designer.

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

Care should be taken during construction to reduce the impact of trenching and excavations on adjacent structures and pavements. Excavations should be located so that no structures, foundations, and slabs, existing or new, are located above an imaginary plane projected 1H:1V upward from any point in an excavation unless the excavation is properly shored and excavated in stages. Furthermore, excavations should be set back a minimum of 10 feet horizontally measured from the outer edge of the perimeter foundation of existing structures to the excavation slope face. **If structures are located within this 1H:1V project line and/or this minimum setback is not feasible, the shoring should be designed to handle the applicable vertical and/or horizontal surcharge loading from the adjacent structure and allow no**

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¹ The Earth Pressures (Active, At-Rest, Passive) are given in pounds per square foot (pcf).
² The Passive Pressure may be applied to a width of twice the diameter of the soldier piles.
**horizontal movement of the excavation.** Prior to the installation of the shoring and excavation, monitoring points should be established immediately behind the shoring, at midway points between the adjacent structure and the shoring, and at the edge of the adjacent structure. These points should be surveyed daily during installation of the shoring and staged excavation. If any lateral movement is detected, the excavation operation should be stopped immediately and measures should be taken to halt further movement, such as placing a fill buttress in front of the shoring. The shoring design should then be reevaluated and revised as needed.

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

5.9.6  **Excavation for Below-Grade Structures**

5.9.6.1 Overexcavation

The sidewalls of below-grade structures will likely expose a layer of peat, especially at Oxidation Ditch No. 4 (OD4). According to our supplemental investigation and past investigation at the Site, the peat layer ranges from 1 to 5 feet in thickness and extends to depths of up to about 9 feet BGS. The peat layer will not provide suitable support for the sloped OD4 sidewalls. Also, if not removed, the peat layer could cause long-term settlement of the surficial improvements located immediately adjacent to OD4 and other proposed below-grade structures. It could also adversely impact passive resistance for these structures depending on the depth of their foundations. Therefore, where high organic content soils or peat are exposed on the sides of these excavations, we recommend overexcavating the entire perimeter of the proposed below-grade structures a minimum depth of 8 feet BGS (unless otherwise indicated in the project plans or specifications) to expose native soil. The overexcavation should extend a minimum of 5 feet beyond outer limits of the below-grade structures where feasible unless otherwise indicated in the project plans or specifications.

During excavation, the peat material and soil containing more than 3 percent organic content by dry unit weight should be segregated from soil material that is considered suitable for re-use as general fill. The peat and high organic content soils should then either be disposed offsite or used in landscaping areas only.

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22 Soils containing more than 3 percent organic content by dry unit weight.
5.9.6.2 Backfill

After the overexcavation is completed for OD4, we recommend one of the following alternatives be used to backfill the overexcavation:

1. **Alternative 1**: Backfill the entire excavation with properly compacted and moisture conditioned engineered fill meeting the requirements of Appendix D in lifts of 8-inch maximum uncompacted thickness. Once the backfill operation is completed, the area can be re-excavated to finished design subgrade elevations for the below-grade structures to expose a firm and stable bottom and side slopes.

2. **Alternative 2**: If desired, the overexcavation could also be backfilled to match the finished design subgrade elevations for the OD4. This would also have to be accomplished by placing properly compacted and moisture conditioned engineered fill meeting the requirements of Appendix D in lifts of 8-inch maximum uncompacted thickness. However, the side slopes would need to be overbuilt laterally a minimum of 1 foot and then be trimmed back to the finished design subgrade elevation to expose a firm and stable surface. **Due to the large size of the excavation for the OD4, we believe that Alternative 1 would be more cost-effective than Alternative 2.**

We anticipate that the overexcavation for below-grade structures with vertical sidewalls will be backfilled after the below-grade walls are constructed as part of the overall backfill behind such walls. The requirements discussed throughout the “Earthwork” section and Appendix D of this report, including maximum backfill lifts, should be followed during placement of backfill behind vertical below-grade walls.

5.9.6.3 Maintaining Moisture of Subgrade Soils

Due to the high to very high expansion potential of the surficial clay soils at the Site, after the excavation for the proposed below-grade structures is completed per the preceding sections above, it is critical that the surficial clay soils be kept properly moisturized. Otherwise, there is a very high potential that the soils at the sidewalls and bottom of the excavation will dry out and experience significant cracking. **If this were to occur, the contractor should be required to overexcavate the entire surface of the below-grade structures a minimum lateral distance of 2 feet or as require by BSK, moisture condition the excavated soils, and place them back per Alternatives 1 or 2 of the preceding section for the OD4 excavation or as properly compacted backfill behind vertical below-grade walls.** To reduce the risk of this happening, we recommend that the contractor properly maintain the moisture in the clay soils exposed at the bottom and sidewalls of excavations for below-grade structures via one of the following options.

1. **Option 1A (applies to OD4 excavation)**: Immediately after the excavation for the OD4 is completed, a 2- to 3-inch rat slab consisting of a 1-sack sand-cement slurry mix should be placed over the bottom and sidewalls of the OD4 excavation. If this option is selected, per the “Below-Grade Structures” section of this report, we recommend first placing a layer a minimum of 12 inches thick of compacted 1-inch crushed drain rock over the bottom of the excavation. Just
before the shotcrete facing is placed for the OD4, one or more apertures should be made through pre-selected location(s) at the bottom of the OD4 for the installation of hydraulic relief valves.

2. **Option 1B (applies to vertical below-grade walls):** Immediately after the excavation for the below-grade structures with vertical walls is completed, a 2- to 3-inch rat slab consisting of a 1-sack sand-cement slurry mix should be placed over the bottom of the excavation. If this option is selected, per the “Below-Grade Structures” section of this report, we recommend first placing a layer a minimum of 12 inches thick of compacted 1-inch crushed drain over the bottom of the excavation. **If this option is selected, one of the following two options would still be required for the vertical sidewalls.**

3. **Option 2:** Immediately after the excavation for the below-grade structures is completed, the entire exposed surface of the excavation should be covered with burlap and periodically moisturized. During hot and or windy weather, we recommend moisturizing the surface of the burlap a minimum of twice a day to keep the burlap moist. However, the intensity of water application and number of applications per day should be adjusted as needed in order to keep the burlap moist. The contractor should be responsible for the means and methods for periodically moisturizing the burlap, but these could consist of using garden hoses, fire houses, and sprinklers.

4. **Option 3:** Immediately after the excavation for the below-grade structures is completed, the entire exposed surface of the excavation could be covered with a vapor barrier consisting of a minimum 15 mil extruded polyolefin plastic, such as 15 mil Stego® Wrap vapor barrier or equivalent. **The limitation with this option is that any tears or punctures made before placement of the shotcrete facing is placed will need to be repaired immediately** or moisture will escape, which will result in drying of the surficial clayey soils that will in turn lead to significant cracking of these soils. **Another limitation for this option is that the vapor barrier will not allow the contractor to periodically add moisture to the soil below without first removing the vapor barrier.**

It is important to note that the contractor is responsible for maintaining proper moisture in the clayey soils exposed at the bottom and sidewalls of excavations for below-grade structures regardless of which option above is selected.

**5.9.7 Overexcavation of Foundations for At Grade Ancillary Structures**

The upper approximately 5 feet BGS below planned at grade ancillary structures should be potholed prior to construction of their foundations. Where peat or soils containing more than 3 percent organic content by dry unit weight are encountered, the entire area within 5 feet laterally of the foundation perimeter, where feasible, should be overexcavated to a depth of 5 feet BGS and then be backfilled with properly compacted aggregate base or on-site soil containing less than 3 percent organic content by dry unit weight.
5.9.8 Overexcavation and Backfill of Test Pits

Where new improvements are constructed over or within 5 feet laterally of the test pits performed for our supplemental investigation, the pertinent test pits should be overexcavated at least 2 feet past the depths indicated in our test pit logs during grading and the resulting excavation should then be backfilled to finished design grades with onsite soil suitable for use as general engineered fill properly compacted and moisture conditioned per the requirements of Appendix D. The overexcavated areas should measure 10 feet by 10 feet in plan dimensions. The approximate location of our test pits are shown on Figure 2 and their approximately coordinates are provided in the test pit logs in Appendix A.

5.10 New Asphalt Concrete Pavement

New asphalt concrete pavement sections for this project have been calculated using the Caltrans Flexible Pavement Design Method. We ran an R-Value test on a sample collected from the upper 5 feet BGS at boring B-2, which resulted in an R-Value of 9. Due to the potential variability of the site soils, we have based our pavement design sections on an R-value of 5.

Pavement designs for various Traffic Index values ranging from 5.0 to 9.0 based on an R-Value of 5 are presented in Table 10 below. These pavement design sections can also be used to perform full-depth pavement reconstruction of distressed pavement areas of the Site. Each TI represents a different level of use. Based on our experience, we believe the lower Traffic Index values are appropriate for lightly loaded vehicular parking and driveway areas, while the higher Traffic Index values are appropriate for heavily loaded driveways and the entrance driveway to Plant No. 2. Because portions of Plant No. 2 are underlain by surficial soils with high organic content or containing peat, we recommend that the pavement sections for Alternative 1 presented in Table 10 be underlain with a geotextile fabric consisting of Mirafi® RS280i or equivalent unless the project plans or specifications indicate a different geotextile fabric. If Alternative 2 is selected, then the AC layer should be underlain by a pavement reinforcing geotextile fabric, such as Petromat® or equivalent. The Owner or designer should determine which level of use best reflects the project and select appropriate pavement sections. The recommended pavement sections presented in Table 10 include factors of safety of 0.2 and 0.1 feet for Alternative 1 and Alternative 2, respectively, as per the Caltrans Design Manual.
TABLE 10
PAVEMENT DESIGN RECOMMENDATIONS
(R-VALUE = 5)

<table>
<thead>
<tr>
<th>Traffic Index</th>
<th>Alternative 1^3</th>
<th>Alternative 2^4</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AC^1</td>
<td>Class 2 AB^2</td>
</tr>
<tr>
<td></td>
<td>(inches)</td>
<td>(inches)</td>
</tr>
<tr>
<td>5.0</td>
<td>2.5</td>
<td>11.0</td>
</tr>
<tr>
<td>5.5</td>
<td>3.0</td>
<td>11.5</td>
</tr>
<tr>
<td>6.0</td>
<td>3.0</td>
<td>14.0</td>
</tr>
<tr>
<td>6.5</td>
<td>3.5</td>
<td>14.5</td>
</tr>
<tr>
<td>7.0</td>
<td>4.0</td>
<td>15.5</td>
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<td>7.5</td>
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<td>8.0</td>
<td>4.5</td>
<td>18.5</td>
</tr>
<tr>
<td>8.5</td>
<td>5.0</td>
<td>19.5</td>
</tr>
<tr>
<td>9.0</td>
<td>5.5</td>
<td>20.5</td>
</tr>
</tbody>
</table>

1. Asphalt Concrete
2. Caltrans Class 2 Aggregate Base (Minimum R-Value = 78)
3. If Alternative 1 is selected, the AB layer should be underlain with geotextile fabric consisting of Mirafi® RS280i or equivalent unless the project plans or specifications indicate a different geotextile fabric.
4. If Alternative 2 is selected, the AC layer should be underlain with a pavement reinforcing geotextile fabric consisting of Petromat® or equivalent.

