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Geotechnical Investigation

Russell City Energy Center

Hayward, California

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GEOTECHNICAL INVESTIGATION

RUSSELL CITY ENERGY CENTER

HAYWARD, CALIFORNIA

1.0 INTRODUCTION

In this report we present the results of our geotechnical investigation for the Russell City Energy Center power plant, to be located in Hayward, California. The location of the site is shown on the Vicinity Map, Figure 1. The purpose of our investigation was to evaluate the subsurface conditions at the site and to provide geotechnical recommendations for design of the power plant.

1.1 Project Description

We understand that Calpine is planning to construct an approximately 600 Megawatt natural gas power plant on an approximately 20-acre parcel located between Depot Road and Enterprise Avenue in Hayward, California. The power plant construction will include construction of Steam Turbine/Generators (STG), Combustion Turbine/Generator (CTG), Heat Recovery Steam Generator (HRSG), Cooling Tower and Basin, chlorine contact basin, transformers, and several water tanks up to approximately 40 feet high, with diameters ranging from about 28 to 60 feet. Additionally there are 16-diameter, 81-foot high brine concentrator tanks and miscellaneous one-story pre-engineered buildings planned for the project. Additionally, a storm water detention pond may be planned at the site. We were provided preliminary structural loads and tolerable settlements for the project, which are discussed in the "Foundation" section of this report. Additionally, we understand that visual berms may be constructed at the site.

1.2 Scope of Services

The scope of our services was defined in Section 02100, of our October 16, 2006 Continuing Services Agreement with Russell City Energy Center, LLC.

- Our field work included drilling, sampling and logging of subsurface soils in 10 hollow-stem borings of depths of about 10 to 20 feet, 6 rotary-wash borings, each to a depth of approximately 160 feet, and 4 rotary-wash borings each to a depth of approximately 100 feet. Additionally, we performed 4 shallow test pits to depths of about 5 to 12 feet. Six CPTs were performed each to a depth of 100 feet. Two of the CPTs included seismic measurements for development of the shearwave velocity profile and seismic response analysis. One additional hollow-stem boring was performed later to help collect additional samples for CPT correlation and liquefaction assessment to a depth of approximately 30 feet. Additionally, 12 electrical resistivity measurements were performed at the site. The approximate locations of our borings, CPTs, test pits and electrical resistivity measurements are shown on the site plan, Figure 2 of this report.

- Evaluation of the physical and engineering properties of the subsurface soils by visually classifying the samples and performing various laboratory tests on

selected samples. Laboratory testing included consolidation, grain size, moisture content, density, shear strength, liquid limit, plastic limit, corrosivity, and other testing. Results of our laboratory tests are included in Appendix B.

- Engineering analysis to evaluate site earthwork, liquefaction potential, building and structure foundations, slabs-on-grade, retaining walls, seismic site response and pavements.

- Preparation of this report to summarize our findings and to present our conclusions and recommendations.

Environmental services were not included as part of this study.

2.0 SITE CONDITIONS

2.1 Exploration Program

Our exploration included a combination of hollow-stem (H) borings, rotary-wash (RW) borings, cone penetration tests (CPTs), and test pits (TP). The table below summarizes our explorations, and the date explored. Borings were visually logged in the field, and samples were collected for various laboratory testing. Details regarding our field and laboratory programs are discussed in Appendix A and Appendix B of this report, respectively. The approximate locations of our explorations are shown on the attached site plan, Figure 2.

Table 1. Summary of Explorations

Exploration	Approximate Northing[1]	Approximate Easting	Date Performed	Approximate Surface Elevation (ft)	Approximate Depth of Exploration (ft)
RW-1	2056745	6088450	10/30-31/06	6.1	160
RW-2	2057710	6088695	11/1-2/06	7.0	160
RW-3	2057445	6088565	10/26-27/06	4.4	160
RW-4	2057485	6088685	11/3-6/06	5.4	160
RW-5	2057730	6088570	11/7-8/06	7.2	160
RW-6	2057730	6088855	11/9-10/06	9.0	160
RW-7	2057692	6088290	11/13/06	7.0	100
RW-8	2058060	6088300	11/14/06	9.5	100
RW-9	2058215	6088045	11/15/06	9.0	100
RW-10	2058030	6088705	11/16/06	10.0	100
H-1	2058245	6088435	11/9/06	8.5	15
H-2	2058110	6088215	11/9/06	8.2	20
H-3	2057910	6088315	11/9/06	8.7	15
H-4	2058015	6088520	11/9/06	8.7	15
H-5	2057920	6088500	11/9/06	8.3	15
H-6	2057920	6088680	11/9/06	9.7	15
H-7	2057765	6088725	11/9/06	8.1	15
H-8	2057490	6088855	11/9/06	8.7	20
H-9	2057490	6088430	11/9/06	4.0	10
H-10	2057825	6088322	11/9/06	8.8	15

[1] State Plane Coordinate NAD 83 California Zone 3

Table 1. Summary of Explorations, continued

Exploration	Approximate Northing[2]	Approximate Easting	Date Performed	Approximate Surface Elevation (ft)	Approximate Depth of Exploration (ft)
H-11	2057635	6088565	12/18/06	5.8	30
CPT-1[3]	2057710	6088450	10/31/06	7.1	100
CPT-2	2057635	6088685	10/31/06	7.0	100
CPT-3	2057928	6088178	10/31/06	8.0	100
CPT-4	2057445	6088685	10/30/06	5.4	100
CPT-5	2057635	6088565	10/30/06	5.7	100
CPT-6	2058245	6088560	10/30/06	8.8	100
TP-1	2057450	6088810	11/17/06	10	12
TP-2	2057700	6068850	11/17/06	9½	9
TP-3	2057710	6088360	11/17/06	7½	12
TP-4	2057440	6088415	11/17/06	4½	7

2.2 Surface

The approximately 20-acre site is relatively level, with surface grades ranging from about Elevation 3 feet to approximately 11 feet. The proposed power plant extends on to 3 parcels areas, known as the “City”, “Eash”, and “Alladin” parcels. Currently, the southern portion of the project site(the “City Parcel”) is part of the City of Hayward waste water treatment facility, and is currently used for sludge drying and handling. The northern portions of the project site are identified as the “Eash” and “Aladdin” parcels and are currently used as pallet and automobile junk yards, respectively. The site contains numerous structures, fences, different pavements, and vegetation associated with the waste water treatment plant, pallet yard, and automobile junk yard.

The project site is relatively level, and we understand that site grades will remain relatively unchanged. Some fills on the order of 5 to 6 feet are planned for the sludge handling area of the of the waste water treatment plant area to bring the site to about Elevation 9 to 10 feet. The general grade of sludge handling area of the waste water treatment plant portion of the project is several feet lower than the surrounding waste water treatment plant site grades and the adjacent Eash and Aladdin parcels to the north. During our exploration, approximately 2 to 4 feet of sludge was spread across the site for drying. Access areas were cut through the sludge at each exploration location.

2.3 Subsurface

Subsurface conditions at the site are consistent with alluvial soil conditions expected for this area. In general, our explorations encountered relatively shallow thickness (0 to 7 feet) of undocumented fill over generally clayey soils. The site is located on the fringes of the Bay Mud marine deposits, although Bay Mud was not clearly encountered in any of our explorations. Some thin layers of organic soil (less than about 1 foot) were observed in Test Pits 1, 2, and borings H-6 and H-8 within and

[2] State Plane Coordinate NAD 83 California Zone 3
[3] Shear wave velocity measurements were performed at CPT-1 and CPT-2 on November 13, 2006

below the fill. The organic materials were observed on the City parcel, but could occur at other site locations. These organic materials are likely associated with the previous natural site grades, and may occur at other locations across the site. Native soils below the fill generally consisted of clay soils with interbedded layers of sands, and silts. In general the clay and silt was generally stiff to very stiff, and the sands were generally medium to very dense.

The clayey soils in the upper 100 feet of the soil profile are generally considered moderately compressible, and slightly over consolidated. The consolidation test results and a summary of our consolidation test interpretations are included in Appendix B of this report.

2.4 Ground Water

Free ground water was measured in each of our 11 hollow-stem borings during drilling. Borings were backfilled with grout immediately after drilling, therefore the ground water depths may not represent stabilized ground water levels. Free ground water was not measured in the rotary-wash borings due to the use of drilling fluid. Table 2 summarizes our ground water observations.

Table 2. Depth to Ground Water

Boring*	Date Drilled	Elevation of Top of Boring (feet)	Depth to Ground Water (feet)	Elevation of Ground Water (feet)
H-1	11/9/2006	8.5	6.1	2.4
H-2	11/9/2006	8.2	8	0.2
H-3	11/9/2006	8.7	8.4	0.3
H-4	11/9/2006	8.7	8.8	-0.1
H-5	11/9/2006	8.3	8.6	-0.3
H-6	11/9/2006	9.7	9.8	-0.1
H-7	11/9/2006	8.1	9	-0.9
H-8	11/9/2006	8.7	15	-6.3
H-9	11/9/2006	4	5.3	-1.3
H-10	11/9/2006	8.8	10	-1.2
H-11	12/18/2006	5.8	8.2	-2.4

* Ground water levels were not determined in the rotary-wash borings due to the use of drilling fluid

As shown in the table above, based on the hollow-stem borings, ground water levels during the time of drilling were observed to be as high as approximately Elevation 2.4 feet. Maps published by the California Geological Survey (CGS) indicate that historically high ground water depths are around 5 feet at the project site. Based on a general ground Elevation of about 8 to 9 feet we recommend a design ground water at Elevation 4 feet be used for design. Fluctuations in the level of the ground water may occur due to variations in rainfall, underground drainage patterns, and other factors not evident at the time measurements were made.

3.0 GEOLOGIC HAZARDS

A brief qualitative evaluation of geologic hazards was made during this investigation. Our comments concerning these hazards are presented below.

3.1 Fault Rupture Hazard

A Regional Fault Map illustrating known active faults relative to the site is presented in Figure 3. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone. As shown on Figure 3, no known surface expression of active faults is believed to cross the site. Fault rupture through the site, therefore, is not anticipated.

3.2 Ground Shaking

Strong ground shaking can be expected at the site during moderate to severe earthquakes in the general region. This is common to virtually all developments in the San Francisco Bay Area. The "Seismicity" section that follows summarizes potential levels of ground shaking at the site.

3.3 Liquefaction

During strong seismic shaking, some soils may experience a phenomenon known as liquefaction, where the soil may undergo significant loss of strength, ground settlement, ground cracking, flowing ground, and/or lateral spreading due to increased soil pore water pressure as a result of cyclic shaking. It is generally accepted within the engineering community that loose to moderately dense, saturated, non-cohesive soils (sands and silts) are most susceptible to liquefaction. In general, soils with the highest liquefaction potential are generally found in loosely placed artificial fill and younger alluvial soils. The project site is overlain by significant thickness of alluvial deposits, and is located within an area zoned by the State of California as having potential for seismically-induced liquefaction hazards.

Liquefaction is a highly researched topic within the geotechnical community. Some recently published research indicates that some low plasticity clay soils may have undergone significant strength loss during the large 1999 Kocaeli earthquake in Turkey. Case studies of liquefaction after other large earthquakes have not indicated that clay liquefaction is a widespread phenomenon. Ongoing research indicates that the cyclic behavior of clayey soil is more complex than that of sand and silt, and depends on many factors including soil plasticity, shear strength, stress history, mineralogy, sensitivity, and other factors. Based on guidelines published by UC Davis that relate undrained shear strength to liquefaction susceptibility, the silty/clayey soils at the Russell City Energy site appear generally too stiff to liquefy. Some non-cohesive sand and silt layers have potential to liquefy under seismic shaking and are discussed in Appendix C. A more detailed discussion of our liquefaction analysis methods, including several figures and tables are included in Appendix C. Below, we have summarized the main results and conclusions regarding our liquefaction hazard evaluation for the site.

3.3.1 Summary of Results

In general, it appears that the site has a low to moderate liquefaction hazard risk. Several thin soil layers are potentially liquefiable as discussed below. We anticipate that after a large seismic event some ground settlement on the order of 1½ inches could occur, with up to about ¾-inch of differential settlement over a distance of about 30 feet could occur due to seismically induced liquefaction. For the proposed

structures it does not appear that the potentially liquefiable layers will cause lateral bearing failure, or ground rupture, provided the recommendations presented in this report are properly implemented. Details regarding foundation design are presented in the "Foundation" section of this report.

3.4 Differential Compaction

If near-surface soils vary in composition both vertically and laterally, strong earthquake shaking can cause non-uniform compaction of soil strata, resulting in movement of the near-surface soils. Because the subsurface soils encountered at the site are generally uniform and do not appear to change in thickness or consistency abruptly over short distances, we judge the probability of significant differential compaction at the site to be low.

Where undocumented fills or organic soils are encountered, over-excavation and recompaction of the fill should be performed as discussed in the "Earthwork and Foundation" sections of this report, to help prevent differential compaction.

3.5 Lateral Spreading

Lateral spreading typically occurs as a form of horizontal displacement of relatively flat-lying alluvial material toward an open or "free" face such as an open body of water, channel, or excavation. In soils this movement is generally due to failure along a weak plane, and may often be associated with liquefaction. Although there are several channel faces along the west side of the site, based on our explorations, it does not appear that there are any continuous liquefiable layers that are likely to cause lateral spreading to the open faces. Therefore, it is our opinion that the risk of lateral spreading affecting the proposed power plant structures is low.

4.0 SEISMICITY

4.1 Regional Active Faults

The San Francisco Bay Area is one of the most seismically active regions in the United States. The significant earthquakes that occur in the Bay Area are generally associated with crustal movement along well-defined, active fault zones of the San Andreas Fault system, which regionally trend in a northwesterly direction. The Hayward Fault, is located approximately 4 kilometers to the east of the project site. The San Andreas fault is located approximately 23 kilometers to the west of the project site. Both the Hayward and San Andreas faults are known to be active and capable of producing very large earthquakes with magnitudes greater than 7Mw.

4.2 Future Earthquake Probabilities

Although research on earthquake prediction has greatly increased in recent years, seismologists cannot predict when or where an earthquake will occur. The U.S. Geological Survey's Working Group on California Earthquake Probabilities (2002), referred to as WG02, determined there is a 62 percent chance of at least one magnitude 6.7 or greater earthquake striking the San Francisco Bay region between 2003 and 2032. This result is an important outcome of WG02's work, because any major earthquake can cause damage throughout the region.

This potential was demonstrated when the 1989 Loma Prieta earthquake caused severe damage in Oakland and San Francisco, more than 50 miles from the fault rupture. Although earthquakes can cause damage at a considerable distance, shaking will be very intense near the fault rupture. Therefore, earthquakes located in urbanized areas of the region have the potential to cause much more damage than the 1989 Loma Prieta earthquake.

4.3 Seismic Response Spectra

In accordance with our agreement, we performed a site specific seismic response analysis for the project, based on soil lithology and the soil shear wave velocity profile. The response analysis was performed using actual earthquake time histories recorded at sites similar distances from the faults. Details regarding our analysis, and the recommended spectral response for 5 percent damping for a 10 percent chance in 50 year seismic event are presented in the "Site Response Analysis" appendix of this report.

5.0 DYNAMIC SHEAR MODULUS

As discussed in the subsurface exploration section of this report, we performed shear wave velocity measurements at CPT-1 and CPT-2 to depths of approximately 100 feet. Figure 4 presents the recommended dynamic shear modulus for the soil profile for small strains (less than about 0.01 percent). If larger dynamic shear strains are anticipated, lower modulus values may be more appropriate.

6.0 THERMAL CONDUCTIVITY

Soil thermal conductivity is a function of mineralogy, dry density, gradation, water content, aging and temperature. Between elevations of about -8 to +8 feet there is little variation if the moisture content and dry density of the soil, even between fill and native soil (excluding any thin organic layers, which should be removed if encountered in shallow foundation areas). The average moisture content is about 23 percent and the average dry density is about 98 pounds per cubic foot. Based on conductivity charts published in Mitchell (1993), the thermal conductivity for a clayey soil that has a 100 pcf dry density and 23 percent moisture is about 0.85 BTU/hr/sq.ft./ft./°F. If design is sensitive to thermal conductivity parameters, additional exploration and laboratory testing could be considered, once electrical line locations have been determined, to refine the thermal conductivity estimates.

7.0 ELECTRICAL RESISTIVITY

As discussed, we retained a subcontractor to perform electrical resistivity measurements at twelve (12) locations at the site using the Wenner 4-point method. The resistivity measurements were performed at intervals outlined by RCEC. The results are included as Appendix E of this report.

8.0 CORROSION TESTING

Sixteen soil samples were submitted to an independent laboratory to test for corrosivity. Each sample was tested for electrical resistivity, pH, soluble sulfate and chloride content. The results of the testing are presented in Table 4, below.

Table 3. Results of Corrosivity Testing

Boring Number	Depth (feet)	Chloride (mg/kg)	Sulfate (mg/kg)	pH	Resistivity (ohm-cm)	Estimated Corrosivity Based on Resistivity	Estimated Corrosivity Based on Sulfates
RW-5	5.5	206	471	8.2	503	Very Severe	Negligible
RW-6	2.0	90	287	7.6	923	Very Severe	Negligible
RW-6	6.0	28	<5	7.7	1,727	Severe	Negligible
RW-7	14.0	102	<5	7.8	1,057	Severe	Negligible
R-10	9.0	34	212	7.7	665	Very Severe	Negligible
H-1	5.0	40	280	8.4	585	Very Severe	Negligible
H-2	3.0	137	295	8.0	682	Very Severe	Negligible
H-2	7.0	714	505	7.6	372	Very Severe	Negligible
H-5	4.0	53	<5	7.6	1,205	Severe	Negligible
H-6	3.5	116	<5	8.0	604	Very Severe	Negligible
RW-1	4.0	183	136	8.0	905	Very Severe	Negligible
RW-1	11.0	225	219	7.6	692	Very Severe	Negligible
RW-2	4.0	118	152	7.9	841	Very Severe	Negligible
RW-3	3.5	85	164	8.0	1,498	Severe	Negligible
RW-3	5.5	115	267	7.5	683	Very Severe	Negligible
RW-4	6.0	61	46	7.4	923	Very Severe	Negligible

Note: mg/kg = milligrams per kilogram

Many factors can affect the corrosion potential of soil including soil moisture content, resistivity, permeability and pH, as well as chloride and sulfate concentration. In general, soil resistivity, which is a measure of how easily electrical current flows through soils, is the most influential factor. Based on the findings of studies presented in ASTM STP 1013 titled "Effects of Soil Characteristics on Corrosion" (February, 1989), the approximate relationship between soil resistivity and soil corrosiveness was developed as shown in Table 4 below.

Table 4. Relationship Between Soil Resistivity and Soil Corrosivity

Soil Resistivity (ohm-cm)	Classification of Soil Corrosiveness
0 to 900	Very Severe Corrosion
900 to 2,300	Severely Corrosive
2,300 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
10,000 to >100,000	Very Mildly Corrosive

Chloride and sulfate ion concentrations, and pH appear to play secondary roles in affecting corrosion potential. High chloride levels tend to reduce soil resistivity and break down otherwise protective surface deposits, which can result in corrosion of buried metallic improvements or reinforced concrete structures. Sulfate ions in the soil can lower the soil resistivity and can be highly aggressive to portland cement concrete by combining chemically with certain constituents of the concrete, principally tricalcium aluminate. This reaction is accompanied by expansion and eventual disruption of the concrete matrix. High sulfate content could also potentially cause corrosion of the reinforcing steel in concrete. The 2001 California Building Code (CBC) Table No. 19-A-4 provides requirements for concrete exposed to sulfate-containing solutions as summarized in Table 5.

Table 5. Relationship Between Sulfate Concentration and Sulfate Exposure (CBC Table No. 19-A-4)

Water-Soluble Sulfate (SO_4) in soil, ppm	Sulfate Exposure
0 to 1,000	Negligible
1,000 to 2,000	Moderate[1]
2,000 to 20,000	Severe
over 20,000	Very Severe

[1] = seawater

Acidity is an important factor of soil corrosivity. The lower the pH (the more acidic the environment), the higher will the soil corrosivity with respect to buried metallic structures. As soil pH increases above 7 (the neutral value), the soil is increasingly more alkaline and less corrosive to buried steel structures due to protective surface films which form on steel in high pH environments. A pH between 5 and 8.5 is generally considered relatively passive from a corrosion standpoint.

As shown in Table 5, soil resistivity results range from 503 to 1,727 ohm-centimeters. In our opinion, based on the field resistivity results shown in Table 4 and the resistivity correlations presented in Table 5, it appears that the corrosion potential to buried metallic improvements may be characterized as very severe to severely corrosive. Based on our previous experience and Table No. 19-A-4 of the CBC, in our opinion, sulfate exposure to Portland Cement Concrete (PCC) may be considered negligible for the native subsurface materials sampled, and in general it appears that Type I cement is suitable based on soil-concrete compressibility. A Corrosion Engineer should be retained to design appropriate corrosion protection for the project.

9.0 CONCLUSIONS AND DEVELOPMENT CONSIDERATIONS

9.1 Conclusions

From a geotechnical engineering viewpoint the proposed power plant may be constructed as planned, provided design and construction is performed in accordance with the recommendations presented in this report.

The primary geotechnical concerns and potential mitigations are shown in Table 6 below.

Table 6. Primary Geotechnical Concerns and Potential Mitigation Options

Geotechnical Concerns	Potential Mitigation Options
Undocumented fill poses a risk of settlement to shallow foundations/pavements supported on undocumented fill	Undocumented fill should be over excavated and compacted in accordance with the recommendations presented in the "Earthwork" section of this report.
Strong ground shaking during earthquakes	Design structures using the site seismic response spectra and the seismic provisions of the California Building Code
Potential for some soil layers to liquefy	Design shallow foundations (mats and footings) for the estimated settlements discussed in the "Foundations" section of this report Piles should be designed for seismically induced downdrag and strength loss
Site filling in sludge handling area may cause some ground settlement	Piles should be designed for downdrag. Other site improvements should be designed for the estimated settlements
Heavy structural loads cause high ground settlements Some ground settlement should be anticipated adjacent to heavy structures supported on mats.	Where settlement estimates greatly exceed the tolerable settlement for a structure, the structure will need to be supported on deep foundations to support the structure within the tolerable settlement range. Details are discussed in the "Foundations" section of this report. Improvements such as buildings and utilities located near larger structures supported on mat foundations should be designed for estimated settlements.
Shallow ground water may cause construction difficulties during excavation near or below the ground water	Localized dewatering may be required for deeper excavations Unstable and wet excavation bottoms may require stabilization using geotextiles, aggregates, and/or chemical treatment.
Near surface native clays are highly expansive	Shallow foundations and pavements should be designed for expansive soil conditions as described in the foundations section of this report.

9.2 Final Geotechnical Investigation

The intent of this investigation is to provide enough information for RCEC to complete design of the project and prepare reasonable cost estimation. We understand that the project is a design-build contract. If the contractor requires additional information, additional field, lab, and engineering analysis may be required.

Many of the recommendations in the following sections require that the Geotechnical Engineer-of-Record provide observation and testing during construction to confirm that the geotechnical recommendations are implemented appropriately. Our recommendations presented in this report can only be relied upon if we are retained to perform all required geotechnical consultation and all geotechnical construction observation and testing as the Geotechnical Engineer-of-Record. If during the design-build process, we are not retained to provide any of the required supplemental consultation and construction observation and testing and another engineer is retained in our place, that engineer should review this report, accept or modify the recommendations as they feel necessary, and provide a letter to Russell City Energy Center, LLC acknowledging their acceptance of the role as Geotechnical Engineer-of-Record and presenting any of their recommendations.

Because subsurface conditions may vary from those predicted by the relatively small diameter borings and CPTs, and in order to confirm that our report recommendations and any other supplemental recommendations have been properly implemented, we recommend that the Geotechnical Engineer-of-Record be retained to 1) perform any required supplemental geotechnical investigation to update this report, as necessary, 2) review the final construction plans and specifications, and 3) observe the earthwork and foundation installation.

10.0 EARTHWORK

10.1 Clearing and Site Preparation

The site should be cleared of all surface and subsurface improvements to be removed and deleterious materials including existing building foundations, slabs, irrigation lines, fills, pavements, debris, designated trees, shrubs, and associated roots. Abandonment of existing buried utilities is discussed below. Excavations extending below the planned finished site grades should be cleaned and backfilled with suitable material compacted as recommended in the “Compaction” section of this report. We recommend that the backfilling be carried out under observation of the Geotechnical Engineer-of-Record.

After clearing, any vegetated areas should be stripped to sufficient depth to remove all surface vegetation and topsoil containing greater than 3 percent organic matter by weight. At the time of our field investigation, we estimated that a stripping depth of approximately 4 inches would be required in vegetated areas. The actual stripping depth required depends on site usage prior to construction and should be established in the field by the Geotechnical Engineer-of-Record at the time of construction. The stripped materials should be removed from the site or may be stockpiled for use in landscaped areas, if desired.

We recommend that backfilling of holes or pits resulting from demolition and removal of existing building foundations, buried structures or other improvements be carried out under observation by the Geotechnical Engineer-of-Record and that the backfill be observed and tested during placement. Alternatively, the loose backfill locations should be carefully documented during demolition for excavation and re-compaction during site grading. Additionally, our exploration test pits will need re-compaction during site construction. The location of the test pits are shown on the Site Plan, Figure 2.

10.2 Removal of Existing Fill and Organic Soil Layers

As discussed in the subsurface section off this report, undocumented fill was observed in many of our explorations. The following table summarizes the fill depth at each exploration location, and the estimate top native soil elevation.