We recommend that the subgrade soil over which the pavement sections are to be placed be moisture conditioned and compacted according to the recommendations in Appendix D. Subgrade preparation should extend a minimum of 3 feet laterally beyond the back of curb or edge of pavement, where feasible.

Paved areas should be sloped and drainage gradients maintained to carry all surface water to appropriate collection points. Surface water ponding should not be allowed anywhere on the Site during or after construction. We recommend that the pavement section be isolated from non-developed areas and areas of intrusion of irrigation water from landscaped areas. Concrete curbs should extend a minimum of 2 inches below the aggregate base and into the subgrade to provide a barrier against drying of the subgrade soils, or reduction of migration of landscape water, into the pavement section. Weep holes spaced at 4 feet on centers should also be provided. In lieu of the weep holes, a more effective system is to install a subdrain behind the curbs.

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

- All trench backfills underneath pavements, including utility and sprinkler lines, should be properly placed and adequately compacted to provide a stable subgrade, in accordance with the compaction recommendations in Appendix D;

- An adequate drainage system should be provided to prevent surface water or subsurface seepage from saturating the subgrade soil;
The asphalt concrete and aggregate base materials should conform to Caltrans Specifications, latest edition and project specifications; and

Placement and compaction of pavements should be performed and tested in accordance to appropriate Caltrans procedures.

5.11 Rehabilitation of Existing Pavements

5.11.1 Anticipated Pavement Rehabilitation Measures

Pavement rehabilitation measures for this project will consist of the temporary repair of AC driveways and longer-term rehabilitation of AC driveways and parking stalls. As discussed previously, we recommend that areas experiencing significant failures be temporarily repaired in the interim via localized excavation, and scarification and recompaction of the excavation bottom and then repaved. For long-term rehabilitation of the site pavements, we recommend that full-depth pavement reconstruction be used to rehabilitate the driveways experiencing significant pavement distress. In addition, we recommend that PCC pavement be used for the 90-degree turn of the L-shaped entrance driveway at Plant No. 2. Areas exposed to light vehicular traffic, such as parking areas and associated driveways, can be rehabilitated via application of crack sealing followed by application of a seal coat.

Our recommendations presented below are based on the assumption that the pavement subgrade soil is similar to the near surface soils described in the boring logs in Appendix A. We ran an R-Value test on a sample collected from the upper 5 feet BGS at boring B-2, which resulted in an R-Value of 9. Due to the potential variability of the site soils, we used an R-Value of 5 for design of the rehabilitation measures presented herein.

5.11.2 Temporary Repair of Distressed AC Driveways

Until the long-term repairs presented in subsequent sections of this report can be implemented during construction of this project, we recommend that existing AC driveway areas of Plant No. 2 currently experiencing significant pavement failure be temporarily repaired as follows:

1. The repair limits should extend a minimum of 2 feet laterally, where feasible, beyond the limits of significantly failed pavement areas.
2. The AC layer should be pulverized and the AC layer and a portion of the AB layer underlying it should be excavated and removed to a depth of 6 inches below existing grade.
3. The upper approximately 6 inches of the exposed excavation bottom should be scarified and recompacted per the requirements of the “Earthwork” section of this report.
4. The excavated areas should then be backfilled with at least 3 inches of AB and 3 inches of new AC. The new AB and AC layers should be compacted per the requirements of the “Earthwork” section of this report. The removed AB material may be re-used to backfill the repair areas provided it is not contaminated with the pulverized AC material, vegetation, or on-site soils.
5. Where there are existing depressions, more than 3 inches of AB may be required to match the finish grade of the repairs to existing grades beyond the repair areas. Where this is the case, imported AB or the pulverized AC material could be used to backfill the lower portion of the repair areas. If pulverized AC material is used, it should not contain particles greater than 1-inch in maximum size.

Note that the repair recommendations presented above should be considered temporary and are not expected to last beyond 6 months to 2 years. To help extend the longevity of these repairs, consideration should be given to placing a geotextile fabric, such as of Mirafi® RS280i or equivalent (unless the project plans or specifications indicate a different geotextile fabric), at the bottom of the repair excavations prior to placement of the AB layer.

5.11.3 Seal Coat (Long-Term Repair)

As discussed in the “Anticipated Pavement Rehabilitation Measures” section of this report, areas exposed to light vehicular traffic, such as parking areas and associated driveways, can be rehabilitated via application of crack sealing followed by application of a seal coat. A seal coat is a surface seal consisting of a mixture of emulsified asphalt, water, mineral fillers (i.e., aggregates), and various other mixtures. Surface seal mixtures that do not include mineral fillers are referred to as fog seals. Because they are considered to be a non-structural pavement rehabilitation measure, seal coats are generally recommended where the existing pavement is in good to very good condition or where pavement is first rehabilitated via localized pavement repairs.

We recommend that cracks wider than 1/8-inch be sealed with crack filler, such as Sikaflex®-1a or equivalent, prior to seal coat application. The crack filler material and application should meet the requirements of Section 37-5 of the 2015 Caltrans Standard Specifications. Materials, preparation, and placement of fog seal coats should conform to the requirements and provisions of Section 37-2 of the 2015 Caltrans Standard Specifications.

Prior to the placement of the asphaltic emulsion (i.e., seal coat), all existing painted and thermoplastic traffic stripes, pavement markings, and pavement markers should be removed in accordance with the provisions of Sections 81-8 and 84-9 of the 2015 Caltrans Standard Specifications.

Immediately prior to placing the asphalt binder, the existing pavement surface should be thoroughly cleaned of all vegetation, loose materials, dirt, mud, and all other extraneous materials and then allowed to dry. The asphalt binder should fill all minor cracks, depressions, or low areas, and leave a uniform surface free from ruts, humps, depressions, or irregularities. Any ridges, indentation, or other objectionable marks left in the surface should be eliminated by rolling, or other means. These cleaning procedures should be repeated prior to the application of the slurry seal.
5.11.4 Full-Depth Pavement Reconstruction (Long-Term Repair)

As discussed in the “Anticipated Pavement Rehabilitation Measures” section of this report, driveways experiencing significant pavement distress should be rehabilitated via full-depth pavement reconstruction. The existing AC, AB, and geotextile fabric layers should be removed from such driveways. The removed geotextile fabric should be disposed offsite, while the removed AC and AB material may be reused at the Site as discussed in the “Reuse of On-site Concrete, and Asphalt Concrete, and Aggregate Base” section of this report. Depending on the Traffic Index applicable to each pertinent driveway and the existing pavement section, the contractor may need to excavate the subgrade below the existing AB layer in order to provide sufficient depth for the new pavement section. One of the pavement sections presented in Table 10 above should then be used to reconstruct these driveways.

5.11.5 Portland Cement Concrete Pavement (Long-Term Repair)

The PCC pavement section should have a minimum thickness of 10 inches supported over 9 inches of Caltrans Class 2 aggregate base. The aggregate base and subgrade for the PCC Pavement section should be properly moisture conditioned and compacted. Because portions of Plant No. 2 are underlain by surficial soils with high organic content or containing peat, we recommend that the PCC pavement section be underlain with a geotextile fabric consisting of Mirafi® Tencate® RS280i or equivalent unless the project plans or specifications indicate a different geotextile fabric. Construction joints should be located no more than 12 feet apart in both directions. Concrete compressive strength should be tested in lieu of third point loading for rupture strength. A minimum 28-day compressive strength of 4,000 pounds per cubic foot (psi) should be specified for the concrete mix design. The PCC pavement should be continuously reinforced using No. 4 bars (or larger) spaced no more than 18 inches on center in both directions. Final design of the PCC pavement is the responsibility of the civil or structural engineer for the project.

5.12 Corrosion

Soil samples were collected at each Site during our field investigation at a depth of approximately 2½ feet BGS in both borings B-1 and B-2 and were submitted for corrosion testing. The samples were tested by CERCO Analytical, a State-certified laboratory in Concord, California, for redox potential, pH, resistivity, chloride content, and sulfate content in accordance with ASTM test methods. The test results are presented at the end of Appendix C. Also included is the evaluation by CERCO Analytical of the corrosion test results. Because we are not corrosion specialists, we recommend that a corrosion specialist be consulted for advice on proper corrosion protection for underground piping which will be in contact with the soils and other design details. CERCO’s evaluation included the following statements:

- “Based upon the resistivity measurements, both samples tested are classified as "severely corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel, and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.”
• “The chloride ion concentrations were 270 & 500 mg/kg and are determined to be sufficient to attack steel embedded in a concrete mortar coating. Chloride ion concentrations greater than 300 mg/kg are considered corrosive to embedded reinforcing steel; and, as such, the concrete mix design must be adjusted accordingly by a qualified corrosion engineer.”

• “The sulfate ion concentrations were 1200 & 1300 mg/kg and are determined to be sufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations. Therefore, concrete that comes into contact with this soil should use sulfate resistant cement such as Type II, in accordance with the California Building Code requirements with a maximum water-to-cement ratio of 0.50.”

• “The pH of the soils were 4.96 & 7.97 which does present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures. Any soils with a pH of <6.0 is considered to be corrosive to buried iron, steel, mortar-coated steel, and reinforced concrete structures. Therefore, corrosion prevention measures need to be considered for structures to be placed in the acidic soil.”

• “The redox potentials were 36 & 49-mV and are indicative of potentially "severely corrosive" soils resulting from anaerobic soil conditions.”

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

5.13 Plan Review and Construction Observation

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

Variations in soil types and conditions are possible and may be encountered during construction. To permit correlation between the soil data obtained during this investigation and the actual soil conditions encountered during construction, we recommend that BSK be retained to provide observation and testing services during site earthwork and foundation construction. This will allow us the opportunity to compare actual conditions exposed during construction with those encountered in our investigation and to provide supplemental recommendations if warranted by the exposed conditions. Earthwork should be performed in accordance with the recommendations presented in this report, or as recommended by BSK during
construction. BSK should be notified at least two weeks prior to the start of construction and prior to when observation and testing services are needed.
ADDITIONAL SERVICES AND LIMITATIONS

6.1 Additional Services

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

- review of plans and specifications;
- observations of site grading, including stripping and engineered fill construction;
- observation of foundation and below-grade wall excavations;
- Observation and testing of pavement rehabilitation operations; and
- in-place density testing of fills, backfills, and finished subgrades.

6.2 Limitations

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

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

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

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

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

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

We recommend that all earthwork during construction be monitored by a representative from BSK, including site preparation, foundation excavation, placement of engineered fill, trench/wall backfill, and pavement rehabilitation. The purpose of these services would be to provide BSK the opportunity to observe the actual soil conditions encountered during construction, evaluate the applicability of the recommendations presented in this report to the soil conditions encountered, and recommend appropriate changes in design or construction procedures if conditions differ from those described herein.
FIGURES
The information included on this graphic representation has been compiled from a variety of sources and is subject to change without notice. BSK makes no representations or warranties, express or implied, as to accuracy, completeness, timeliness, or rights to the use of such information. This document is not intended for use as a land survey product nor is it designed or intended as a construction design document. The use or misuse of the information contained on this graphic representation is at the sole risk of the party using or misusing the information.


ASSOCIATES

PROJECT NO. G19-194-11L
DRAWN: 10/22/20
DRAWN BY: D. Tower
CHECKED BY: C. Melo
FILE NAME: Figures.indd

Denitrification Basin Project
Wastewater Treatment Plant
Discovery Bay, California

FIGURE 1
Approximate Limits of Denitrification Basins

Approximate Limits of Pump Stations

Approximate Limits of Oxidation Ditch No. 4

The information included on this graphic representation has been compiled from a variety of sources and is subject to change without notice. BSK makes no representations or warranties, express or implied, as to accuracy, completeness, timeliness, or rights to the use of such information. This document is not intended for use as a land survey product nor is it designed or intended as a construction design document. The use or misuse of the information contained on this graphic representation is at the sole risk of the party using or misusing the information.

Entrance driveway for Plant No. 2 (17501 Highway 4) of the wastewater treatment plant (looking Northeast).

Entrance driveway for Plant No. 2 (looking East).
Driveway within Plant No. 2 (17501 Highway 4) of the wastewater treatment plant (looking West).

Vehicular parking area near entrance to Plant No. 2 (looking south).
\[ \sigma_h = 0.65 K_A \gamma H \]

WHERE \( K_A = \tan^2 (45 - \phi/2) \)

\( \gamma = \) soil unit weight (pcf)
\( \phi = \) soil friction angle (degrees)
\( c = \) cohesion (psf)
\( K_A = \) active pressure coefficient (unitless)
\( H = \) depth of excavation (feet)
\( \sigma_h = \) horizontal stress (psf)
\( N_o = \) stability number (unitless)

Reference: Figure 6, NAVFAC DM 7.02 (1982).
Correctly and Incorrectly Placed Leveling Wedges

Correctly Placed Leveling Wedges for Overcoming Excessive Crown
APPENDIX A

BORING AND TEST PIT LOGS
# Unified Soil Classification System (ASTM D2487/2488)

## Major Divisions

<table>
<thead>
<tr>
<th>Major Divisions</th>
<th>Graphic Log</th>
<th>Typical Descriptions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clean Gravels with &lt;5% Fines</td>
<td>GW</td>
<td>Well-graded gravels, gravel-sand mixtures with little or no fines</td>
</tr>
<tr>
<td>Gravels with 5 to 12% Fines</td>
<td>GP</td>
<td>Poorly-graded gravels, gravel-sand mixtures with little or no fines</td>
</tr>
<tr>
<td>Gravels with &gt;12% Fines</td>
<td>GW-GM</td>
<td>Well-graded gravels, gravel-sand mixtures with little clay fines</td>
</tr>
<tr>
<td>Clean Sands with &lt;5% Fines</td>
<td>SW</td>
<td>Well-graded sands, sand-gravel mixtures with little or no fines</td>
</tr>
<tr>
<td>Sands with 5 to 12% Fines</td>
<td>SP</td>
<td>Poorly-graded sands, sand-gravel mixtures with little or no fines</td>
</tr>
<tr>
<td>Sands with &gt;12% Fines</td>
<td>SM</td>
<td>Silty sands, sand-gravel-silt mixtures</td>
</tr>
<tr>
<td>Silts and Clays (Liquid limit less than 50)</td>
<td>ML</td>
<td>Inorganic silts and very fine sands, silty or clayey fine sands, silts with slight plasticity, gravelly clays, sandy clays, silty clays, lean clays</td>
</tr>
<tr>
<td>Silts and Clays (Liquid limit greater than 50)</td>
<td>OL</td>
<td>Organic silts &amp; organic silty clays of low plasticity</td>
</tr>
<tr>
<td></td>
<td>MH</td>
<td>Inorganic silts, micaceous or diatomaceous fine sand or silt</td>
</tr>
<tr>
<td></td>
<td>OH</td>
<td>Organic clays &amp; organic silts of medium-to-high plasticity</td>
</tr>
</tbody>
</table>

## Descriptions

- **Gravels** (More than half of coarse fraction is larger than the #4 sieve)
- **Sands** (More than half of coarse fraction is smaller than the #4 sieve)
- **Silt and Clays**
  - **Clean Gravels with <5% Fines**
  - **Gravels with 5 to 12% Fines**
  - **Gravels with >12% Fines**
  - **Clean Sands with <5% Fines**
  - **Sands with 5 to 12% Fines**
  - **Sands with >12% Fines**
  - **Inorganic Silts and Very Fine Sands**
  - **Organic Silts & Organic Silty Clays of Low Plasticity**
  - **Inorganic Silts, Micaceous or Diatomaceous Fine Sand or Silt**
  - **Organic Clays & Organic Silts of Medium-to-High Plasticity**

---

**Figure A-1**

Denitrification Basin Project
Wastewater Treatment Plant
Discovery Bay, California
### SOIL DESCRIPTION KEY

#### MOISTURE CONTENT

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>ABBR</th>
<th>FIELD TEST</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry</td>
<td>D</td>
<td>Absence of moisture, dusty, dry to the touch</td>
</tr>
<tr>
<td>Moist</td>
<td>M</td>
<td>Damp but no visible water</td>
</tr>
<tr>
<td>Wet</td>
<td>W</td>
<td>Visible free water, usually soil is below water table</td>
</tr>
</tbody>
</table>

#### FIRMNESS

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

#### REACTION WITH HCL

- **Weakly**
  - Crumbles or breaks with handling or slight finger pressure
- **Moderately**
  - Crumbles or breaks with considerable finger pressure
- **Strongly**
  - Will not crumble or break with finger pressure

#### GRAIN SIZE

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>SIEVE SIZE</th>
<th>GRAIN SIZE</th>
<th>APPROXIMATE SIZE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Boulders</td>
<td>&gt;12&quot;</td>
<td>Larger than basketball-sized</td>
<td></td>
</tr>
<tr>
<td>Cobble</td>
<td>3 - 12&quot;</td>
<td>Fishtailed to basketball-sized</td>
<td></td>
</tr>
<tr>
<td>Gravel</td>
<td>coarse: 3/4&quot; - 3&quot;</td>
<td>Thumb-sized to fist-sized</td>
<td></td>
</tr>
<tr>
<td></td>
<td>fine: 0.19&quot; - 0.75&quot;</td>
<td>Pea-sized to thumb-sized</td>
<td></td>
</tr>
<tr>
<td>Sand</td>
<td>coarse: #10 - #40</td>
<td>Rock salt-sized to pea-sized</td>
<td></td>
</tr>
<tr>
<td></td>
<td>medium: #40 - #10</td>
<td>Sugar-sized to rock salt-sized</td>
<td></td>
</tr>
<tr>
<td></td>
<td>fine: #200 - #400</td>
<td>Flour-sized to sugar-sized</td>
<td></td>
</tr>
<tr>
<td>Fines</td>
<td>Passing #200</td>
<td>&lt;0.0029</td>
<td>Flour-sized and smaller</td>
</tr>
</tbody>
</table>

#### ANGULARITY

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

#### STRUCTURE

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

#### REACTION WITH HCL

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>FIELD TEST</th>
</tr>
</thead>
<tbody>
<tr>
<td>None</td>
<td>No visible reaction</td>
</tr>
<tr>
<td>Weak</td>
<td>Some reaction, with bubbles forming slowly</td>
</tr>
<tr>
<td>Strong</td>
<td>Violent reaction, with bubbles forming immediately</td>
</tr>
</tbody>
</table>

#### APPARENT DENSITY - COARSE-GRAINED SOIL

<table>
<thead>
<tr>
<th>APPARENT DENSITY</th>
<th>ABBR</th>
<th>SPT (## blows/ft)</th>
<th>MODIFIED CA SAMPLER (## blows/ft)</th>
<th>CALIFORNIA SAMPLER (## blows/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Loose</td>
<td>VL</td>
<td>&lt;4</td>
<td>&lt;4</td>
<td>&lt;5</td>
</tr>
<tr>
<td>Loose</td>
<td>L</td>
<td>4 - 10</td>
<td>5 - 12</td>
<td>5 - 15</td>
</tr>
<tr>
<td>Medium Dense</td>
<td>MD</td>
<td>10 - 30</td>
<td>12 - 35</td>
<td>15 - 40</td>
</tr>
<tr>
<td>Dense</td>
<td>D</td>
<td>30 - 50</td>
<td>35 - 60</td>
<td>40 - 70</td>
</tr>
<tr>
<td>Very Dense</td>
<td>VD</td>
<td>&gt;50</td>
<td>&gt;60</td>
<td>&gt;70</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>RELATIVE DENSITY (%)</th>
<th>FIELD TEST</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 15</td>
<td>Easily penetrated with 1/2-inch reinforcing rod by hand</td>
</tr>
<tr>
<td>15 - 35</td>
<td>Difficult to penetrate with 1/2-inch reinforcing rod pushed by hand</td>
</tr>
<tr>
<td>35 - 65</td>
<td>Easily penetrated a foot with 1/2-inch reinforcing rod driven with 5-lb. hammer</td>
</tr>
<tr>
<td>65 - 85</td>
<td>Difficult to penetrate a foot with 1/2-inch reinforcing rod driven with 5-lb. hammer</td>
</tr>
<tr>
<td>85 - 100</td>
<td>Penetrated only a few inches with 1/2-inch reinforcing rod driven with 5-lb. hammer</td>
</tr>
</tbody>
</table>

#### OTHER COMMENTS

- Denitrification Basin Project
- Wastewater Treatment Plant
- Discovery Bay, California

---

**PROJECT NO.** G19-194-11L

**DRAWN:** 11/4/19

**DRAWN BY:** D. Tower

**CHECKED BY:** C. Melo

**FILE NAME:** Legend.indd

**FILE:** Figure A-2
**LOG SYMBOLS**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image" alt="Bulk / Bag Sample" /></td>
<td>Bulk / Bag Sample</td>
<td>-4 PERCENT FINER THAN THE NO. 4 SIEVE (ASTM Test Method C 136)</td>
</tr>
<tr>
<td><img src="image" alt="Split Barrel Sampler" /></td>
<td>Split Barrel Sampler (2-1/2 inch outside diameter)</td>
<td>-200 PERCENT FINER THAN THE NO. 200 SIEVE (ASTM Test Method C 117)</td>
</tr>
<tr>
<td><img src="image" alt="Split Barrel Sampler" /></td>
<td>Split Barrel Sampler (3 inch outside diameter)</td>
<td>LL LIQUID LIMIT (ASTM Test Method D 4318)</td>
</tr>
<tr>
<td><img src="image" alt="Standard Penetration Split Spoon Sampler" /></td>
<td>Standard Penetration Split Spoon Sampler (2 inch outside diameter)</td>
<td>PI PLASTICITY INDEX (ASTM Test Method D 4318)</td>
</tr>
<tr>
<td><img src="image" alt="Continuous Core" /></td>
<td>Continuous Core</td>
<td>TXUU UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION (EM 1110-1-1906)/ASTM Test Method D 2850</td>
</tr>
<tr>
<td><img src="image" alt="Shelby Tube" /></td>
<td>Shelby Tube</td>
<td>EI EXPANSION INDEX (UBC STANDARD 18-2)</td>
</tr>
<tr>
<td><img src="image" alt="Rock Core" /></td>
<td>Rock Core</td>
<td>COL COLLAPSE POTENTIAL</td>
</tr>
<tr>
<td><img src="image" alt="Groundwater Level" /></td>
<td>Groundwater Level (encountered at time of drilling)</td>
<td>UC UNCONFINED COMPRESSION (ASTM Test Method D 2166)</td>
</tr>
<tr>
<td><img src="image" alt="Groundwater Level" /></td>
<td>Groundwater Level (measured after drilling)</td>
<td>MC MOISTURE CONTENT (ASTM Test Method D 2216)</td>
</tr>
<tr>
<td><img src="image" alt="Seepage" /></td>
<td>Seepage</td>
<td></td>
</tr>
</tbody>
</table>

**GENERAL NOTES**

Boring log data represents a data snapshot. This data represents subsurface characteristics only to the extent encountered at the location of the boring. The data inherently cannot accurately predict the entire subsurface conditions to be encountered at the project site relative to construction or other subsurface activities. Lines between soil layers and/or rock units are approximate and may be gradual transitions. The information provided should be used only for the purposes intended as described in the accompanying documents. In general, Unified Soil Classification System designations presented on the logs were evaluated by visual methods. Where laboratory tests were performed, the designations reflect the laboratory test results.