Table 7. Summary of Undocumented Fill

Exploration Identification	Approximate Surface Elevation (ft)	Approximate Depth of Undocumented Fill (ft)	Approximate Elevation of Top of Native Soil (ft)
RW-1	6.1	0	6
RW-2	7.0	2	5
RW-3	4.4	4½	0
RW-4	5.4	5½	0
RW-5	7.2	2	5
RW-6	9.0	3	6
RW-7	7.0	3	4
RW-8	9.5	7	2½
RW-9	9.0	3½	5½
RW-10	10.0	2	8
H-1	8.5	1½	7
H-2	8.2	4	4
H-3	8.7	4½	4
H-4	8.7	1	7½
H-5	8.3	2	6½
H-6	9.7	2½	7
H-7	8.1	3	5
H-8	8.7	7	1½
H-9	4.0	1½	2½
H-10	8.8	2	7
H-11	5.8	2½	3½
TP-1	10	7	3
TP-2	9½	3½	6
TP-3	7½	1½	6
TP-4	4½	2	2½

As shown above, undocumented fill was encountered across the site with thickness ranging from 0 to 7 feet. In addition, a thin organic soil (around 1 foot) layer was observed in boring H-8, and in Test Pits 1 and 2. In addition, some organic soil was observed in the fill at boring H-6. Where shallow foundations are planned to have bearing elevations above the top of native elevation, we recommend that the fill be removed and replaced with engineered fill. If organic soils are observed below the fill, they should also be removed and replaced with engineered fill. Side slopes of fill

excavations in building and pavement areas should be sloped at inclinations no greater than 3:1 (horizontal to vertical) to minimize abrupt variations in fill thickness.

If the existing fill material meets the requirements in the "Material for Fill" section below, it may be reused as engineered fill. We anticipate that most of the existing undocumented fill can be replaced as engineered fill, from a geotechnical viewpoint. Any organic soils should be separated for disposal or use in landscape areas. Organic soils should not be reused as engineered fill. Where structures are to be supported on deep foundations, the fill does not need to be removed and replaced, provided structure slabs are designed to span between deep foundation supports. Where new pavements are planned over existing fill, we recommend that the fill be removed to a depth at least 4 feet below the top of the pavement or to the bottom of the fill, whichever is shallower.

Deep fills such as may be encountered where existing underground tanks or other subsurface structures are located should be completely removed and replaced with engineered fill with structure areas. In pavement sections, the Geotechnical Engineer-of-Record should review the condition on a case by case basis.

All fill should be compacted in accordance with the recommendations for fill presented in the "Compaction" section of this report.

10.3 Abandoned Utilities

Abandoned utilities within the proposed structure areas should be removed in their entirety. Utilities outside the structure areas should be removed or abandoned in-place by grouting or plugging the ends with concrete. Fills associated with utilities abandoned in-place could pose some risk of settlement; utilities that are plugged could also pose some risk of future collapse or erosion should they leak or become damaged. The potential risks are relatively low for small diameter pipes (4 inches or less) abandoned in-place and increasingly higher with increasing diameter. The Geotechnical Engineer-of-Record should evaluate the potential risks associated with abandonment of utilities outside structural areas on a case-by-case basis.

10.4 Subgrade Preparation

After the site has been properly cleared, stripped, and necessary excavations have been made, exposed surface soils in those areas to receive fill, foundations, slabs-on-grade, or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the recommendations for fill presented in the "Compaction" section. The finished compacted subgrade should be firm and non-yielding (less than ½ inch of deflection and no rutting) under the weight of compaction equipment.

Excavations that extend within about 2 feet of the ground water level are likely to be wet and may become soft under construction equipment loads. Subgrade stabilization will likely be required. Stabilization methods will depend on the field conditions during construction, but could require 18-inches of ¾-inch clean crushed aggregate placed over stabilization fabric such as Mirafi 600X or equivalent, or chemical treatment.

Excavations that extend below ground water will require dewatering prior to placement of fill or construction of new foundations.

10.5 Material for Fill

All on-site soils below the stripped layer having an organic content of less than 3 percent by weight are suitable for use as fill at the site. In general, fill material should not contain rocks or lumps larger than 6 inches in greatest dimension, with no more than 15 percent by weight larger than 2½ inches. Imported and non-expansive fill material should be inorganic and should have a Plasticity Index of 15 or less. Proposed imported fill should be approved by the Geotechnical Engineer-of-Record at least four days prior to delivery to the site. Compliance testing for aggregate base may take up to 10 days to complete.

Consideration should also be given to the environmental characteristics as well as the corrosion potential of imported fill. Laboratory testing, including pH, soluble sulfates, chlorides, and resistivity will provide information regarding corrosion potential. Import soils should not be more corrosive than the native materials.

Imported fill should also be approved by the environmental consultant for the project prior to importing the soil to the site.

10.6 Reuse of On-site Recycled Materials

If desired to reuse asphalt or portland cement as engineered fill, we recommend that it be ground up and thoroughly mixed with on-site or import soil. In general, recycled asphalt or concrete should be ground down to less than 4 inches in greatest dimension, with no more than 25 percent larger than 2½ inches. Recycled material should be thoroughly mixed with a sufficient amount of soil, such that there is no more than 40 percent by weight of recycled material in the final mix.

We recommend that fill containing recycled asphalt and concrete be placed near the bottom of the proposed fills and/or spread out evenly across the site. Recycled fill should not be used within 2 feet of finished grade in building or roadway areas or beneath habitable building areas such as offices.

10.7 Compaction

All fill, as well as scarified surface soils in those areas to receive fill or slabs-on-grade, should be compacted to at least 90 percent relative compaction as determined by ASTM Test Designation D1557, latest edition, at a moisture content near the laboratory optimum, except for the native expansive clays (Plasticity Index greater than or equal to 20). The native expansive clays should be compacted to between 87 and 92 percent relative compaction at a moisture content at least 3 percent over optimum. Fill should be placed in lifts no greater than 8 inches in uncompacted thickness. Each successive lift should be firm and non-yielding under the weight of construction equipment.

In pavement areas, the upper 6 inches of subgrade and full depth of aggregate base should be compacted to at least 95 percent relative compaction (ASTM D1557, latest edition), except for the native expansive clays, which should be compacted as noted

above. Aggregate base and all import soils should be compacted at a moisture content near the laboratory optimum.

10.8 Wet Weather Conditions

Earthwork contractors should be made aware of the moisture sensitivity of site soils and potential compaction difficulties. If construction is undertaken during wet weather conditions, the surficial soils may become saturated, soft and unworkable. The soils within about 2 feet of the ground water are near saturated and may become unstable during construction loading. Subgrade stabilization techniques might include the use of engineering fabrics and/or crushed rock or chemical treatment. Soil stabilization could also be required for excavations extending within about 2 feet of the ground water table, as discussed in the "Subgrade Preparation" section of this report.

10.9 Trench Backfill

Bedding and pipe embedment materials to be used around underground utility pipes should be well graded sand or gravel conforming to the pipe manufacturer's recommendations and should be placed and compacted in accordance with project specifications, local requirements or governing jurisdiction. General fill to be used above pipe embedment materials should be placed and compacted in accordance with local requirements or the recommendations contained in this section, whichever is more stringent.

On-site soils may be used as general fill above pipe embedment materials provided they meet the requirements of the "Material for Fill" section of this report. General fill should be placed in lifts not exceeding 8 inches in uncompacted thickness and should be compacted to at least 90 percent relative compaction (ASTM D1557, latest edition) by mechanical means only. Water jetting of trench backfill should not be allowed. The upper 6 inches of general fill in all pavement areas subject to wheel loads should be compacted to at least 95 percent relative compaction, except for the expansive clays(PI of 20 or greater) which should be compacted as noted in the "Compaction" section of this report.

Where relatively higher permeability sand or gravel backfill is used in trenches excavated within lower permeability (clayey) soils, we recommend that a cut-off plug of low permeability soil, controlled density fill, or other cement mixture be placed where such trenches enter the building and pavement areas. This would reduce the likelihood of water traveling along utility trenches and seeping through the trench backfill into the building and pavement areas and coming into contact with expansive subgrade material.

For trenches located near foundations, please refer to the "Foundations" section of this report for backfill recommendations.

10.10 Temporary Slopes and Excavations

The contractor should be responsible for all temporary slopes and trenches excavated at the site and design of any required temporary shoring. Shoring, bracing, and benching should be performed by the contractor in accordance with the strictest

governing safety standards. We recommend an OSHA soil type C be used for design of excavation slopes and shoring.

10.11 Area Fills on "City Parcel"

As discussed previously, up to about 5 to 6 feet of fill is planned for the sludge handling area of the "City Parcel" to raise the site grades to about Elevation 9 to 10 feet. We estimate that the area fills may cause about 1 to 1½-inches of consolidation settlement of the underlying clay soils. We anticipate that the structures planned for this area will be supported on piles, and should therefore be designed to resist downdrag loads, as discussed in the "Foundations" section of this report.

10.12 Visual Berm Permanent Slopes

In the event that a visual berm is constructed around the project, all fill and cut slopes in soil may have a maximum slope of 2:1 (horizontal:vertical). Exposed slopes require periodic maintenance due to minor sloughing and erosion. We recommend that all slopes be landscaped to minimize erosion. For slopes greater than about 5 feet high, the Geotechnical Engineer-of-Record should be contacted to review the slope stability, and provide supplemental recommendations, as necessary.

10.13 Detention Pond Discussion

We understand that a detention pond may be constructed to retain surface water for on-site treatment prior to discharge. Two hydraulic conductivity tests were performed to evaluate the permeability. The results indicate hydraulic conductivities of about $5x10^{-5}$ cm/sec and $2x10^{-7}$ cm/sec, indicating that the on-site soils likely have low to very low vertical hydraulic conductivity. Due to the horizontal layering of fill, organic soil, and natural deposition at the site, horizontal permeability could be much faster. If detention pond is to be designed, the Geotechnical Engineer-of-Record should be contacted to verify design assumptions regarding permeability and seepage.

Shallow ponds or trenches that extend below approximately Elevation 4 feet may fill with ground water, reducing the storage capacity. Slopes should have horizontal: vertical (H:V) slopes no steeper than 2:1 to ensure stability. Slopes should be protected from erosion using appropriate vegetation. For deep slopes or if rapid water level changes are planned flatter slopes may be more appropriate. Any artificial fill and organic soil adjacent to any ponds/trenches should be removed and recompacted to a horizontal distance at least 1½ times the depth of the pond/trench, measured from the top of the slope.

10.14 Temporary Dewatering

As previously discussed, measured ground water elevations and historic high ground water levels are above some of the planned excavation depths; therefore, temporary dewatering will be necessary during construction. Temporary dewatering for construction should be the responsibility of the contractor. The selection of equipment and methods of dewatering should be left up to the contractor and, due to the variable nature of the subsurface conditions, they should be aware that modifications to the dewatering system, such as adding well points, may be required during construction depending on the conditions encountered.

We recommend that any dewatering of the site be carried out in such a manner as to maintain the ground water a minimum of 3 feet below the bottom of mass excavations. The contractor should design a system to achieve this criteria. Additionally, the ground water should be maintained a minimum of 2 feet below all local excavations for deepened foundations, utilities or other structures. We understand that some localized dewatering may occur at the Cooling Basin pump pit and other small sumps for excavations as deep as Elevation -12 feet.

Should dewatering be temporarily shut down, it could have considerable detrimental affects on the excavations, including flooding, destabilization of the bottom of the excavation, shoring failures, etc. Therefore, we recommend that consideration be given to having the dewatering contractor provide backup power in case of loss of power or other redundancies, as deemed necessary.

In addition, drawdown of ground water during dewatering can cause subsidence outside of the excavation area. If drawdown to greater than Elevation -2 feet is desired for large areas (greater than about 2,500 square feet), we recommend that the affects be further evaluated by the Geotechnical Engineer-of-Record.

Special considerations may be required prior to discharge of ground water from dewatering activities depending on the environmental impacts at the site or at nearby locations. These requirements may include storage and testing under permit prior to discharge. Impacted ground water may require discharge at an offsite facility.

10.15 Surface Drainage

Positive surface water drainage gradients should be provided adjacent to the structures to direct surface water away from foundations and slabs towards suitable discharge facilities. Ponding of surface water should not be allowed on or adjacent to structures, slabs-on-grade, or pavements. Roof runoff should be directed away from foundation and slabs-on-grade. Roof runoff should be carried at least 5 feet away from foundations and slabs in closed conduits and directed to suitable discharge facilities. Downspouts may discharge onto splash-blocks provided the area is covered with concrete slabs or asphalt concrete pavements.

10.16 Landscaping Considerations

As the near-surface soils are highly expansive, we recommend greatly restricting the amount of surface water infiltrating these soils near structures and slabs-on-grade. This may be accomplished by:

- Selecting landscaping that requires little or no watering, especially within 3 feet of structures, slabs-on-grade, or pavements,
- Using low precipitation sprinkler heads,
- Regulating the amount of water distributed to lawn or planter areas by installing timers on the sprinkler system,
- Providing surface grades to drain rainfall or landscape watering to appropriate collection systems and away from structures, slabs-on-grade, or pavements,

- Preventing water from draining toward or ponding near building foundations, slabs-on-grade, or pavements, and

- Avoiding open planting areas within 3 feet of the building perimeter.