The Responsible Geotechnical Engineer, Professional Engineer, or Professional Geologist uses professional judgement and visual-manual procedures in general conformance with ASTM D2488 to classify soil when the full classification suite of tests per ASTM D2487 is not conducted.
**LOG OF BORING NO. B-2**

**Surface El.:** 91 feet (Plant Datum)

**Location:** 37.8894, -121.58469

---

### MATERIAL DESCRIPTION

<table>
<thead>
<tr>
<th>Depth, feet</th>
<th>Surface El.: 91 feet (Plant Datum)</th>
<th>Location: 37.8894, -121.58469</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>ASPHALT: approximately 2.5 inches of asphalt</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>GRAVEL: approximately 3 inches of possible aggregate base (FILL)</td>
<td></td>
</tr>
<tr>
<td>5-10</td>
<td>Fat CLAY with Sand (CL): dark yellowish brown to very dark brown, moist, hard, medium to high plasticity, fine grained sand, mottled with organic matter</td>
<td></td>
</tr>
<tr>
<td></td>
<td>R-Value =9 (see Figure C-3), firm to hard, increased organic content</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Organic Content= 7%</td>
<td></td>
</tr>
<tr>
<td>5-10</td>
<td>hard, mottled, medium to high plasticity</td>
<td></td>
</tr>
<tr>
<td>5-10</td>
<td>Lean CLAY (CL): olive gray, moist, firm to hard, medium to high plasticity, trace fine grained sand</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Organic Content= 3%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>TXUU (see Figure C-2) c= 2,250 psf</td>
<td></td>
</tr>
<tr>
<td>5-10</td>
<td>9:50 am, 10/31/19</td>
<td></td>
</tr>
<tr>
<td></td>
<td>iron oxide staining, organic odor</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>Lean CLAY with Sand (CL): yellowish brown, moist, firm, medium plasticity, fine grained sand, manganese oxide staining</td>
<td></td>
</tr>
<tr>
<td></td>
<td>TXUU (see Figure C-2) c= 1,100 psf</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>8:15 am, 10/31/19</td>
<td></td>
</tr>
<tr>
<td></td>
<td>soft to firm, clayey sand lens</td>
<td></td>
</tr>
<tr>
<td></td>
<td>TXUU (see Figure C-2) c= 770 psf</td>
<td></td>
</tr>
<tr>
<td>15-20</td>
<td>Clayey SAND (SC): yellowish brown, moist, loose, fine grained sand</td>
<td></td>
</tr>
</tbody>
</table>

---

### LOG OF BORING NO. B-2

<table>
<thead>
<tr>
<th>Depth, feet</th>
<th>Sample Number</th>
<th>Penetration Blows / 6 inches</th>
<th>Pocket Penetrometer, TSF</th>
<th>% Passing No. 200 Sieve</th>
<th>In-Situ Dry Weight (pcf)</th>
<th>In-Situ Moisture Content (%)</th>
<th>Liquid Limit</th>
<th>Plastic Limit</th>
<th>Plasticity Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1A</td>
<td>9</td>
<td>4.5</td>
<td>84</td>
<td>31</td>
<td>67</td>
<td>28</td>
<td>39</td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>1B</td>
<td>13</td>
<td>4.5</td>
<td>3.5</td>
<td>96</td>
<td>28</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>1C</td>
<td>18</td>
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<td>4</td>
<td>4.5</td>
<td>3.5</td>
<td>96</td>
<td>28</td>
<td></td>
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</tr>
<tr>
<td>5-10</td>
<td>3A</td>
<td>5</td>
<td>4.5</td>
<td>3.5</td>
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<td></td>
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<tr>
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<td>7A</td>
<td>3</td>
<td>4.5</td>
<td>3.5</td>
<td>96</td>
<td>28</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>10-20</td>
<td>7B</td>
<td>3</td>
<td>4.5</td>
<td>3.5</td>
<td>96</td>
<td>28</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10-20</td>
<td>7C</td>
<td>4</td>
<td>4.5</td>
<td>3.5</td>
<td>96</td>
<td>28</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

**Completion Depth:** 31.5

**Date Started:** 10/31/19

**Date Completed:** 10/31/19

**California Sampler:** 2.5-inch inner diameter

**SPT Sampler:** 2.5-inch inner diameter

**Drilling Equipment:** Taber Drilling CME 55 Track Rig

**Drilling Method:** Hollow Stem

**Drive Weight:** 140 lbs

**Hole Diameter:** 8-in

**Drop:** 30-in

**Remarks:**
**LOG OF BORING NO. B-2**

**Project Name:** Discovery Bay Denitrification Basin Project
**Project Number:** G19-194-11L
**Project Location:** Discovery Bay WWTP

**Logged by:** O. Khan  
**Checked by:** M. Romero

---

**Surface El.:** 91 feet (Plant Datum)  
**Location:** 37.8894, -121.58469

---

**MATERIAL DESCRIPTION**

- **Clayey SAND (SC):** yellowish brown, moist, loose, fine-grained sand (continued)
- **Sandy Lean CLAY (CL):** yellowish brown, very moist, soft to firm, medium plasticity, fine to medium grained sand increased in fines content, interbedded with sandy lean clay
- **Silty SAND (SM):** yellowish brown, wet, loose, fine-grained sand
- **Lean CLAY (CL):** yellowish brown, moist, firm, medium plasticity

Boring terminated at approximately 31.5 feet. Free groundwater was first observed at approximately 15 feet and then rose to approximately 7.5 feet before borehole backfilled. Boring was backfilled with cement grout and topped with approximately 6 inches of Quikrete.

---

**Completion Depth:** 31.5  
**Date Started:** 10/31/19  
**Date Completed:** 10/31/19  
**Hole Diameter:** 8-in  
**Drop:** 30-in  

---

**Drilling Equipment:** Taber Drilling CME 55 Track Rig  
**Drilling Method:** Hollow Stem  
**Drive Weight:** 140 lbs  
**SPT Sampler:** 2.5-inch inner diameter  
**Remarks:**

---

### Graphic Log

- **Samples**
  - 8A
  - 8B
  - 8C
  - 9

- **Penetration**
  - 3
  - 4
  - 5
  - 0

- **Pocket Penetrometer, TSF**
  - 0.5

- **In-Situ Dry Weight (pcf)**

- **In-Situ Moisture Content (%)**

- **Plastic Limit**

- **Plasticity Index**

- **Liquid Limit**

- **In-Situ Moisture Content (%)**
**LOG OF BORING NO. B-20-1**

**Project Name:** Discovery Bay Denitrification Basin Project  
**Project Number:** G19-194-11L  
**Project Location:** Discovery Bay WWTP  
**Logged by:** K. Yang  
**Checked by:** M. Romero

### MATERIAL DESCRIPTION

**Sandy Lean CLAY (CL):** dark grayish brown, moist, fine to medium grained sand, trace fine to coarse gravel (FILL)

**Elastic SILT (MH):** dark gray, moist, firm to hard, medium plasticity, fine grained sand, iron oxide staining

**Silty SAND (SM):** yellowish brown, wet, very loose, fine to medium grained sand

**Lean CLAY with Sand (CL):** yellowish brown, moist, firm, medium plasticity, fine grained sand

### LOG OF BORING NO. B-20-1

<table>
<thead>
<tr>
<th>Depth, feet</th>
<th>Surface El.: 89 feet (Plant Datum)</th>
<th>Location: 37.88933, -121.58311</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>10:00 AM, 9/23/20</td>
<td>yellowish brown to grayish brown, soft to firm, porous TXUU (see figure C-7) c= 690 psf</td>
</tr>
<tr>
<td>5</td>
<td>9:00 AM, 9/23/10</td>
<td>Sandy Lean CLAY (CL): yellowish brown, very moist, medium plasticity, fine grained sand</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>Sitty SAND (SM): yellowish brown, wet, very loose, fine to medium grained sand</td>
</tr>
<tr>
<td>15</td>
<td></td>
<td>Lean CLAY with Sand (CL): yellowish brown, moist, firm, medium plasticity, fine grained sand</td>
</tr>
<tr>
<td>20</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Drilling Equipment:
- **Drilling Equipment:** Taber Drilling Dierich 120
- **Drilling Method:** Hollow Stem
- **Drive Weight:** 140 lbs
- **Hole Diameter:** 8-in
- **Drop:** 30-in

### Project Information:
- **Completion Depth:** 31.5
- **Date Started:** 9/23/20
- **Date Completed:** 9/23/20
- **California Sampler:** 2.5-inch inner diameter
- **SPT Sampler:** 1.4-inch inner diameter

---

**In-Situ Dry Weight (pcf)**  
**Liquid Limit**  
**Plastic Limit**  
**Plastic Index**  
**% Passing No. 200 Sieve**  
**Pocket Penetrometer, TSF**  
**In-Situ Moisture Content (%)**
Lean CLAY with Sand (CL): yellowish brown, moist, firm, medium plasticity, fine grained sand (continued)

Sandy Lean CLAY (CL): yellowish brown, wet, soft to firm, medium plasticity, fine grained sand

Boring terminated at approximately 31.5 feet. Free groundwater was first observed at approximately 15 feet. Boring was backfilled with cement grout.
**LOG OF BORING NO. B-20-2**

**MATERIAL DESCRIPTION**

<table>
<thead>
<tr>
<th>Depth, feet</th>
<th>Graphic Log</th>
</tr>
</thead>
<tbody>
<tr>
<td>93 feet (Plant Datum)</td>
<td></td>
</tr>
<tr>
<td>Location: 37.88844, -121.58311</td>
<td></td>
</tr>
</tbody>
</table>

**Sandy Lean CLAY (CL):** dark brown to yellowish brown, moist, firm to hard, medium plasticity, fine to medium grained sand, trace fine subangular gravel, interbedded with clayey sand (FILL)

**PEAT with Sand (PT):** black, moist, firm to hard, medium plasticity, fine to medium grained sand, high organic content

Organic content = 42%

**Fat CLAY (CH):** pale olive mottled with gray, moist, firm, medium plasticity, trace fine grained sand

1:22 pm, 9/23/20

**Lean CLAY with Sand (CL):** yellowish brown, wet, soft, low to medium plasticity, fine grained sand

11:40 am, 9/23/20

**Surface El.:** 93 feet (Plant Datum)

**Location:** 37.88844, -121.58311

---

**LOG OF BORING NO. B-20-2**

| Completion Depth: | 30.0 |
| Date Started: | 9/23/20 |
| Date Completed: | 9/23/20 |
| California Sampler: | 2.5-inch inner diameter |
| SPT Sampler: |  |

**Drilling Equipment:** Taber Drilling Dierich 120
**Drilling Method:** Hollow Stem
**Drive Weight:** 140 lbs
**Hole Diameter:** 8-in
**Drop:** 30-in

**Remarks:**
**LOG OF BORING NO. B-20-2**

- **Project Name:** Discovery Bay Denitrification Basin Project
- **Project Number:** G19-194-11L
- **Project Location:** Discovery Bay WWTP
- **Logged by:** K. Yang
- **Checked by:** M. Romero

### MATERIAL DESCRIPTION

<table>
<thead>
<tr>
<th>Depth, feet</th>
<th>Surface Elevation: 93 feet (Plant Datum)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location:</td>
<td>37.88844, -121.58311</td>
</tr>
</tbody>
</table>

#### Lean CLAY with Sand (CL): yellowish brown, wet, soft, low to medium plasticity, fine grained sand (continued)

- | Sample | Pocket Penetrometer, TSF | In-Situ Dry Weight (pcf) | Liquid Limit | Plastic Limit |
- | 6A     | 2                        |                          |              |              |
- | 6B     | 2                        |                          |              |              |
- | 6C     | 4                        |                          |              |              |

- | Sample | Penetration Blows / 6 inches | % Passing No. 200 Sieve | In-Situ Moisture Content (%) | Plastic Index |
- | 7A     | 4                        | 0.5                     |                         |              |
- | 7B     | 5                        | 1.5                     |                         |              |
- | 7C     | 5                        |                         |                         |              |

- Grayish brown, wet, soft to firm, increased fine to medium grained sand content with depth

Boring terminated at approximately 30 feet. Free groundwater was first observed at approximately 20 feet. Boring was backfilled with cement grout.

### Drilling Details

- **Completion Depth:** 30.0
- **Date Started:** 9/23/20
- **Date Completed:** 9/23/20
- **California Sampler:** 2.5-inch inner diameter
- **SPT Sampler:**

**Drilling Equipment:** Taber Drilling Dierich 120

**Drilling Method:** Hollow Stem

**Drive Weight:** 140 lbs

**Hole Diameter:** 8-in

**Drop:** 30-in

**Remarks:**
### TABULATED LOGS OF TEST PITS

<table>
<thead>
<tr>
<th>Location</th>
<th>Approximate Depth BGS (feet)</th>
<th>Description</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP-20-1</td>
<td>0 to 1</td>
<td>Clayey Sand with Gravel (SC) – yellowish brown, moist, fine- to medium-grained sand, fine subrounded gravel (FILL)</td>
<td>- Performed on 9/24/2020 - Approximate test pit coordinates: 37.88903, -121.58303 - Estimated ground surface elev. = 89 feet (plant datum) - Test pit terminated at approximately 15 feet BGS - Atterberg limits testing at approximately 1½ feet BGS - Organics Content = 15% at approximately 1½ feet BGS</td>
</tr>
<tr>
<td></td>
<td>1 to 3</td>
<td>Sandy Organic Silt (OH) – very dark gray to black, fine grained sand</td>
<td>- Free groundwater observed at approx. 10½ feet BGS</td>
</tr>
<tr>
<td></td>
<td>3 to 15</td>
<td>Lean Clay (CL) – gray to dark gray, moist, firm to hard, medium to high plasticity, hard, iron oxide stains, PP = 3.0 to &gt;4.5</td>
<td>- Approximately test pit coordinates: 37.88894, -121.58329 - Estimated ground surface elevation = 93 feet (plant datum) - Test pit terminated at approximately 15 feet - No free groundwater observed - Yellowish brown to grayish brown at approximately 10 feet BGS</td>
</tr>
<tr>
<td>TP-20-2</td>
<td>0 to 2</td>
<td>Sandy Lean Clay with Gravel (CL) – dark grayish brown, dry, hard, low to medium plasticity, fine to medium grained sand, fine to coarse gravel, interbedded with clayey sand, PP = &gt;4.5 (FILL)</td>
<td>- Performed on 9/24/2020 - Approximate test pit coordinates: 37.88894, -121.58329 - Estimated ground surface elevation = 93 feet (plant datum) - Test pit terminated at approximately 15 feet - No free groundwater observed - Yellowish brown to grayish brown at approximately 10 feet BGS</td>
</tr>
<tr>
<td></td>
<td>2 to 6</td>
<td>Sandy Lean Clay (CL) – mottled yellowish brown to dark grayish brown, moist, hard, medium plasticity (FILL)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>6 to 8</td>
<td>Peat (PT) – very dark gray</td>
<td></td>
</tr>
<tr>
<td></td>
<td>8 to 15</td>
<td>Sandy Lean Clay (CL) – gray, moist, med. plasticity, iron oxide staining</td>
<td></td>
</tr>
<tr>
<td>TP-20-3</td>
<td>0 to 3½</td>
<td>Clayey Sand with Gravel to Sandy Lean Clay with Gravel (SC/CL) – yellowish brown, dry to moist, rootlets, loose, low to medium plasticity, fine to medium grained sand, fine to coarse gravel, geotextile fabric (FILL)</td>
<td>- Performed on 9/24/2020 - Approximate test pit coordinates: 37.88877, -121.58304 - Estimated ground surface elevation = 93 feet (plant datum) - Test pit terminated at approximately 15 feet BGS - Free groundwater observed at approximately 13 feet BGS - Atterberg limits testing at approximately 6 feet BGS (see Figure C-1): LL = 94, PI = 19 - Organic Content = 40% at approximately 6 feet BGS</td>
</tr>
<tr>
<td></td>
<td>3½ to 5½</td>
<td>Lean Clay with Sand (CL) – dark yellowish brown to dark olive brown, moist, medium plasticity, fine grained sand, PVC pipe fragments, PP = &gt;4.5 (FILL)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5½ to 6</td>
<td>Poorly Graded Sand with Gravel (SP) – yellowish brown, loose, fine to medium grained sand, fine gravel (FILL)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>6 to 7½</td>
<td>Peat (PT) – very dark gray, moist</td>
<td></td>
</tr>
<tr>
<td></td>
<td>7½ to 15</td>
<td>Sandy Lean Clay (CL) – gray, moist, medium plasticity, fine grained sand, iron oxide staining, gypsum crystals</td>
<td></td>
</tr>
</tbody>
</table>
### TABULATED LOGS OF TEST PITS

<table>
<thead>
<tr>
<th>Location</th>
<th>Approximate Depth BGS (feet)</th>
<th>Description</th>
<th>Remarks</th>
</tr>
</thead>
</table>
| TP-20-4  | 0 to 4                       | Clayey Sand with Gravel (SC) – grayish brown to dark grayish brown, dry to moist, interbedded with firm to hard sandy lean clay with gravel, low to medium plasticity, fine to coarse grained sand, fine to coarse gravel, nails, geotextile fabric, caution tape (FILL) | - Performed on 9/24/2020  
- Approximate test pit coordinates: 37.88855, -121.58318  
- Estimated ground surface elevation = 94 feet (plant datum) |
|          | 4 to 6½                      | Sandy Lean Clay (CL) – dark grayish brown, moist, hard, medium plasticity, interbedded with yellowish brown, loose poorly graded sand with gravel, fine to medium grained sand, fine gravel (FILL) | - Test pit terminated at approximately 15 feet BGS  
- Free groundwater observed at approximately 14 feet BGS  
- Organic Content = 39% at approximately 6½ feet BGS |
|          | 6½ to 7½                     | Peat (PT) – very dark gray, moist | |
|          | 7½ to 15                     | Lean Clay with Sand (CL) – gray to yellowish brown, moist, firm, medium plasticity, fine grained sand, iron oxide staining | |

**Notes:**
- PP= Pocket Penetrometer in TSF (tons/square-foot)
- BGS = Below the existing ground surface
- Estimated ground surface elevations are based on the topographic data available on Sheet C-204 of the grading plans entitled *Discovery Bay Community Services District, Denitrification & Master Plan Upgrades Project, Plant No. 2, Paving & Grading – Plan*, dated March 2021 prepared by HERWIT Engineering (Job No. 2019-135 T01).
- Elevations are based on a plant datum of 100 feet that is equivalent to an actual elevation of 0 feet.
- LL = Liquid Limit
- PI = Plasticity Index
- Thicknesses and depths are approximate.
- Test pit locations were backfilled with the excavated soil cuttings.
- It should be noted that groundwater levels can fluctuate several feet depending on factors such as seasonal rainfall, groundwater withdrawal, and construction activities on this or adjacent properties. Also, the initial and (when applicable) final free groundwater levels shown in the current and previous exploration points may not be representative of stabilized groundwater levels.
APPENDIX B

CPT LOGS AND LIQUEFACTION ANALYSIS
Input parameters and analysis data

Analysis method: B&I (2014)
Fines correction method: B&I (2014)
Points to test: Based on Ic value
Earthquake magnitude $M_w$: 6.52
Peak ground acceleration: 0.54
Depth to water table (nsblu): 7.50 ft

Depth to GWT (erthq.): 5.00 ft
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Use fill: No
Fill height: N/A

Fill weight: N/A
Transition detect. applied: Yes
K_e applied: Yes
Clay behavior: Sands only
Limit depth applied: No
Limit depth: N/A

SBT legend
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

SBT legend
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained
Input parameters and analysis data

<table>
<thead>
<tr>
<th>Analysis method:</th>
<th>B&amp;I (2014)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fines correction method:</td>
<td>B&amp;I (2014)</td>
</tr>
<tr>
<td>Points to test:</td>
<td>Based on Ic value</td>
</tr>
<tr>
<td>Earthquake magnitude Mw:</td>
<td>6.52</td>
</tr>
<tr>
<td>Peak ground acceleration:</td>
<td>0.54</td>
</tr>
<tr>
<td>Depth to water table (in situ):</td>
<td>7.50 ft</td>
</tr>
</tbody>
</table>

Depth to GWT (erthq.): 5.00 ft
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Use fill: No
Fill height: N/A

Fill weight: N/A
Transition detect. applied: Yes
Ks applied: Yes
Clay like behavior applied: Sands only
Limit depth applied: No
Limit depth: N/A

F.S. color scheme
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme
- Very high risk
- High risk
- Low risk

Liquefaction analysis overall plot

Vertical settlements
Lateral displacements
APPENDIX C

LABORATORY TEST RESULTS
The information included in this graphic representation has been compiled from a variety of sources and is subject to change without notice. BSK makes no representations or warranties, express or implied, as to accuracy, completeness, timeliness, or rights to the use of such information. The document is not intended for use as a land survey product nor is it designed or intended as a construction design document. The use or misuse of the information contained on this graphic representation is at the sole risk of the party using or misusing the information.