We recommend that the landscape architect incorporate these items into the landscaping plans, and that we review the plans before construction.

10.17 Construction Observation

All grading and earthwork should be performed under the observation of the Geotechnical Engineer-of-Record to check that the site is properly prepared, that selected fill materials are satisfactory, and that placement and compaction of fills is performed in accordance with our recommendations and the project specifications. Sufficient notification to us prior to earthwork is essential. The project plans and specifications should incorporate all recommendations contained in this report.

11.0 FOUNDATIONS

As discussed previously, numerous structures, with different types of loading are planned to be constructed at the site. We were provided with preliminary loads, and preliminary tolerable settlement estimates by RCEC, LLC. In general, we anticipate that due to the moderate compressibility of the soil and due to some liquefaction induced settlement, the Steam Turbine/Generator, Combustion Turbine/Generator, HRSG/Stack, Generator Step-up Transformer, and some Pipe Racks will be required to be supported on pile foundations. For smaller water tanks and moderate equipment loads, it may be feasible to support the structures on mat foundations on native soil. Other light one-story buildings may be supported on conventional spread footings or mat foundations, provided the estimated settlements are tolerable from a structural engineering viewpoint.

As discussed in the subsurface section of this report, the site subsurface soil conditions are generally clayey, and have moderate compressibility properties. Based on the results of our consolidation testing we estimate that virgin compression indexes range from about 0.11 to 0.16 and that recompression indexes range from about 0.007 to about 0.015 (compression indexes are expressed in strain). Higher over consolidation ratios (OCRs) were observed in the upper portion of the profile. In general, the compressibility properties did not appear to vary significantly across the site or with depth indicating a generally uniform soil profile. Based on the data collected, we used an OCR of 2.4 to a Elevation of about 0 feet, an OCR of about 1.8 to an elevation of about -10 feet, and an OCR of 1.5 below -10 feet. We chose a virgin index of 0.14 and a recompression index of 0.012 as appropriate design values. Because consolidation is the primary settlement mechanism for the site, settlement for shallow and mat foundations are likely to take several months to a few years to complete.

In addition to settlement due to static loading, some ground settlement may occur after seismic shaking due to seismically induced liquefaction should be considered in design. Incorporation of the liquefaction induced settlement should reflect tolerable levels of damage after a large earthquake (i.e. minimal damage, damaged but repairable, damaged and not repairable but not collapsed). Additionally, where the

site grades are to be raised several feet, such as in the sludge handling area, some ground settlement may occur, as described in the "Earthwork" section of this report.

11.1 General Building Footings

We anticipate that relatively light, miscellaneous buildings, such as office and administrative buildings, storage rooms and other light structures may be supported on conventional shallow foundations, provided the settlement estimates are tolerable from a structural engineering viewpoint. Additionally, we recommend that all existing undocumented fill and any encountered organic soil layers be removed below the building footprint and be replaced as engineered fill. Estimated fill thicknesses are presented in the "Removal of Fill" Section of this report. We estimate that typical column loads will be on the order of 100 kips for dead plus live loading, that wall footings will have dead plus live loads of about 3 kips per foot, and that some buildings may have floor loads of about 500 psf due to storage and equipment.

All footings should have a minimum width of 12 inches and should extend at least 24 inches below lowest adjacent finished grade. Lowest adjacent finished grade may be taken as the bottom of interior slab-on-grade or the finished exterior grade, excluding landscape topsoil, whichever is lower. We anticipate that the bearing elevation of the footings will be about Elevation 7 feet or higher. If footings have loads greater that about 100 kips/column or bear deeper than about 6 feet, additional analysis may be required.

Footings constructed in accordance with the above recommendations would be capable of supporting maximum allowable bearing pressures of 2,000 pounds per square foot (psf) for dead loads, 3,000 psf for combined dead and live loads, and 4,000 psf for all loads including wind or seismic. These allowable bearing pressures are based upon factors of safety of 3.0, 2.0, and 1.5 for dead, dead plus live, and seismic loads, respectively.

These maximum allowable bearing pressures are net values; the weight of the footing may be neglected for design purposes. All footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing. If it is not desired to deepen footings to meet this criteria, where trenches extend below the foundation plane of influence sand-cement slurry or a lean concrete mix may be placed as trench backfill up to the plane of influence. The mix should be able to support the required bearing pressures without significant deformation of the pipe. The concrete/slurry mix should extend to at least the invert elevation of the pipe.

All continuous footings should be reinforced with top and bottom steel to provide structural continuity and to help span local irregularities. Footing excavations should be kept moist by regular sprinkling with water to prevent desiccation. It is essential that the Geotechnical Engineer-of-Record observe the all footing excavations before reinforcing steel is placed.

Based on the assumed loads described above, and the maximum allowable bearing pressures recommended above, we estimate that total footing settlement should be less than approximately 1-inch, with post-construction differential movement between adjacent columns of approximately ½-inch. The Geotechnical Engineer-of-Record

should be retained to review the final foundation plans and structural loads to verify the above settlement estimates.

Additionally we recommend that ¾-inch of differential settlement over a distance of about 30 feet, be considered due to post-earthquake liquefaction induced settlement.

11.2 Slabs-on-Grade with Footings

Due to the moderate to high expansion potential of the near surface soils, we recommend that slabs-on-grade used in conjunction with shallow footings be supported on (at-least 24 inches of non-expansive fill to reduce the likelihood of slab damage from heave. If desired to limit moisture rise through slab-on-grade floors, the guidelines presented in the "Moisture Considerations" section of this report should be considered.

Post-construction cracking of concrete slabs-on-grade is inherent in any project, especially where soil expansion potential is high. In our opinion, consideration should be given toward a maximum control joint spacing of 10 to 15 feet in both directions for the interior slab-on-grade construction.

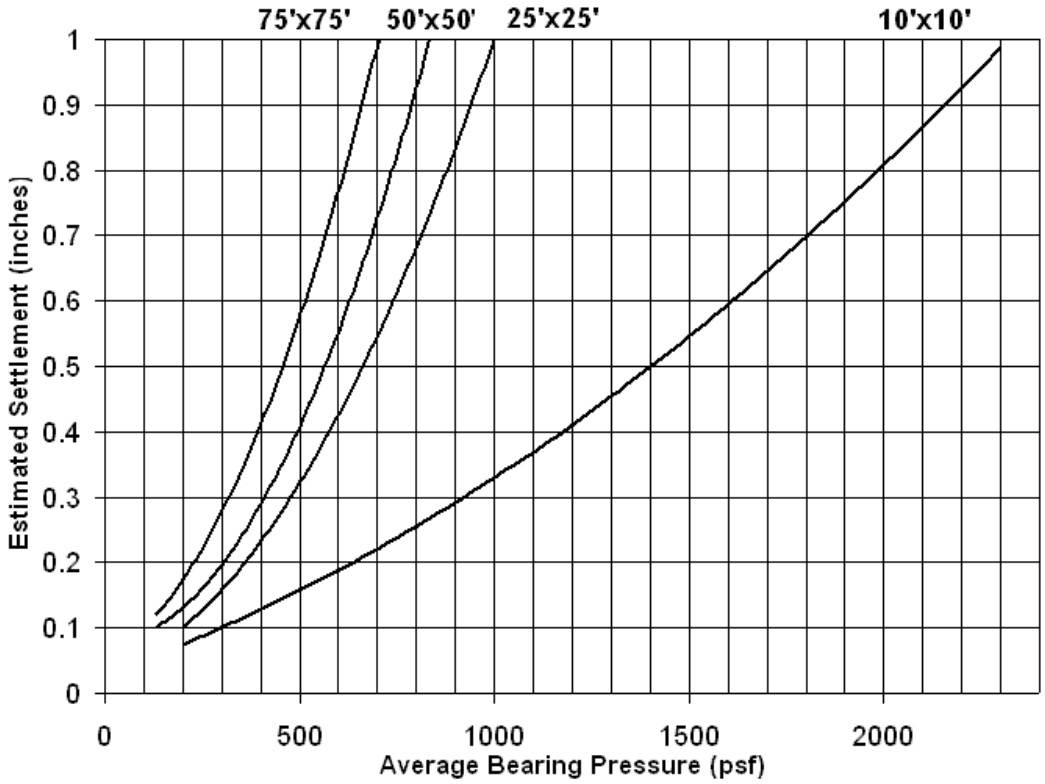
Due to the high expansion potential of the clayey native soil, we recommend that the contractor take special measures to protect the subgrade from any inflow of water during construction, especially after the floor slab has been cast. Areas to receive special attention include slab joints and areas where building columns pass through the floor slab.

Adequate slab reinforcement should be provided to satisfy with the anticipated use and loading requirements. For slabs that will be designed for heavy warehouse or wheel loads special design will be required. For floor slabs that will experience vehicle loads, the slab should be designed in accordance with the PCC pavements.

11.3 Reinforced Mat Foundations for General Structures

For heaver equipment and structures, it may be desired to use reinforced concrete mat foundations. We recommend that the perimeter edges of the mats be embedded a minimum of 24 inches below grade, be a minimum of 24-inches thick and designed in accordance with the recommendations below. We understand that some miscellaneous mats are planned to support relatively light loads, with average bearing pressures on the order of 500 pounds per square foot. We have prepared the following chart to help guide mat foundation design. The mat center settlement has been estimated up to 1 inch of settlement, for 10 foot by 10 foot, 25 foot by 25 foot square, 50 foot by 50 foot square and 75 by 75 foot square mats with different average bearing pressures. We have assumed that the mats will bear no deeper than about Elevation 5 feet. Mat foundations less than about 3 feet thick should also be designed to meet CBC conventionally reinforced mat design guidelines for highly expansive soil. CBC design criteria are presented below.

Chart 1. Estimated Settlement of General Mats (center)



We recommend that the above chart be used to estimate settlement at center of the mat for uniformly loaded square mats, up to a total static settlement of about 1-inch. Localized bearing pressures should not exceed 3,000 psf under concentrated loading areas such as wall or column loads. Average and concentrated bearing pressures may be increased by 1/3 for all loads including seismic. The bearing surface should be undisturbed native soil or compacted fill. Any existing fill and organic soil layers should be removed and recompacted if encountered at the bearing elevation of the mats. The ratio of the bearing pressure/ estimated settlement represents the modulus of subgrade reaction at the center of the mat. The modulus of subgrade reaction will be approximately double at the center of each edge of the mat, and can be interpolated between the two locations. Additionally about ½ of the settlement predicted at the center of the mat can be anticipated at the edge of the mat. The weight of the mat can be neglected for design purposes. In addition we estimate that approximately ¾-inch of differential settlement could occur between the center and corner of the mat due to seismically induced liquefaction. During design, we should work iteratively with the structural engineer to revise modulus of subgrade reaction values as necessary, based on the actual load distribution and geometry.

As discussed above, for lightly loaded mats that are less than about 3 feet thick, we recommend that they be designed for the highly expansive site soil conditions, based on the Section 1815 of the 2001 CBC. We recommend that mats be designed for an

effective plasticity index of 30, corresponding to a cantilever length (l_c) of approximately 6 feet. We understand that it may be desired to import non-plastic (sand or gravel) soil for use as general fill, and to reduce the effective plasticity of the soil for mat foundation design. The table below, summarizes how the effective plasticity can be reduced by replacing existing expansive soil with non-plastic (PI=0) fill.

Table 8. Conventional Mat Design Criteria

Depth of Non-Plastic (PI=0) Fill (feet)	Effective Plasticity Index (PI)	Approximate Cantilever Length (l_c) (feet)
0	30	6
1	26	4½
2	22	3
3	18	1¾

All mats should be reinforced, as appropriate, to provide structural continuity and to help span local irregularities. These recommendations may be revised depending on the particular design method selected by the structural engineer. It is essential that the Geotechnical Engineer-of-Record observe the mat foundation pads prior to placement of reinforcing steel.

11.4 Post-Tensioned Mats for General Structures

As an alternative to shallow footings or conventional mat foundations, the proposed general light buildings may be supported on post-tensioned mats bearing on prepared natural soil or compacted fill, prepared in accordance with the recommendations presented in the “Subgrade Preparation” and “Compaction” section of this report. Before slab construction, the subgrade surface should be proof-rolled to provide a smooth, firm surface for slab support.

Post-tension mats should be designed with the criteria presented in Table 9 below, using an average allowable bearing pressure of 700 pounds per square foot (psf) for dead plus live loads, with maximum localized bearing pressures of 3,000 psf at column or wall loads. Allowable bearing pressures may be increased by one-third for all loads including wind or seismic. The structural engineer should determine slab thickness and reinforcing in accordance with the anticipated use and loading of the slab. Settlements are anticipated to be similar to the reinforced mat foundation settlement estimates, provided in the section above.