**PROJECT NO.** G19-194-11L  
**DRAWN:** 11/14/19  
**DRAWN BY:** D. Tower  
**CHECKED BY:** C. Melo  
**FILE NAME:** Figures.indd

---

**ATTERBERG LIMITS**

**FIGURE** C-1

Denitrification Basin Project  
Wastewater Treatment Plant  
Discovery Bay, California
Unconsolidated-Undrained Triaxial Test
ASTM D2850

Sample Data

<table>
<thead>
<tr>
<th>Sample</th>
<th>Moisture %</th>
<th>Dry Den,pcf</th>
<th>Void Ratio</th>
<th>Saturation %</th>
<th>Height in</th>
<th>Diameter in</th>
<th>Cell psi</th>
<th>Strain %</th>
<th>Deviator, ksf</th>
<th>Rate %/min</th>
<th>in/min</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>22.2</td>
<td>104.8</td>
<td>0.609</td>
<td>98.4</td>
<td>5.00</td>
<td>2.41</td>
<td>3.5</td>
<td>15.00</td>
<td>4.445</td>
<td>1.00</td>
<td>0.050</td>
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<tr>
<td>2</td>
<td>27.5</td>
<td>96.4</td>
<td>0.748</td>
<td>99.1</td>
<td>5.01</td>
<td>2.41</td>
<td>3.5</td>
<td>15.00</td>
<td>4.509</td>
<td>1.00</td>
<td>0.050</td>
</tr>
<tr>
<td>3</td>
<td>20.3</td>
<td>108.6</td>
<td>0.553</td>
<td>99.1</td>
<td>5.01</td>
<td>2.42</td>
<td>4.9</td>
<td>15.00</td>
<td>2.194</td>
<td>1.00</td>
<td>0.050</td>
</tr>
<tr>
<td>4</td>
<td>25.2</td>
<td>100.1</td>
<td>0.684</td>
<td>99.6</td>
<td>4.99</td>
<td>2.42</td>
<td>7.1</td>
<td>15.00</td>
<td>1.545</td>
<td>1.00</td>
<td>0.050</td>
</tr>
</tbody>
</table>

Job No.: 664-294
Client: BSK Associates
Project: G19-194-11L
Boring: B-1 B-2 B-2 B-2
Sample: 1C 3C 5C 6C
Depth ft: 3 6 11 16

Sample #
1 Fat Clay with Sand (CH)
2 Lean Clay (CL)
3 Lean Clay with Sand (CL)
4 Lean Clay with Sand (CL)

Visual Soil Description

Fat Clay with Sand (CH)
Lean Clay (CL)
Lean Clay with Sand (CL)
Lean Clay with Sand (CL)

Remarks:
Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.
**R-Value Test**

**Caltrans Test Method 301**

<table>
<thead>
<tr>
<th>Project Name</th>
<th>Discovery Bay</th>
</tr>
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<tbody>
<tr>
<td>Project Number</td>
<td>G1919411L</td>
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<tr>
<td>Sample Source</td>
<td>B-2@0.5-5'</td>
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<tr>
<td>Lab Tracking ID</td>
<td>5249</td>
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<tr>
<td>Sample Location</td>
<td></td>
</tr>
<tr>
<td>Sample Date</td>
<td>11/5/2019</td>
</tr>
<tr>
<td>Sample By</td>
<td>OK</td>
</tr>
<tr>
<td>Test Date</td>
<td>11/7/2019</td>
</tr>
<tr>
<td>Report Date</td>
<td>11/8/2019</td>
</tr>
<tr>
<td>Tested By</td>
<td>RC</td>
</tr>
</tbody>
</table>

**Table**

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>A</th>
<th>B</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>EXUDATION PRESSURE, LOAD (lb)</td>
<td>6794</td>
<td>4381</td>
<td>2240</td>
</tr>
<tr>
<td>EXUDATION PRESSURE, PSI</td>
<td>541</td>
<td>349</td>
<td>178</td>
</tr>
<tr>
<td>EXPANSION, * 0.0001 IN</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>EXPANSION PRESSURE, PSF</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>STABILOMETER PH AT 2000 LBS</td>
<td>134</td>
<td>138</td>
<td>142</td>
</tr>
<tr>
<td>DISPLACEMENT</td>
<td>3.65</td>
<td>3.72</td>
<td>3.67</td>
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<tr>
<td>RESISTANCE VALUE <em>R</em></td>
<td>12</td>
<td>10</td>
<td>8</td>
</tr>
<tr>
<td><em>R</em> VALUE CORRECTED FOR HEIGHT</td>
<td>12</td>
<td>10</td>
<td>8</td>
</tr>
<tr>
<td>% MOISTURE AT TEST</td>
<td>18.9</td>
<td>20.9</td>
<td>22.7</td>
</tr>
<tr>
<td>DRY DENSITY AT TEST, PCF</td>
<td>103.0</td>
<td>102.8</td>
<td>102.0</td>
</tr>
<tr>
<td><em>R</em> VALUE AT 300 PSI</td>
<td>9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>EXUDATION PRESSURE</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><em>R</em> VALUE BY EXPANSION</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PRESSURE TI = 4.0, GF=1,50</td>
<td>N/A</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Diagram**

- **Graph 1**: Exudation Pressure, PSI vs. Cover Thickness by Expansion Pressure, Inches
- **Graph 2**: Cover Thickness by Stabilometer, Inches vs. Cover Thickness by Expansion Pressure, Inches

Sample Description: Fat CLAY with Sand (CH)

---

** remarked: G1919411L 5249**

**Reviewed By:** JKA

**Figure C-3**

Denitrification Basin Project
Wastewater Treatment Plant
Discovery Bay, California

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Moisture Density Relationship

Client: Town of Discovery Bay
1800 Willow Lake Rd
Discovery Bay, CA 94505

Project: G1919411L
Discovery Bay Denitrification Project
2305 Cherry Hill Drive
Discovery Bay, CA

Sample Date: 10/31/2019
Sample Number: 5949

Technician: Michael Romero
Sample Location: B-2/Bulk @ 0.5-5'
Lab: BSK Livermore

ASTM D1557

Laboratory Data

Method: B (ASTM D1557)
Preparation Method: Moist
Rammer Type: Mechanical Round
Specific Gravity: 2.66
Maximum Dry Density (pcf): 110.7
Optimum Moisture (%): 17.8

Soil Classification: Fat Clay with Sand (CH)

General

Test Notes: B-2/Bulk @ 0.5-5'
Test Completed By: Nicholas Shelly
Test Completed Date: 11/08/2019
Approved By: Randy Cortez
Approved Date: 11/11/2019

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BSK ASSOCIATES

PROJECT NO. G19-194-11L
DRAWN: 11/14/19
DRAWN BY: D. Tower
CHECKED BY: C. Melo
FILE NAME: Figures.indd

MOISTURE-DENSITY RELATIONSHIP

Denitrification Basin Project
Wastewater Treatment Plant
Discovery Bay, California

FIGURE C-4
**MATERIAL DESCRIPTION** | **LL** | **PL** | **PI** | **%<#40** | **%<#200** | **USCS**
---|---|---|---|---|---|---
Elastic SILT (MH) | 100 | 59 | 41 |  |  |  
Peat with Sand (PT) | 94 | 78 | 16 |  |  |  
Fat CLAY (CH) | 55 | 21 | 34 |  |  |  
Sandy Organic Silt (OH) | 86 | 65 | 21 |  |  |  
Peat (PT) | 94 | 75 | 19 |  |  |  

**Remarks:**

- Source: **B-20-1**
  - Sample No.: 1C
  - Elev./Depth: 3’

- Source: **B-20-2**
  - Sample No.: 2C
  - Elev./Depth: 6’

- Source: **B-20-2**
  - Sample No.: 3C
  - Elev./Depth: 9.5’

- Source: **TP-20-1**
  - Elev./Depth: 1.5’

- Source: **TP-20-3**
  - Elev./Depth: 6’

**LIQUID AND PLASTIC LIMITS TEST REPORT**

**COOPER TESTING LABORATORY**

**PROJECT NO. G19-194-11L**

**DRAWN:** 10/22/20

**DRAWN BY:** D. Tower

**CHECKED BY:** C. Melo

**FILE NAME:** Figures.indd

**ATTERBERG LIMITS**

Denitrification Basin Project
Wastewater Treatment Plant
Discovery Bay, California

**FIGURE**

C-5

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### Stress-Strain Curves

<table>
<thead>
<tr>
<th>Sample</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture %</td>
<td>29.0</td>
<td>27.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dry Den,pcf</td>
<td>94.4</td>
<td>96.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Void Ratio</td>
<td>0.786</td>
<td>0.755</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Saturation %</td>
<td>99.5</td>
<td>97.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Height in</td>
<td>5.02</td>
<td>4.99</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Diameter in</td>
<td>2.39</td>
<td>2.37</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cell psi</td>
<td>3.6</td>
<td>7.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strain %</td>
<td>15.00</td>
<td>15.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Deviator, ksf</td>
<td>1.376</td>
<td>2.501</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rate %/min</td>
<td>1.00</td>
<td>1.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>in/min</td>
<td>0.050</td>
<td>0.050</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Visual Soil Description

1. Elastic Silt (MH)
2. Fat Clay (CH)
3. 
4. 

Remarks:

Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.
13 November, 2019

Job No. 1911034
Cust. No. 12667

Mr. Michael Romero
BSK Associates Engineers & Laboratories
399 Lindbergh Avenue
Livermore, CA 94551

Subject: Project No.: G19-19411L
        Project Name: Discovery Bay
        Corrosivity Analysis – ASTM Test Methods

Dear Mr. Romero:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on November 06, 2019. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, both samples are classified as “severely corrosive”. All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations were 270 & 500 mg/kg and are determined to be sufficient to attack steel embedded in a concrete mortar coating. Chloride ion concentrations greater than 300 mg/kg are considered corrosive to embedded reinforcing steel; and, as such, the concrete mix design shall be adjusted accordingly by a qualified corrosion engineer.

The sulfate ion concentrations were 1200 & 1300 mg/kg and are determined to be sufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations. Therefore, concrete that comes into contact with this soil should use sulfate resistant cement such as Type II, in accordance with the California Building Code requirements with a maximum water-to-cement ratio of 0.50.

The pH of the soils were 4.96 & 7.97 which does present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures. Any soils with a pH of <6.0 is considered to be corrosive to buried iron, steel, mortar-coated steel and reinforced concrete structures. Therefore, corrosion prevention measures need to be considered for structures to be placed in this acidic soil.

The redox potentials were 36 & 49-mV and are indicative of potentially “severely corrosive” soils resulting from anaerobic soil conditions.

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

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

Very truly yours,

CERCO ANALYTICAL, INC.