Table 9. Post-Tension Mat Design Criteria

Condition	Center Lift	Edge Lift
Edge Moisture Variation (ft.)	5.5	2½
Differential Soil Movement (in.)	4.1	0.8

The above design criteria are based on the procedure developed by the Post-Tensioning Institute (1982) and presented in the 1997 Uniform Building Code (UBC)

using a depth to constant soils suction of 5 feet, and the Plasticity Index data presented on Figure B-1. The soil conditions were modeled using predominantly clay mineral type of Montmorillonite and 60 percent clay, as well as our engineering judgment and experience.

11.5 Water Tank Foundations

We understand that several storage tanks are planned for the project, including a 60-foot-diameter, 37–foot-high recycled water tank, a 36-foot-diameter, 37-foot-high fire storage tank, a 28-foot-diameter, and 40-foot-high demineralized water tank. We anticipate that the tank foundations will be designed in accordance with API Standard 650 Appendix B using concrete ring wall foundations. The following table summarizes the estimated ground settlement at the center of each tank under full loading and along the tank ringwalls. The Brine Concentrator tanks should be supported on deep foundations.

Table 10. Tank Settlement Estimates

Tank Identification	Average Dead Plus Live Bearing Pressure (psf)	Estimated Static Settlement at Center of Tank (inches)	Estimated Static Settlement at Ringwall (inches)
60-foot diameter Recycled Water	2,300	6	4
36-foot diameter fire/storage	2,300	4	2 to 3
28-foot diameter demineralized water tank	2,500	4	2 to 3

Because of the existing expansive soils at the site we recommend that all concrete ringwall footings extend at least 36 inches below the lowest adjacent grade, and that the tank be supported on at least 30 inches of non-expansive fill. Drainage layers below the tank, may be considered part of the non-expansive fill. Ringwall dead plus live bearing pressures should not exceed 3,000 psf. The allowable bearing pressure may be increase by 1/3 for all loads including wind and seismic. In addition to the static settlement estimated in the above, we recommend that the tanks also be designed for an additional ¾-inch of differential ground settlement over a distance of approximately 30 feet due to post-earthquake liquefaction induced settlement. Lateral loads may be resisted as described in the “Lateral Load” Section of this report.

Pipe connections, and other improvements adjacent to the tanks should be designed for the ring wall settlements described above.

11.6 Reinforced Mat Foundations for Power Plant Structures

Although it was anticipated before analysis that some equipment will be required to be supported on deep foundations or improved ground, we checked the settlement of all of the equipment loads, to see if mat foundations bearing on the existing ground would be suitable for the project. Table 11, below, summarizes our understanding of the estimated loading, tolerable settlement, and our estimated settlement if the structures were supported on flexible mat foundations with uniform bearing pressures.

Because of the relatively large areas of the new structure mats, we assumed that the soil would be completely rebounded before loading, so even though the net pressure change is small for some structures, such as the Cooling Tower and Basin, recompression settlement was judged to be likely. In addition to the settlement estimates shown in the table, mats bearing on improved ground or on the existing ground conditions should be expected to experience an additional ¾-inch of differential settlement due to seismically induced liquefaction.

Based on our analysis it appears that the Cooling Tower and Basin will be able to be supported on a conventional mat foundation. The CTG, and chlorine contact basin may be able to be designed for the estimated settlements, or alternatively may require deep foundations to support the structure. Additionally, pipe supports and other structures connected to structures supported on deep foundations should be designed for up to 1½-inches of post earthquake differential settlement, and may require deep foundations to mitigate differential settlement risks.

For the cooling Tower and Basin we recommend a variable modulus of subgrade reaction (MSGR)of 6 pounds per cubic inch(pci) at the center of the mat and 12 pci at the center of each edge. For initial analysis, we recommend that for the CTG, the 45-foot high water tank and the Chlorine Contact Basin a MSGR of 3 pci at the center and 6 pci at the center of the edge of the mat for the existing soil conditions. The MSGR can initially be assumed to vary linearly between the given locations. During design, the Geotechnical Engineer-of-Record should work iteratively with the structural engineer to verify the loading distribution and revise the MSGR as necessary. The bearing surface should be undisturbed native soil or compacted fill. Concentrated loads should not exceed 3,000 psf for dead plus live loading. Average and concentrated bearing pressures may be increase by 1/3 for all loads including seismic. Any existing fill and organic soil layers should be removed and recompacted, if encountered, at the bearing elevation of the mats.

Table 11. Settlement Estimate Summary for Mat Foundations on Existing (non-improved) Soil Conditions

Structure	Estimated Bearing Elevation (ft)	Loading Condition	Approximate Bearing Pressure (psf)	Tolerable Settlement (in)[1]		Estimated Static Settlement (in)[2]		Comments
				Total	Diff.	Total	Diff.	
STG	3 to 4	Dead	2,900	¾	3/8	16	8	Exceeds tolerable limits – Deep foundations will be required to mitigate settlement
		+ Live	3,250	½	¼	2	1	
			Total Static Settlement Estimate			18	9	
HRSG	5 to 6	Dead	2,900	1	½	13	6	Exceeds tolerable limits – Deep foundations required to mitigate settlement
		+ Live	3,100	¾	½	1	½	
			Total Static Settlement Estimate			14	6½	
CTG	4 to 5	Dead	1,100	1	½	3	1½	Exceeds tolerable limits - Deep required to mitigate settlement
		+Live	1,200	½	¼	½	<½	
			Total Static Settlement Estimate			3½	2	
Cooling Tower & Basin	0 to 1	Dead	400	1½	¾	-	-	Settlement appears to be acceptable for mat foundation
		+ Live	800	2	1	1	½	
			Total Static Settlement Estimate			1	½	
Transformers	5 to 6	Dead	-	-	-	-	-	Exceeds tolerable limits – Deep foundations required to mitigate settlement
		+ Live	2,500	1½	¾	6	3	
			Total Static Settlement Estimate			6	3	
Chlorine Contact Basin	2 to 4	Dead	-	-	-	-	-	Loading estimated by TRC, tolerable settlement estimates not available from RCEC
		+ Live	1,400	n/a[3]	n/a	3	1½	
			Total Static Settlement Estimate			3	1½	

[1] Calpine Provided Preliminary Tolerable Settlement Estimates
[2] In addition to static settlements, seismically induced liquefaction settlement should also be considered for building design
[3] Tolerable settlement estimates were not proved by Calpine for the chlorine contact basin.

All mats should be reinforced, as appropriate, to provide structural continuity and to help span local irregularities. These recommendations may be revised depending on the particular design method selected by the structural engineer. It is essential that the Geotechnical Engineer-of-Record observe the mat foundation pads prior to placement of reinforcing steel.

11.6 Lateral Loads for Mats and Footings

Lateral loads may be resisted by friction between the bottom of mats or footings and the supporting subgrade. A maximum allowable frictional resistance of 0.25 may be used for design. In addition, lateral resistance may be provided by passive pressures acting against foundations poured neat against competent soil. We recommend that an allowable passive pressure based on an equivalent fluid pressure of 300 pounds per cubic foot (pcf) be used in design. The upper 12 inches of soil should be neglected when determining lateral passive resistance.

11.8 Moisture Protection Considerations Mats and Slabs

We do not anticipate that moisture protection will be required for most of the equipment pads; however, for office areas or other interior spaces to receive floor coverings, the guidelines for floor moisture protection are included for completeness. Since the long-term performance of concrete floor coverings depends to a large degree on good design, workmanship, and materials, the following general guidelines are presented for consideration by the developer, design team, and contractor. The purpose of these guidelines is to aid in producing a concrete mat of sufficient quality to allow successful installation of floor coverings and reduce the potential for floor covering failures due to moisture-related problems associated with mat foundation construction. These guidelines may be supplemented, as necessary, based on the specific project requirements.

- A minimum 10-mil thick vapor barrier meeting ASTM E 1745 Class C should be placed directly below the mat or concrete slab. The vapor barrier should extend to within 12 to 18 inches of the edge of the mat, as determined by the structural engineer. At least 4 inches of free-draining gravel, such as ½-inch or ¾-inch crushed rock with no more than 5 percent passing the ASTM No. 200 sieve, should be placed below the vapor barrier to serve as a capillary break. The vapor barrier should be sealed at all seams and penetrations, in accordance with ASTM E1643.

- The concrete water/cement ratio should not exceed 0.45. Midrange plasticizers could be used to facilitate concrete placement and workability.

- Water should not be added after initial batching, unless the slump of the concrete is less than specified, and the resulting water/cement ratio will not exceed 0.45.

- Polishing the concrete surface with metal trowels should not be permitted.

- All concrete surfaces to receive any type of floor covering should be moist cured for a minimum of 7 days. Moist curing methods may include frequent sprinkling, or using coverings such as burlap, cotton mats, or carpet. The

covering should be placed as soon as the concrete surface is firm enough to resist surface damage. The covering should be kept continuously wet and not allowed to dry out during the required curing period.

- Water vapor emission levels and pH should be determined as required by the manufacturer's of the floor covering materials before floor installation. Measurements and calculations should be made according to ASTM F1869-98 and F710-98 protocol.

The guidelines presented above are based on information obtained from various technical sources, including the American Concrete Institute (ACI), and are intended to present information that can be used to reduce potential long-term impacts from slab moisture infiltration. It should be noted, the application of these guidelines does not affect the geotechnical aspects of the foundation performance.

11.9 Friction Piles

Pile foundations will, in our opinion, support the proposed structure loads with relatively minor settlement. Therefore, we recommend that where the mat settlements greatly exceed the tolerable limits and ground improvement is not feasible or cost effective, that the structures be supported on driven, precast, prestressed concrete friction piles. Concrete floors should be designed to span between pile supports to prevent unwanted differential movement.

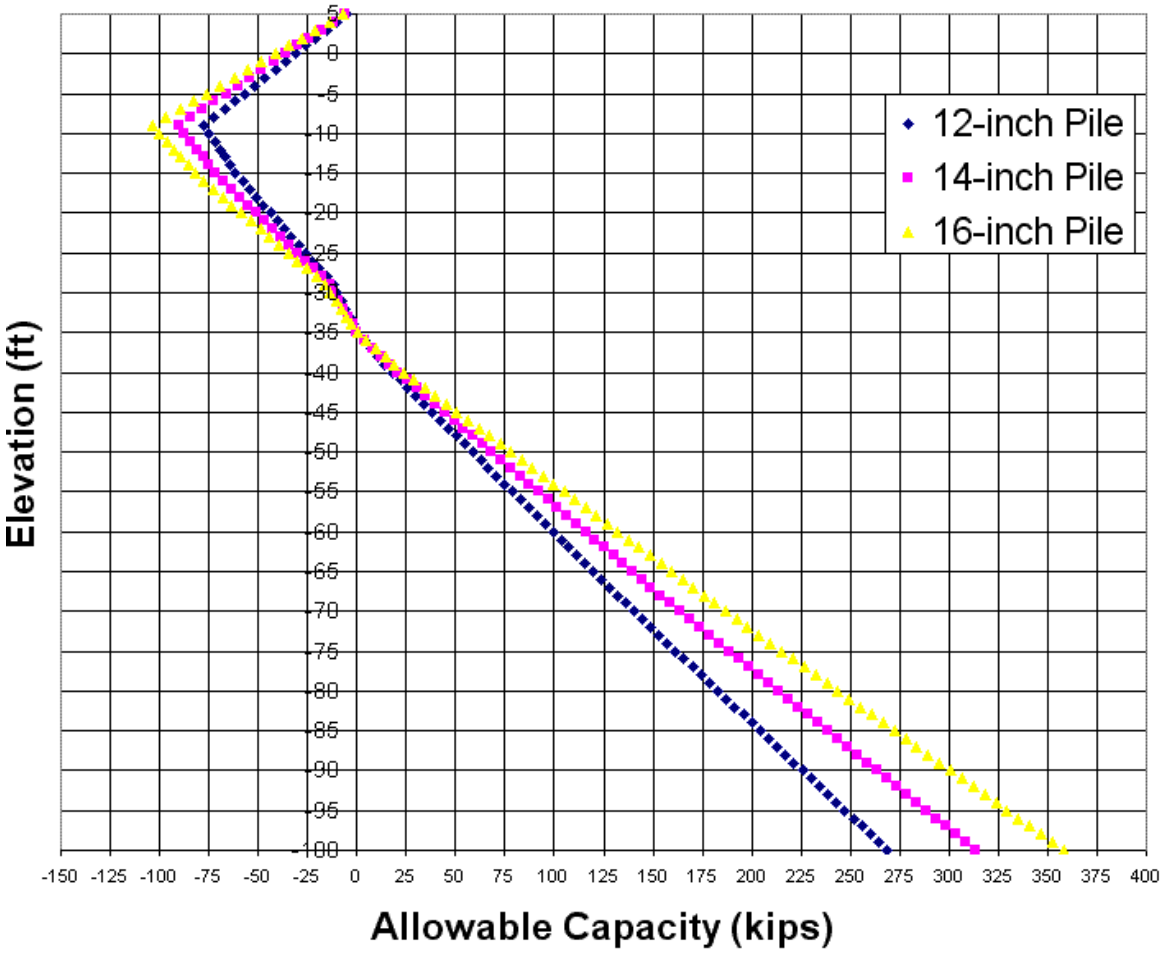
Some sand and silt layers at various depths were encountered that could require pre-drilling, to allow pile driving. If pre-drilling is required, a smaller diameter auger than the width of the pile should be used. Based on the CPTs some relatively thick sand layers were encountered between about 30 and 40 feet in CPT-2 and at a depth of about 50 feet in CPT-1. The layers are too thin, and discontinuous for end bearing, however, the contractor should be aware of possible driving difficulties related to interbedded sand layers and the requirement that piles should reach tip elevation.

11.9.1 Vertical Loads

Our exploration indicates there is no significantly thick or continuous dense sand layer that would provide end bearing support. Therefore, pile support is expected to come predominantly from frictional support in the stiff clays. We computed allowable downward vertical capacities for 12-, 14-, and 16-inch-square concrete piles, the results of which are presented in the Table below. In addition to the structural loads, the 12-, 14-, and 16-inch square piles should be designed to support 76, 90, and 103 kips of downdrag load, respectively. The allowable pile capacity shown in the graph includes downdrag loads. The downdrag loads include loads due to earthwork fill settlement and liquefaction-induced settlement, as both are estimated to cause settlement in the same portion of the soil profile.

Dead loads should not exceed two-thirds of the computed capacities. Uplift loads should also not exceed two-thirds of the computed downward capacities. The pile capacities and uplift loads may be increased by one-third under transient loading, including wind and seismic.

Chart 2. Allowable Driven Pile Capacities



Our allowable pile capacities include 5 feet of reduced strength soil between the approximate depths of 35 and 40 feet to account for potential strength loss of liquefiable soils. Gross capacity of the piles should not exceed the structural capacity. We have assumed a base of pile cap at Elevation 5 feet for our analysis. To effectively minimize pile group effects and reduction in individual pile capacity, piles should be located with a minimum center-to-center spacing of three times the pile width.

Based on the maximum allowable loads for a single pile, we estimate total settlements of less than ¾-inch to mobilize allowable static capacities. Larger pile groups could settle more (greater than 4 by 4). We estimate total static settlement on the order of 1 to 1½ inches under large group loading. Pile numbers, spacing and depth will effect the pile settlement. The Geotechnical Engineer-of-Record should verify settlement estimates once pile loads and layout are determined. Therefore, post-construction pile foundation differential settlements about ½-inch should be considered.

11.9.2 Lateral Loads On Driven Concrete Piles

Lateral load resistance for pile-supported structures may be developed through pile bending/soil interaction. The magnitude of the lateral load resistance is dependent upon many factors, including pile stiffness and embedment length, conditions of fixity at the pile cap, the physical properties of the surrounding soils, the tolerable top deflection and the yield moment capacity of the pile.

To estimate lateral capacities of piles, we used a computer program that models the soil response in the form of load-deflection (p-y) curves to estimate the capacity of the piles to resist the expected lateral loads. The lateral load characteristics for 12-, 14- and 16-inch-square, driven concrete piles with free head fixed head conditions are presented in Table 12 below.

Table 12. Estimated Lateral Pile Performance for Driven Piles

Pile Size	Pile Fixity	Pile cap deflection (inches)	Maximum Moment Developed (ft-kips)	Depth to Maximum Moment (feett)	Depth to first zero moment (feet)	Developed Shear at Pile Cap (kips)
12-inch	Fixed	½	125	at pile cap	18	35
	Free		50	6	14	18
14-inch	Fixed	½	175	at pile cap	19	44
	Free		71	6	16	23
16-inch	Fixed	½	242	at pile cap	21	52
	Free		100	7	18	28

The table represents the probable response of the piles under short-term loading conditions and includes no factor-of-safety. Suitable structural factors-of-safety should be selected on the basis of the type of loading. Pile stiffnesses (EI) of 7.1 x 10^9 lb-in^2, 1.3 x 10^{10} lb-in^2, and 2.2 x 10^{10} lb-in^2 have been assumed in our calculations of load deflection for the 12-,14-, and 16-inch piles, respectively. A minimum compressive strength of 6,000 pounds per square inch was assumed for concrete modulus calculations. If pile stiffness varies by no more than 20 percent than that reported above, load deflection characteristics can be approximated by multiplying the deflection values by the ratio of the pile stiffness (EI). The Geotechnical Engineer-of-Record should evaluate the response of piles with significantly different stiffness.

The above lateral load characteristics are for single piles and may not be characteristic of the lateral load capacity of piles in a group. Group effects may reduce the allowable lateral load for a given deflection. We recommend that a pile group efficiency of 0.75 be used for pile groups 3-by-3 or smaller. A group reduction would not be necessary for groups of 1 or 2 piles. For pile groups larger than 3-by-3 with pile spacing of at least 3 diameters, we recommend a group efficiency factor of 0.65 be used for initial planning and design estimating. Because group pile interaction is a complex soil-structure interaction problem that depends on vertical loading, pile length, pile layout, lateral loading, pile cap size, soil conditions, and other factors, we recommend that lateral pile group analysis be performed using the actual layout and structural loads to verify the adequacy of design. We recommend that the Geotechnical Engineer-of-

Record review the final pile group layout and structural loads to further evaluate the pile group efficiency under lateral loading for pile groups larger than 3-by-3.

11.9.3 Passive Resistance Against Pile Caps and Grade Beams

If desired, the passive resistance of soil against pile caps and grade beams poured neat against well-compacted engineered fill may be used for lateral resistance. We recommend that an allowable passive pressure based on an equivalent fluid pressure of 150 pounds per cubic foot be used in design for pile head deflections of about ½-inch. If deflections are less than this, the passive pressure may not be fully mobilized and the Geotechnical Engineer-of-Record may need to provide alternate, strain compatible passive pressures.

11.9.4 WEAP Analysis

At a minimum, we recommend that the pile contractor have a wave equation analysis of piles (WEAP) performed to confirm compatibility and driveability of the pile driving system with the pile type and soil conditions at the site. The Geotechnical Engineer-of-Record should review the WEAP input parameter and results prior to mobilization of pile driving equipment to the site. The hammer, fuel settings, cushion and pile input parameters should strictly reflect the equipment that will be used during construction.

11.9.5 Indicator Piles

It has been our experience that uncertainties associated with production pile driving can be reduced considerably by implementing an indicator pile program. An indicator pile program will also provide a better means of confirming the limits of layers where high driving resistance may be encountered, and to more accurately estimate final pile lengths.

We recommend that at least three indicator piles be installed for each structure before the final pile casting lengths have been selected. The indicator piles should be driven with the same equipment that will be used to drive the production piles. The Geotechnical Engineer-of-Record should review or select the indicator pile locations when structural drawings are made available. The indicator pile cast lengths should be based on the design lengths required to meet the desired capacity, plus 5 feet. It is expected that some indicator piles may not be driven to their entire length and will require cutting to provide the desired butt elevation. Indicator piles can be used for support of the structure and, therefore, should be located appropriately. We also suggest that one or more spare piles be delivered to the site during the indicator program.

11.9.6 PDA Monitoring

A Pile Driving Analyzer (PDA) should be used during the indicator pile program to determine approximate pile capacities through dynamic testing. PDA monitoring may allow a reduction in production pile lengths and thus cost savings to the project. PDA monitoring should be performed during indicator driving and on selected piles for restrikes; restrikes should be performed no sooner than seven days after initial driving, as much of the pile capacity develops during "set up" after pile driving. Additionally, in some cases even after 7 days piles may not yet demonstrate full capacity and could require additional set up time or lengthening of the piles. *Please note that restrike testing more than one day after installation may significantly alter the contractor's sequencing. Therefore, restrike testing should be clearly identified on the plans and specifications to avoid unexpected costly change-orders for out of sequence moves.* Additional flexibility for PDA monitoring during production would be especially beneficial for checking tensile stresses in the piles and for evaluating pile integrity on any piles suspected of being damaged during indicator or production driving. Piles designated for PDA monitoring during indicator pile installation should be at least 10 feet longer than design length so that the gauges are not driven into the ground. At least one pile per structure should have PDA monitoring or be statically load tested (ASTM D1143). Static load testing equipment should have structural and measurement capacity to fully fail the pile(on the order of 1000 kips), to verify assumptions. If dynamic testing is not utilized, in addition to the pile top, we recommend that at least 2 piles be instrumented with strain gauges in the upper 15 feet of the pile, and at the 1/3 and 2/3 points of the pile, to help verify soil layering strengths.

11.9.7 Production Pile Installation

We recommend that a pile hammer capable of delivering the required energy to both advance the pile and prevent overstressing of the pile be used during construction. The same hammer should be used for both the indicator piles and the production piles. The pile contractor should perform wave equation analysis to confirm the compatibility and driveability of the pile driving system with the pile type and soil conditions at the site. We should review the wave equation results prior to mobilization of pile driving equipment to the site.

Since the piles are designed for skin friction support, they should be driven to the desired tip elevation. If difficult driving conditions are encountered, the Geotechnical Engineer-of-Record should review the driving record and evaluate potential tip capacity to allow reduction in pile length. The Geotechnical Engineer-of-Record may also recommend that a Pile Driving Analyzer (PDA) be used during production driving to determine approximate pile capacities through dynamic analyses. PDA monitoring would be especially beneficial for checking restrike capacities of any piles short of required tip elevation or for evaluating pile integrity on any piles suspected of being damaged during driving. The Geotechnical Engineer-of-Record should observe all indicator and production pile installation on a full-time basis.

11.10 Auger-Cast Piles

As an alternative to driven piles, auger cast piles may be a suitable alternative for the project. Auger-cast piles are typically designed by a design-build contractor. Auger

cast piles have the advantages of lower noise and vibration levels and less chance of pile driving difficulties.

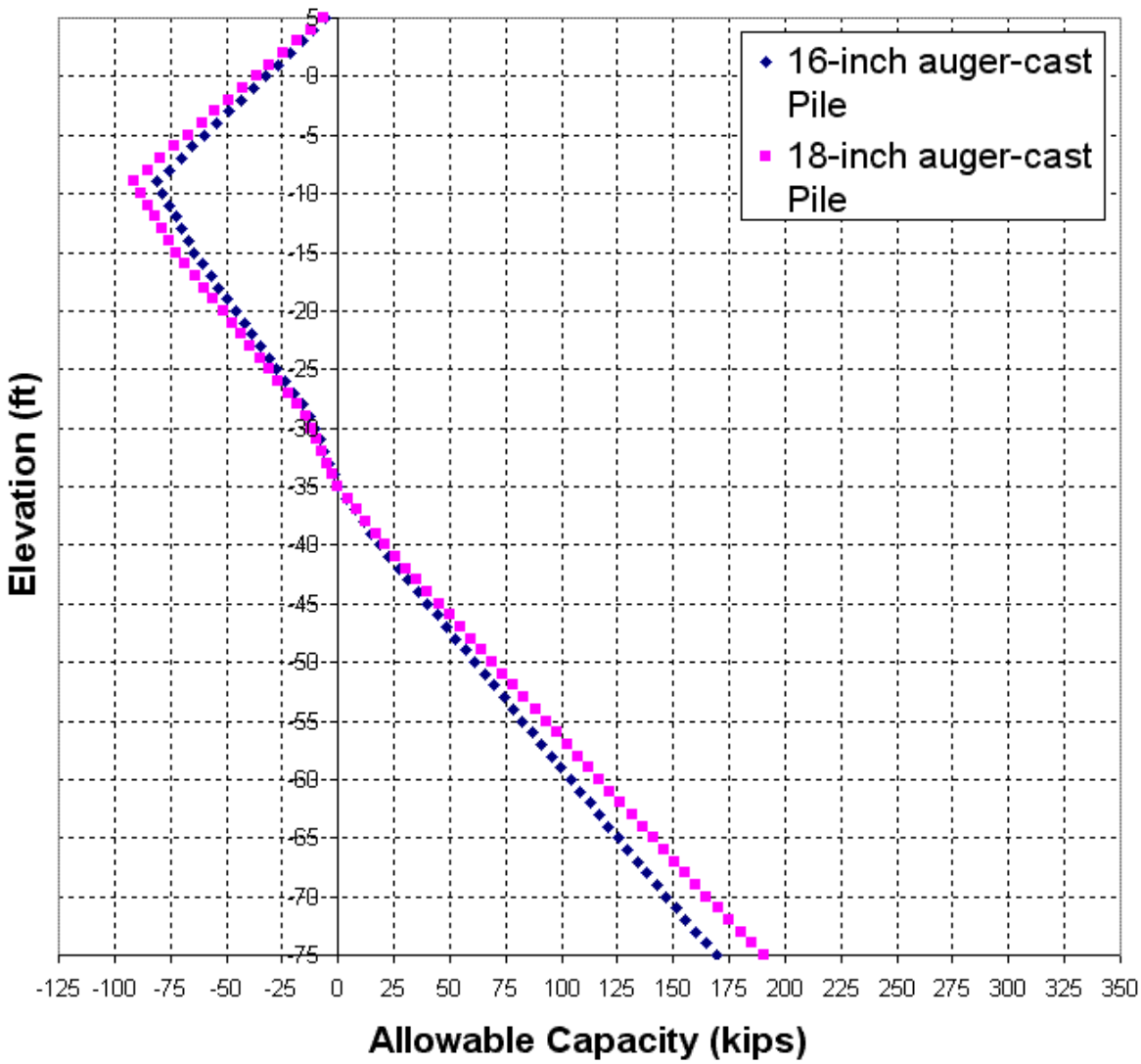
Pressure Grouted Auger-Cast Displacement piles typically can be constructed in 16- to 18- inch diameters to depths of about 70 to 80 feet, and have similar skin friction capacities as driven concrete piles. Drilling spoils are limited as the soil is displaced laterally as the auger is advanced. The auger is withdrawn as the concrete is placed.