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

JDH/jdl
Enclosure
Client: BSK Associates Engineers & Laboratories  
Client's Project No.: G19-194-11L  
Client's Project Name: Discovery Bay  
Date Sampled: 31-Oct-19  
Date Received: 9-Nov-19  
Matrix: Soil  
Authorization: Signed Chain of Custody

<table>
<thead>
<tr>
<th>Job/Sample No.</th>
<th>Sample I.D.</th>
<th>Redox (mV)</th>
<th>pH</th>
<th>Conductivity (1umhos/cm)*</th>
<th>Resistivity (100% Saturation) (ohms-cm)</th>
<th>Sulfide (mg/kg)*</th>
<th>Chloride (mg/kg)*</th>
<th>Sulfate (mg/kg)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>1911034-001</td>
<td>B-1/1B @ 2.5'</td>
<td>36</td>
<td>7.97</td>
<td>-</td>
<td>180</td>
<td>-</td>
<td>500</td>
<td>1300</td>
</tr>
<tr>
<td>1911034-002</td>
<td>B-2/1B @ 2.5'</td>
<td>49</td>
<td>4.96</td>
<td>-</td>
<td>320</td>
<td>-</td>
<td>270</td>
<td>1200</td>
</tr>
</tbody>
</table>

**Method:**  
ASTM D1498  
ASTM D1125M  
ASTM G57  
ASTM D4658M  
ASTM D4327

|-----------------|-------------|-------------|---|-------------|---|-------------|-------------|

*Results Reported on "As Received" Basis

Cheryl McMillen  
Laboratory Director

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

SUMMARY OF COMPACTION REQUIREMENTS
## SUMMARY OF COMPACTION REQUIREMENTS

<table>
<thead>
<tr>
<th>Area</th>
<th>Compaction Requirements(^1, 2, 3, 4, 5)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fill or backfill consisting of aggregate base</td>
<td>Compact entire fill/backfill to a minimum of 95 percent (%) compaction at near optimum moisture content.</td>
</tr>
<tr>
<td>Site Preparation and Placement of General Engineered Fill</td>
<td>Compact upper 12 inches of original grade(^7) to a minimum of 95% compaction at near optimum moisture content for granular soils and to a minimum of 93% compaction at least 2% over optimum moisture content for clayey soils. Compact fill above original grade to a minimum of 90% compaction at near optimum moisture content for granular soils(^6) and to a minimum of 90% compaction at least 2% over optimum moisture content for clayey soils.</td>
</tr>
<tr>
<td>Overexcavation of Foundations for At Grade Ancillary Structures</td>
<td>Compact backfill for excavations resulting from the removal of peat or soils containing more than 3 percent organic content by dry unit weight below at grade ancillary structures to a minimum of 95% compaction at near optimum moisture content for granular soils and to a minimum of 93% compaction at least 2% over optimum moisture content for clayey soils.</td>
</tr>
<tr>
<td>Trenches and Excavations</td>
<td>Compact trench fine grading and pipe bedding to a minimum of 95% and 90% compaction, respectively, at near optimum moisture content. Compact trench backfill above pipe bedding to a minimum of 90% compaction at near optimum moisture content for granular soils(^6) and to a minimum of 90% compaction at a minimum of 2% over optimum moisture content for clayey soils. Compact upper 24 inches of trench backfill to a minimum of 95% compaction at near optimum moisture content for granular soils and a minimum of 93% compaction at a minimum of 2% over optimum moisture content for clayey soils.</td>
</tr>
<tr>
<td>Exterior Flatwork</td>
<td>Compact upper 12 inches of subgrade(^8) to a minimum of 95% compaction at near optimum moisture content for granular soils and to a minimum of 93% compaction at a minimum of 2% over optimum moisture content for clayey soils. Compact aggregate base to a minimum of 95% compaction at near optimum moisture content.</td>
</tr>
<tr>
<td>Pavements</td>
<td>Compact upper 12 inches of subgrade to a minimum of 95% compaction at near optimum moisture content for granular soils and to a minimum of 93% compaction at a minimum of 2% over optimum moisture content for clayey soils.</td>
</tr>
</tbody>
</table>

**Notes:**

1. Depths are below original grade or finished subgrade elevation as indicated.
2. All compaction requirements refer to relative compaction as a percentage of the laboratory standard described by ASTM D1557 (latest edition).
3. Fill material should be compacted in lifts not exceeding 8 inches in loose thickness.
4. All subgrades should be firm and stable.
5. Where fills are greater than 7 feet in depth below finish grade, the zone below a depth of 7 feet should be compacted to a minimum of 95% compaction.
6. If granular soil consists of aggregate base, compact to a minimum of 95% compaction.
7. **Original grade:** ground elevation prior to grading.
8. **Subgrade:** Final soil elevation prior to placement of select fill, such as aggregate base course and trench fine grading, asphalt concrete, or concrete for foundations or other improvements.
APPENDIX E

SUBSURFACE DATA FROM PREVIOUS INVESTIGATIONS
<table>
<thead>
<tr>
<th>Depth in Feet</th>
<th>Dry Density lb/ft³</th>
<th>Moisture Content %</th>
<th>Blow/Fl</th>
<th>Sample No.</th>
<th>TP-28 USCS</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>OL</td>
<td>Black Organic Silty Clay with Peat</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>CL</td>
<td>Mottled Grey-Brown Silty Clay</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>CL</td>
<td>Color to Greyish-Brown Silty Clay</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>TEST PIT TERMINATED AT 6-FOOT DEPTH</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>NOTES:</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1. Test Pit Excavated on 12-12-85.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2. No Groundwater Encountered.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3. No Caving Noted.</td>
</tr>
<tr>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>PT/OL</td>
<td>Black Organic Silty Clay/Peat</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>OL</td>
<td>Dark Grey Organic Silty Clay</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>CL</td>
<td>Grey Silty Clay with Rust Stains</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>TEST PIT TERMINATED AT 6-FOOT DEPTH</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>NOTES:</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1. Test Pit Excavated on 12-12-85.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2. No Groundwater Encountered.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3. No Caving Noted.</td>
</tr>
<tr>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>TP-30</td>
<td>Fill Black Organic Silty Clay</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>OL</td>
<td>Black Organic Clay &amp; Peat</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>CL</td>
<td>Grey-Rust Silty Clay</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>TEST PIT TERMINATED AT 4-FOOT DEPTH</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>NOTES:</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1. Test Pit Excavated on 12-12-85.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2. Test Pit Flooded During Excavation.</td>
</tr>
<tr>
<td>Depth (feet)</td>
<td>Sample Type</td>
<td>Sample No.</td>
<td>Blow Count</td>
<td>Density (pcf)</td>
<td>Moisture Content (%)</td>
<td>Liquid Limit</td>
</tr>
<tr>
<td>-------------</td>
<td>-------------</td>
<td>------------</td>
<td>-------------</td>
<td>---------------</td>
<td>---------------------</td>
<td>--------------</td>
</tr>
<tr>
<td>1.5</td>
<td></td>
<td></td>
<td>1.75</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td>4.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td>1.5</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Test Pit completed at a depth of approximately 6.5 feet below existing site grade.
**Surface Conditions:** Scattered weeds

**Groundwater:** No free groundwater encountered.

**Date Completed:** 8/27/2003

**Logged By:** RJIO

**Total Depth:** 15 feet

<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>Sample Type</th>
<th>Sample No.</th>
<th>Brossift</th>
<th>Pan (1st)</th>
<th>Dry Density (pcf)</th>
<th>Moisture Content (%)</th>
<th>Liquid Limit</th>
<th>Plasticity Index</th>
<th>Passing #4 Sieve (%)</th>
<th>Passing #200 Sieve (%)</th>
<th>Other Tests</th>
<th>Lithography</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.5-5.5-1</td>
<td></td>
<td></td>
<td></td>
<td>3.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>iCL) SILTY CLAY - Dark brown, moist, very stiff, moderate plasticity, scattered roots</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td>2.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Gray-brown</td>
</tr>
</tbody>
</table>

**Organic content = 3.7%**

Test Pit completed at a depth of approximately 6.5 feet below existing site grade.
<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>Sample Type</th>
<th>Sample No.</th>
<th>Blowsift</th>
<th>Pen (tf)</th>
<th>Dry Density (pcf)</th>
<th>Moisture Content (%)</th>
<th>Liquid Limit</th>
<th>Plasticity Index</th>
<th>Passing #4 Sieve (%)</th>
<th>Passing #200 Sieve (%)</th>
<th>Other Tests</th>
<th>Lithography</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>4.25</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>(CL) SILTY CLAY - Brown, moist, hard, moderate plasticity</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>&gt;4.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Light grey-brown</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Gray-brown, stiff</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Brown</td>
</tr>
</tbody>
</table>
| Test Pit completed at a depth of approximately 7 feet below existing site grade.
## FIELD EXPLORATION

<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>Graphical Log</th>
<th>Sample Number</th>
<th>Sample Type</th>
<th>Sample Length (In.)</th>
<th>Indurated</th>
<th>Core Recovery</th>
<th>USCS Symbol</th>
<th>Water Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Particle Size (No. 200)</th>
<th>Particle Size (No. 4)</th>
<th>Plasticity Index</th>
<th>Liquid Limit</th>
<th>Plasticity Index (NP No. Plasticity)</th>
<th>Other Tests/Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>40-</td>
<td></td>
<td>1-35-1</td>
<td>BC=3</td>
<td></td>
<td>4</td>
<td>7</td>
<td>CL</td>
<td>23.4</td>
<td>110</td>
<td>64</td>
<td>33</td>
<td>19</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Clayey SAND (SC): brown, wet, medium dense, 40% fines, 60% fine sand</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>40-</td>
<td></td>
<td>1-40-1</td>
<td>BC=2</td>
<td></td>
<td>6</td>
<td>1.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Sandy Lean Clay (CL): low plasticity fines, grayish brown, wet, firm, 85% fines, 15% fine sand</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>1-45-1</td>
<td>BC=3</td>
<td>4</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>Clayey SAND (SC): low plasticity fines, olive brown, wet, loose, 55% fines, 2% fine to medium sand</td>
<td></td>
<td></td>
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<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>50-</td>
<td></td>
<td>1-50-1</td>
<td>BC=6</td>
<td></td>
<td>7</td>
<td>9</td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Sandy Lean Clay (CL): low plasticity fines, yellowish brown, wet, loose, 75% fines, 25% fine sand</td>
<td></td>
<td></td>
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<td></td>
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<td></td>
</tr>
</tbody>
</table>

The exploration was terminated at approximately 51.5 ft. below ground surface. The exploration was backfilled with bentonite grout on December 13, 2012.

## LABORATORY RESULTS

<table>
<thead>
<tr>
<th>Recovery</th>
<th>USCS Symbol</th>
<th>Water Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Particle Size (No. 200)</th>
<th>Particle Size (No. 4)</th>
<th>Plasticity Index</th>
<th>Liquid Limit</th>
<th>Plasticity Index (NP No. Plasticity)</th>
<th>Other Tests/Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>22.7</td>
<td>104</td>
<td>36</td>
<td>22</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

## GROUNDWATER LEVEL INFORMATION:
Due to rotary wash method being used, groundwater was not measured during drilling or after completion.

## GENERAL NOTES:
**FIELD EXPLORATION**

<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>Graphical Log</th>
<th>Sample Number</th>
<th>Sample Type</th>
<th>USCS Symbol</th>
<th>Water Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Other Tests/Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-2-1</td>
<td>2-2-1</td>
<td>BC=5</td>
<td>98</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2-5-1</td>
<td>2-5-1</td>
<td>BC=3</td>
<td>58</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2-10-1</td>
<td>2-10-1</td>
<td>BC=1</td>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2-15-1</td>
<td>2-15-1</td>
<td>BC=2</td>
<td>8</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2-20-1</td>
<td>2-20-1</td>
<td>BC=2</td>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**LABORATORY RESULTS**

<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>Graphical Log</th>
<th>Sample Number</th>
<th>Sample Type</th>
<th>USCS Symbol</th>
<th>Water Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Other Tests/Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-2-1</td>
<td>2-2-1</td>
<td>BC=5</td>
<td>98</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2-5-1</td>
<td>2-5-1</td>
<td>BC=3</td>
<td>58</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2-10-1</td>
<td>2-10-1</td>
<td>BC=1</td>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2-15-1</td>
<td>2-15-1</td>
<td>BC=2</td>
<td>8</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2-20-1</td>
<td>2-20-1</td>
<td>BC=2</td>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The exploration was terminated at approximately 21.5 ft. below ground surface. The exploration was backfilled with bentonite grout on December 13, 2012.

**GROUNDWATER LEVEL INFORMATION:**

Groundwater was observed at approximately 4.3 ft. below ground surface during drilling.

**GENERAL NOTES:**
**FIELD EXPLORATION**

<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>Graphical Log</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>Sandy organo Clay (OL): non-plastic fines, black, moist, 60% fines, 40% fine sand</td>
</tr>
<tr>
<td>10</td>
<td>Fat Clay (CH): high plasticity fines, gray, moist, firm, 60% fines, 10% sand, with rust stains</td>
</tr>
<tr>
<td>15</td>
<td>Sandy Lean Clay (CL): medium plasticity fines, yellowish brown, moist, firm, 70% fines, 30% fine sands</td>
</tr>
<tr>
<td>20</td>
<td>Soft, 60% fines, 40% sand</td>
</tr>
</tbody>
</table>

The exploration was terminated at approximately 21.5 ft. below ground surface. The exploration was backfilled with bentonite grout on December 13, 2012.

**LABORATORY RESULTS**

<table>
<thead>
<tr>
<th>Sample Number</th>
<th>Sample Type</th>
<th>Recovery (ml)</th>
<th>Wet Recovery (ml)</th>
<th>USCS Symbol</th>
<th>Water Content (%)</th>
<th>Gravel Density (pcf)</th>
<th>Fine-Gravel Recovery (%)</th>
<th>Plastic Index</th>
<th>Plasticity Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>BC=10</td>
<td>10</td>
<td>21</td>
<td>17</td>
<td></td>
<td>28.0</td>
<td>95</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BC=3</td>
<td>3</td>
<td>4</td>
<td>5</td>
<td></td>
<td></td>
<td></td>
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<td>25.1</td>
<td>108</td>
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<tr>
<td>BC=5</td>
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<td>4</td>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BC=3</td>
<td>3</td>
<td>4</td>
<td>5</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**GROUNDWATER LEVEL INFORMATION:**

Groundwater was observed at approximately 5.5 ft. below ground surface during drilling.

**GENERAL NOTES:**

- Groundwater was observed at approximately 5.5 ft. below ground surface during drilling.
GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during excavation or after completion.

GENERAL NOTES:
Sandy lean CLAY (CL): brown, loose, 1' to 2' of imported dry to moist fill, trash
Sandy lean CLAY (CL): low plasticity fines, brown, dry to moist, firm
Fat CLAY (CL-CH): medium to high plasticity fines, brown black, mottled, moist

The exploration was terminated at approximately 5 ft. below ground surface. The exploration was backfilled with auger cuttings on July 10, 2013.
Groundwater was not encountered during excavation or after completion.

**General Notes:**

- **Sandy lean CLAY (CL):** brown, dry, 1' imported loose fill, gravels, trash
- **Lean CLAY (CL):** medium plasticity fines, brown, moist, firm
- **PEAT (PT):** black, moist, soft
- **Sandy lean CLAY (CL):** low to medium plasticity fines, gray, moist, firm
- **Lean clay/ fat Clay (CL-CH):** medium plasticity fines, gray, moist, firm

The exploration was terminated at approximately 6 ft. below ground surface. The exploration was backfilled with auger cuttings on July 10, 2013.

**Groundwater Level Information:**

Groundwater was not encountered during excavation or after completion.

**General Notes:**
**Groundwater Level Information:**
Groundwater was not encountered during excavation or after completion.

**General Notes:**
Sandy lean CLAY (CL): 0' - 1' imported loose fill, gravel, trash

<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>Field Exploration</th>
<th>Laboratory Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 1</td>
<td>Sandy lean CLAY (CL): fine to coarse grained, brown, dry to moist</td>
<td>Sample Type</td>
</tr>
<tr>
<td>1 - 5</td>
<td>PEAT (PT): low plasticity fines, black, moist, soft</td>
<td>Water Content (%)</td>
</tr>
<tr>
<td>5 - 10</td>
<td>Sandy lean CLAY (CL): low plasticity fines, gray, moist, soft, iron-oxide staining</td>
<td>Passing No.4 Sieve (%)</td>
</tr>
</tbody>
</table>

The exploration was terminated at approximately 5.5 ft. below ground surface. The exploration was backfilled with auger cuttings on July 10, 2013.

**Groundwater Level Information:**
Groundwater was not encountered during excavation or after completion.

**General Notes:**
- Sandy lean CLAY (CL): 0' - 1' imported loose fill, gravel, trash
- Sandy lean CLAY (CL): fine to coarse grained, brown, dry to moist
- PEAT (PT): low plasticity fines, black, moist, soft
- Sandy lean CLAY (CL): low plasticity fines, gray, moist, soft, iron-oxide staining

**Laboratory Results:**
- Depth (feet): 0 - 10
- Sample Type: USCS Symbol
- Water Content (%)
- Dry Density (pcf)
- Passing No.4 Sieve (%)
- Passing #200 Sieve (%)
- Liquid Limit (NV=No Value)
- Plasticity Index (NP=No Plasticity)
GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during excavation or after completion.

GENERAL NOTES:
- brown, Imported loose fill with trash and cobbles
- PEAT (PT): low plasticity fines, black, moist, very soft
- Lean CLAY (CL): gray with reddish strik (mottled), moist, soft
- Lean clay to fat Clay (CL-CH): medium plasticity fines, brown, moist

The exploration was terminated at approximately 6.5 ft. below ground surface. The exploration was backfilled with auger cuttings on July 10, 2013.

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

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

**GENERAL NOTES:**
- Sandy SILT (ML): brown, dry to moist, imported fill to 2.5', cobbles, trash
- PEAT (PT): low plasticity fines, black, moist, very soft
- Lean CLAY (CL): gray with red (mottled), soft
- Fat CLAY (CH): high plasticity fines, brown, firm

The exploration was terminated at approximately 7 ft. below ground surface. The exploration was backfilled with auger cuttings on July 10, 2013.

The exploration was backfilled with auger cuttings on July 10, 2013.

**TEST PIT LOG TP-5**

<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>Sample Type</th>
<th>USCS Symbol</th>
<th>Water Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Passing No.4 Sieve (%)</th>
<th>Passing #200 Sieve (%)</th>
<th>Liquid Limit (NV=No Value)</th>
<th>Plasticity Index (NP=No Plasticity)</th>
<th>Other Tests/Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>Sandy SILT (ML)</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>Lean CLAY (CL)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>Fat CLAY (CH)</td>
<td></td>
<td></td>
<td></td>
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<td></td>
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<td></td>
</tr>
</tbody>
</table>

**FIELD EXPLORATION**

- No Coordinates Available
- No Elevation Available
- Surface Condition: Silty Sand

**LABORATORY RESULTS**

- 31.8

**DISCOVERY BAY WEST**
**MASTER PLAN IMPROVEMENTS**
**WASTE WATER TREATMENT PLANT**
**DISCOVERY BAY, CALIFORNIA**

---

**KLEINFELDER**
**Bright People. Right Solutions.**

**PROJECT NO.: 126545.M01**
**DRAWN BY:** G.G.
**CHECKED BY:** J.J.
**DATE:** 7/10/2013
**REVISED:**
GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during excavation or after completion.

GENERAL NOTES:
brown, dry, 12" imported fill
Sandy lean CLAY (CL): low plasticity fines, brown, moist, very soft
PEAT (PT): low plasticity fines, black, moist, very soft
Lean CLAY (CL): medium plasticity fines, gray black, moist, soft, reddish stroke
Lean CLAY (CL): medium plasticity fines, brown, moist, firm

The exploration was terminated at approximately 7 ft. below ground surface. The exploration was backfilled with auger cuttings on July 10, 2013.

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

GENERAL NOTES:
5 RESULTS

5.1 GROUNDWATER DEPTH AND ELEVATION

The depth to water level measurements during the monitoring event were converted to elevations in reference to a surveyed datum to calculate a groundwater flow direction and gradient for the site. The following table outlines the depth to water and groundwater elevations in the monitoring wells on January 10, 2006.

**Text Table 3**

<table>
<thead>
<tr>
<th>Location</th>
<th>TOC Elevation msl (feet)</th>
<th>Depth to GW BTOC (feet)</th>
<th>GW Elevation msl (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MW-1</td>
<td>-12.73</td>
<td>3.07</td>
<td>-15.80</td>
</tr>
<tr>
<td>MW-2</td>
<td>-10.85</td>
<td>3.64</td>
<td>-14.49</td>
</tr>
<tr>
<td>MW-3</td>
<td>-8.54</td>
<td>5.74</td>
<td>-14.28</td>
</tr>
</tbody>
</table>

TOC=Top of Well Casing
BTOC=Below Top of Casing
GW=Groundwater
msl = mean sea level

The depths to groundwater and corresponding groundwater elevations from the Second Quarter 2005 through First Quarter 2006 are included in Table 1 in the Appendices of this report.

The depth to groundwater at the site ranged from 3.07 bgs feet to 5.74 feet bgs. The groundwater elevations ranged from -15.80 feet MSL to -14.28 feet MSL. The groundwater gradient was approximately 0.003 feet/foot towards the southeast. Please see Plate 3 for the January 10, 2006 groundwater gradient and elevation contour map.

5.2 GROUNDWATER SAMPLING RESULTS

Total Dissolved Solids (TDS) was detected in each of the three monitoring wells, during the First Quarter 2006 sampling event. Nitrate as N and Total Coliform Organisms were each detected in a single monitoring well during the event.

The following table displays the detected concentrations of the analytes included in this assessment.
TABLE NO. 1
DEPTH TO GROUNDWATER AND GROUNDWATER ELEVATION
DISCOVERY BAY WASTEWATER TREATMENT PLANT NO. 2
DISCOVERY BAY, CALIFORNIA

<table>
<thead>
<tr>
<th>Well#</th>
<th>Date</th>
<th>TOC Elevation msl (feet)</th>
<th>DEPTH TO GROUNDWATER BTOC (feet)</th>
<th>GROUNDWATER ELEVATION msl (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MW-1</td>
<td>4/28/2005</td>
<td>-12.73</td>
<td>3.75</td>
<td>-16.48</td>
</tr>
<tr>
<td></td>
<td>7/26/2005</td>
<td>-12.73</td>
<td>5.49</td>
<td>-18.22</td>
</tr>
<tr>
<td></td>
<td>10/25/2005</td>
<td>-12.73</td>
<td>5.57</td>
<td>-18.30</td>
</tr>
<tr>
<td></td>
<td>1/10/2006</td>
<td>-12.73</td>
<td>3.07</td>
<td>-15.80</td>
</tr>
<tr>
<td></td>
<td>7/6/2005</td>
<td>-10.85</td>
<td>4.93</td>
<td>-16.78</td>
</tr>
<tr>
<td></td>
<td>10/25/2005</td>
<td>-10.85</td>
<td>5.65</td>
<td>-16.40</td>
</tr>
<tr>
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<td>1/10/2006</td>
<td>-10.85</td>
<td>3.64</td>
<td>-14.49</td>
</tr>
<tr>
<td>MW-3</td>
<td>4/28/2005</td>
<td>-8.64</td>
<td>7.47</td>
<td>-5.9</td>
</tr>
<tr>
<td></td>
<td>7/26/2005</td>
<td>-8.64</td>
<td>8.74</td>
<td>-17.28</td>
</tr>
<tr>
<td></td>
<td>10/25/2005</td>
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<td>9.2</td>
<td>-17.74</td>
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<tr>
<td></td>
<td>1/10/2006</td>
<td>-8.64</td>
<td>5.74</td>
<td>-14.28</td>
</tr>
</tbody>
</table>

Notes:
TOC = Top of Casing
BTOC = Below Top of Casing
msl = Mean Sea Level