Conventional auger-cast piles can be constructed in larger diameters, and typically can be constructed deeper than displacement auger cast piles, although drilling spoils are generated and the skin friction capacity is somewhat lower than for displacement or driven piles.

The graph below shows our estimated pile capacity for 16- and 18-inch diameter auger-cast displacement pile for initial pile size estimation for dead plus live loading. Dead loads should not exceed 2/3 of the allowable loads. The allowable capacities may be increased by 1/3 for all loads including seismic. Tensile loading should not exceed two-thirds of the allowable compressive capacity of the pile. Because of depth limitations, we recommend that maximum pile lengths of 70 to 80 feet be used in design. Due to potential downdrag, the pile capacity is shown as negative in the upper portion of the pile.

The Geotechnical Engineer-of-Record should review any auger cast pile designs prior to construction to verify design assumptions by the design-build contractor. Auger cast piles should be statically load tested in accordance ASTM D1143 and with local standard of practice to verify pile capacity. The auger cast pile contractor should submit a load testing plan for approval prior to mobilizing to the site. At a minimum we recommend at least one pile per structure be statically tested. If load testing is performed during the design of the piles, the load testing equipment and strain gauge equipment should be able to loading the pile past failure (at-least 600 kips). Piles to be load tested should be instrumented with strain gauges to measure capacity of different soil layers. At a minimum, strain gauges should be located at the pile top, within the upper 15 feet, at the 1/3 length of the pile, and at the 2/3 length of the pile. Piles should be allowed sufficient time to set up before load testing. If load testing indicates higher than anticipated capacity designs may be revised to result in optimized pile design. Alternatively, load testing of production piles would only be able to verify design capacities.

Chart 3. Auger Cast Displacement Pile Estimated Capacity



11.10.1 Lateral Loads On Auger Cast Displacement Concrete Piles

The lateral load characteristics for 16- and 18-inch auger cast displacement piles with free head fixed head conditions are presented in Table 13 below. Auger cast piles are typically designed by a design-build contractor, the estimated response below is for planning purposes. The Geotechnical Engineer-of-Record should review the designs prior to mobilization to the site.

Table 13. Estimated Lateral Pile Performance for Auger Cast Displacement Piles

Pile Diameter	Pile Fixity	Pile cap deflection (inches)	Maximum Moment Developed (ft-kips)	Depth to Maximum Moment (ft)	Depth to first zero moment (feet)	Developed Shear at Pile Cap (kips)
16-inch	Fixed	½	173	at pile cap	18	44
	Free		69	6	15	23
18-inch	Fixed	½	226	at pile cap	20	52
	Free		89	6	16	27

The table represents the probable response of the piles under short-term loading conditions and includes no factor-of-safety. Suitable factors-of-safety should be selected on the basis of the type of loading. Pile stiffnesses (EI) of 1. 2x 10^{10} lb-in^2 and 2.0 x 10^{10} lb-in^2, have been assumed in our calculations of load deflection for the 16-, and 18-inch piles, respectively. A minimum compressive strength of 4,000 pounds per square inch was assumed for concrete modulus calculations. If pile stiffness varies by no more than 20 percent than that reported above, load deflection characteristics can be approximated by multiplying the deflection values by the ratio of the pile stiffness (EI). The Geotechnical Engineer-of-Record should evaluate the response of piles with significantly different stiffness.

The above lateral load characteristics are for single piles and may not be characteristic of the lateral load capacity of piles in a group. Group effects may reduce the allowable lateral load for a given deflection. We recommend that a pile group efficiency of 0.75 be used for pile groups 3-by-3 or smaller. A group reduction would not be necessary for groups of 1 or 2 piles. For pile groups larger than 3-by-3 with pile spacing of at least 3 diameters, we recommend a group efficiency factor of 0.65 be used for initial planning and design estimating. Because group pile interaction is a complex soil-structure interaction problem that depends on vertical loading, pile length, pile layout, lateral loading, pile cap size, soil conditions, and other factors, we recommend that lateral pile group analysis be performed using the actual layout and structural loads to verify the adequacy of design. We recommend that the Geotechnical Engineer-of-Record review the final pile group layout and structural loads to further evaluate the pile group efficiency under lateral loading for pile groups larger than 3-by-3.

12.0 RETAINING WALLS

12.1 Lateral Earth Pressures

Any proposed retaining walls should be designed to resist lateral earth pressures from adjoining natural materials, backfill, and surcharge loads. Provided that adequate drainage is provided as recommended below, we recommend that walls restrained from movement at the top be designed to resist an equivalent fluid pressure of 45 pounds per cubic foot (pcf) plus a uniform pressure of 8H pounds per square foot, where H is the distance in feet between the bottom of the footing and the top of the retained soil. Restrained walls should also be designed to resist an additional uniform pressure equivalent to one-half of any surcharge loads applied at the surface. Any

unrestrained retaining walls with adequate drainage should be designed to resist an equivalent fluid pressure of 45 pcf plus one-third of any surcharge loads.

The above lateral earth pressures assume level backfill conditions and sufficient drainage behind the walls to prevent build-up of hydrostatic pressure from surface water infiltration and/or a rise in the ground water level. If adequate drainage is not provided, we recommend an equivalent fluid pressure of 40 pcf be added to the values recommended above for both restrained and unrestrained walls. Damp proofing of the walls should be included in areas where wall moisture and efflorescence would be undesirable.

We understand that the structural design may include seismic pressures for critical retaining walls. Seismic soil pressures for cantilever walls are estimated to be an additional resultant load of $13H^2$ (pounds) where H is the height of the wall in feet. The resultant should be distributed as an inverted triangle over the wall with the resultant acting a distance of 1/3 H from the top of the wall. For restrained walls, we recommend that the walls be designed for an additional seismic resultant of $5H^2$ distributed uniformly as 5H (in psf) over the wall.

12.2 Drainage

Adequate drainage may be provided by a subdrain system behind the walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition. The permeable backfill should extend at least 2 feet out from the wall and to within 2 feet of outside finished grade. Alternatively, ½-inch to ¾-inch crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as TCMirafi 140N or equivalent. The upper 2 feet of wall backfill should consist of relatively impervious compacted on-site clayey soil. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or Enkadrain drainage matting may be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. The drainage panel should be connected to the perforated pipe at the base of the wall.

12.3 Backfill

Backfill placed behind the walls should be compacted to at least 90 percent relative compaction using light-weight (walk behind) compaction equipment. If heavy compaction equipment is used, the walls should be temporarily braced.

12.4 Foundation

Retaining walls may be supported on a continuous spread footing designed in accordance with the recommendations presented in the “Footings” section of this report. Lateral load resistance for the walls may be developed in accordance with the recommendations presented in the “Lateral Loads” section.

13.0 PAVEMENTS

13.1 Asphalt Concrete

We obtained three bulk samples of the surface soil for R-value testing to provide data for pavement thickness design. The results of the test are included in Appendix B and indicate R-values of 12 to 19. We judged an R-value of 10 to be applicable for design. Using estimated traffic indices for various pavement-loading requirements, we developed the following recommended pavement sections based on Procedure 608 of the Caltrans Highway Design Manual, presented in Table 14.

Table 14. Recommended Asphalt Concrete Pavement Design Alternatives
Pavement Components
Design R–Value =10

General Traffic Condition	Design Traffic Index	Asphalt Concrete (Inches)	Aggregate Baserock* (Inches)	Total Thickness (Inches)
Automobile Parking	4.0	2.5	7.0	9.5
	4.5	2.5	8.5	11.0
Automobile Parking Channel	5.0	3.0	9.0	12.0
	5.5	3.0	11.0	14.0
Truck Access & Parking Areas	6.0	3.5	11.5	15.0
	6.5	4.0	13.0	17.0

*Caltrans Class 2 aggregate base; minimum R-value equal to 78.

The traffic indices used in our pavement design are considered reasonable values for the proposed development and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. Because the native soils at the site are highly expansive, some increased maintenance and reduction in pavement life can be expected. The traffic parameters used for design were selected based on engineering judgment and not on information furnished to us such as an equivalent wheel load analysis or a traffic study. For unusually heavy trucks or high truck traffic, a thicker pavement section may be required. The Geotechnical Engineer-of-Record should be contacted to provide additional pavement recommendations, for specific loading concerns, if required.

13.2 Portland Cement Concrete Pavements

Recommendations for Portland Cement Concrete (PCC) pavements are presented below in Table 15. Since the expected Average Daily Truck Traffic (ADTT) is not known at this time, we have provided alternatives for minimum pavement thickness. An allowable ADTT should be chosen that is greater than expected for the development.

Table 15. Recommended Minimum PCC Pavement Thickness

Allowable ADTT	Minimum PCC Pavement Thickness (inches)
1	4.5
25	5.5
100	6.0

Our design is based on an R-value of 10 and a modulus of rupture of 600 pounds per square inch. In addition, our design assumes that pavements are restrained laterally by a concrete shoulder or curb and that all PCC pavements are underlain by at least 12 inches of Class 2 aggregate base. We recommend that adequate construction and control joints be used in design of the Portland Cement Concrete pavements to control the cracking inherent in this construction. For unusually heavy trucks or high truck traffic, a thicker pavement section may be required. The Geotechnical Engineer-of-Record should be contacted to provide additional pavement recommendations, for specific loading concerns, if required.

13.3 Pavement Cutoff

Because the native soils at the site are highly expansive, surface water infiltration beneath pavements could significantly reduce the pavement design life. While the amount of reduction in pavement life is difficult to quantify, in our opinion, the normal design life of 20 years may be reduced to less than 10 years. Therefore, long-term maintenance greater than normal may be required.

To limit the need for additional long-term maintenance, it would be beneficial to protect at-grade pavements from landscape water infiltration by means of a concrete cut-off wall, deepened curbs, redwood header, “Deep-Root Moisture Barrier,” or equivalent. However, if reduced pavement life and greater than normal pavement maintenance are acceptable, the cutoff barrier may be eliminated. If desired to install pavement cutoff barriers, they should be considered where pavement areas lie downslope of any landscape areas that are to be sprinklered or irrigated, and should extend to a depth of at least 4 inches below the base rock layer.

13.4 Asphalt Concrete, Aggregate Base and Subgrade

Asphalt concrete and aggregate base should conform to and be placed in accordance with the requirements of Caltrans Standard Specifications, latest edition, except that ASTM Test Designation D1557 should be used to determine the relative compaction of the aggregate base. Pavement subgrade should be prepared and compacted as described in the “Earthwork” section of this report.

13.5 Exterior Concrete Flatwork

Due to the high plasticity soils, we recommend that exterior concrete flatwork be supported on at least 18 inches of non-expansive fill. Exterior concrete sidewalks should be at least 4 inches thick and underlain by at least 4 inches of Class 2 aggregate base compacted to a minimum of 90 percent relative compaction in accordance with ASTM Test Method D1557, latest edition. The 4 inches of aggregate

base may be considered part of the non-expansive fill requirement. If sidewalks are subject to wheel loads, they should be designed in accordance with the "Portland Cement Concrete Pavements" section of this report.

14.0 LIMITATIONS

This report has been prepared for the sole use of Russell City Energy Center, LLC, specifically for design of the Russell City Energy Center in Hayward, California. The opinions presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in the San Francisco Bay Area at the time this report was written. No warranty, expressed or implied, is made or should be inferred.

The opinions, conclusions and recommendations contained in this report are based upon the information obtained from our investigation, which includes data from widely separated discrete locations, visual observations from our site reconnaissance, and review of other geotechnical data provided to us, along with local experience and engineering judgment. The recommendations presented in this report are based on the assumption that soil and geologic conditions at or between borings do not deviate substantially from those encountered or extrapolated from the information collected during our investigation. We are not responsible for the data presented by others.

The Geotechnical Engineer-of-Record should be retained to review the geotechnical aspects of the final plans and specifications for conformance with our recommendations and any other supplemental recommendations developed during the design-build process. The recommendations provided in this report are based on the assumption that we will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, TRC Lowney cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of TRC Lowney' report by others. Furthermore, TRC Lowney will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services and/or at the time another consultant is retained for follow up service to this report. If another engineer is retained in our place, that engineer should review this report, accept or modify the recommendations as they feel necessary, and provide a letter to Russell City Energy Center, LLC acknowledging their assumption of the role as Geotechnical Engineer-of-Record.

The opinions presented in this report are valid as of the present date for the property evaluated. Changes in the condition of the property will likely occur with the passage of time due to natural processes and/or the works of man. In addition, changes in applicable standards of practice can occur as a result of legislation and/or the broadening of knowledge. Furthermore, geotechnical issues may arise that were not apparent at the time of our investigation. Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of our control. Therefore, this report is subject to review and should not be relied upon after a period of three years, nor should it be used, or is it applicable, for any other properties.

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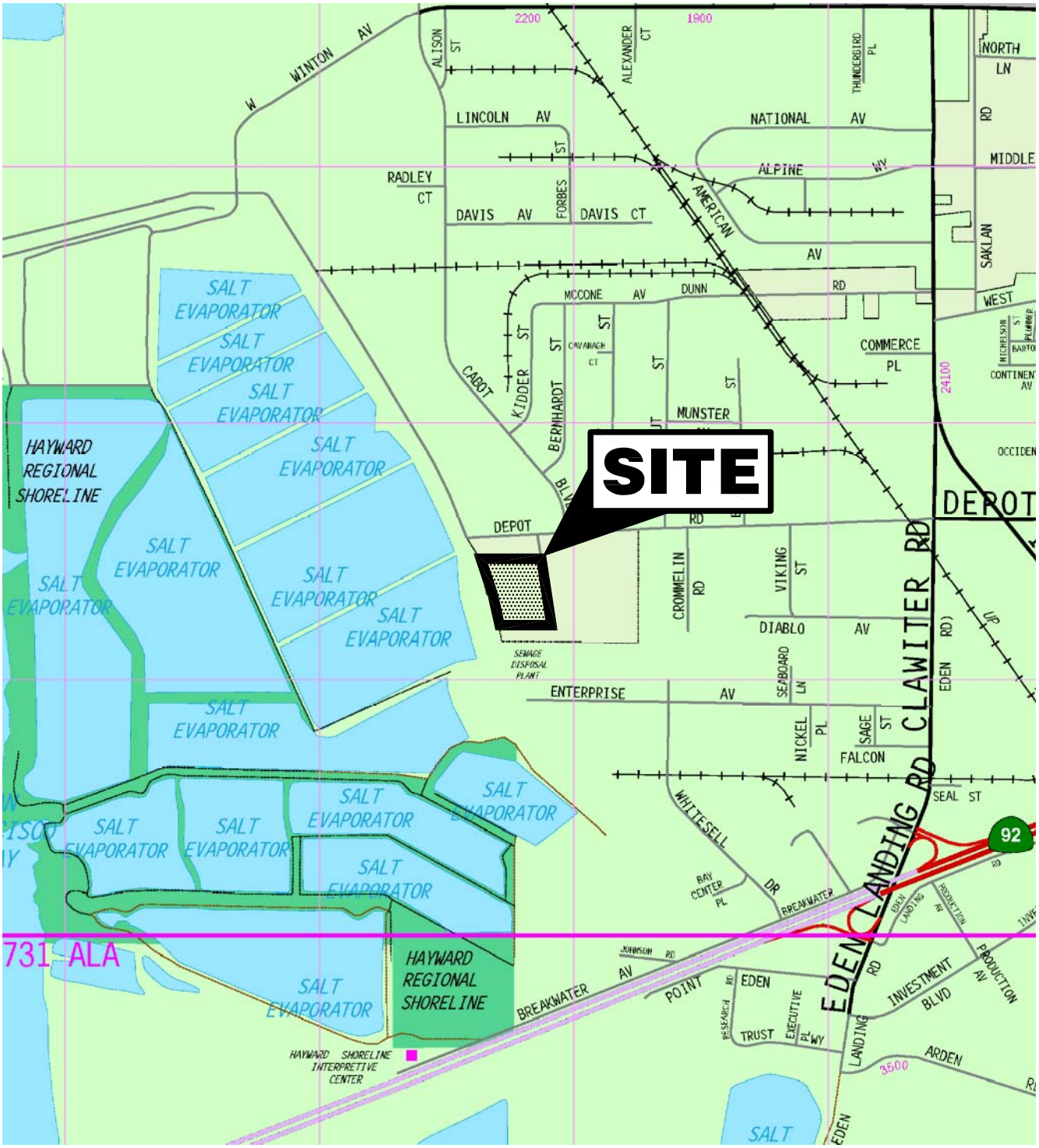

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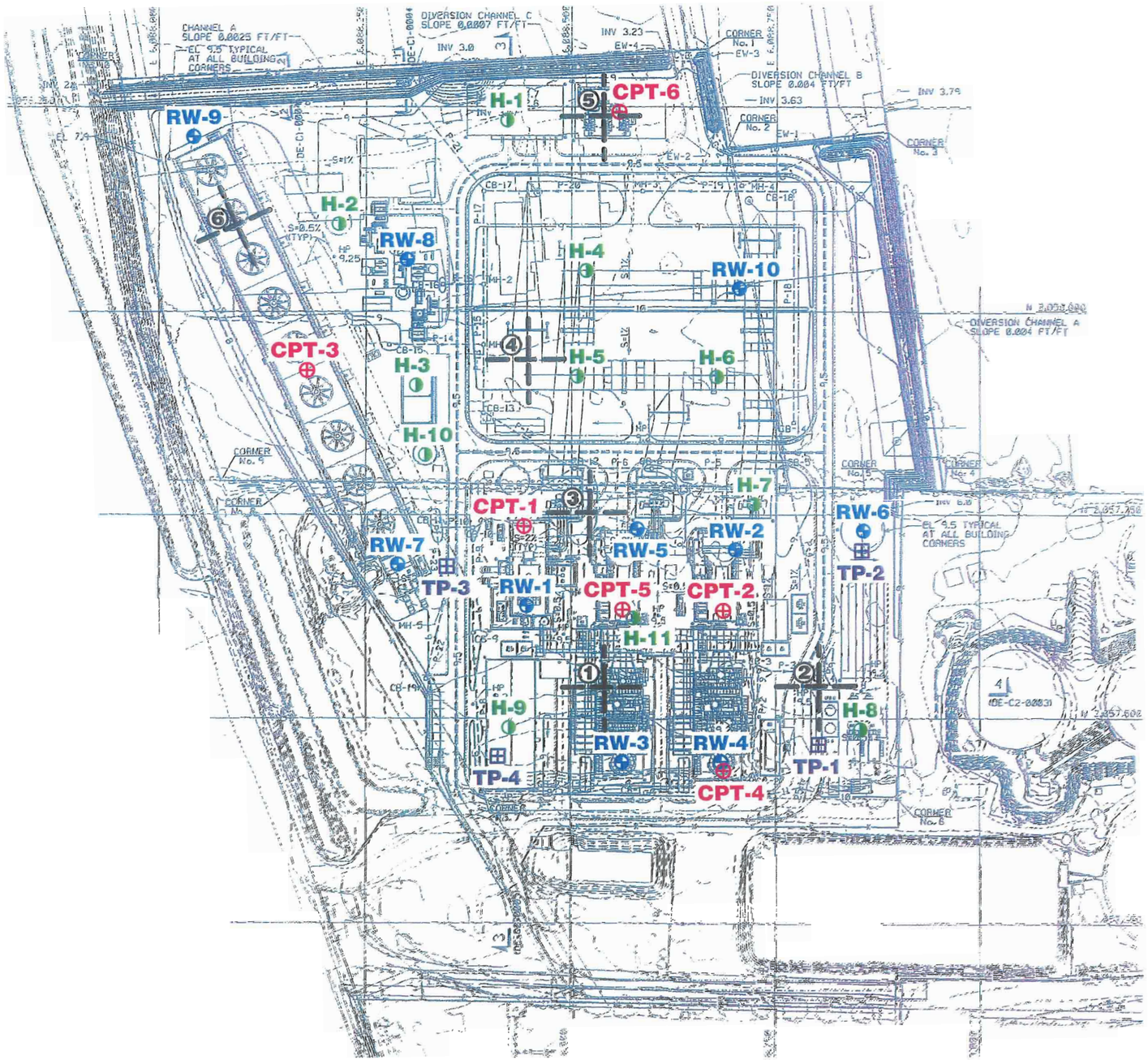
VICINITY MAP

RUSSELL CITY ENERGY CENTER
Hayward, California

TRC Lowney

FIGURE 1
2289-1



LEGEND

- Approximate location of rotary-wash boring
- Approximate location of hollow-stem boring
- Approximate location of cone penetration test
- Approximate location of test pit
- Approximate location of field resistivity test

0 150 ft.

Base by Worley Parsons, dated 11-13-06.

SITE PLAN

RUSSELL CITY ENERGY CENTER
Hayward, California

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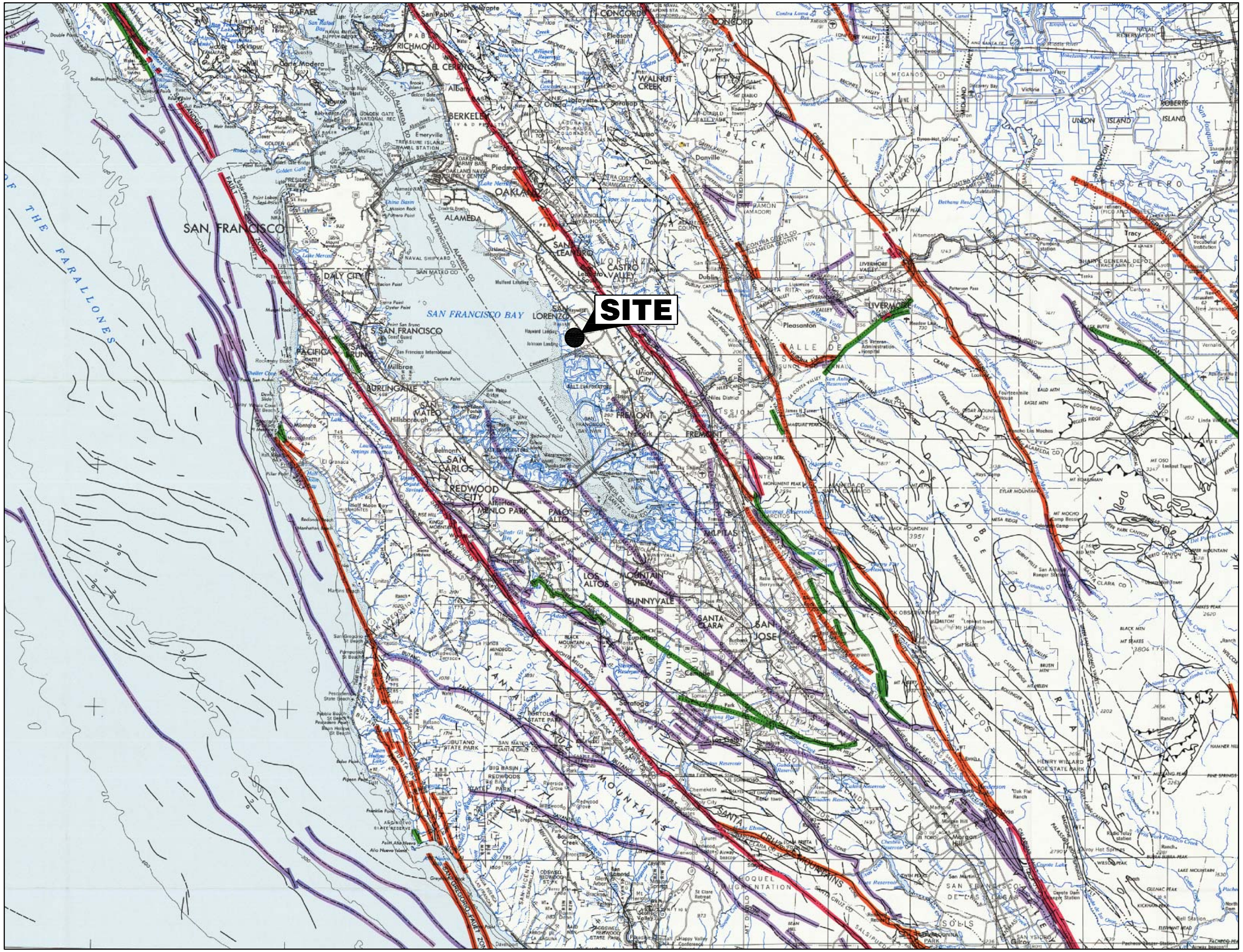
FIGURE 2

2289-1

12/06*EB



Note: Some faults highlighted in purple are not considered active (Holocene Movement) by the State of California.

Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement on Land Offshore[1]	DESCRIPTION
Quaternary	Late Quaternary	Historic	200			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.
		Holocene	10,000			Displacement during Holocene time.[2]
		Pleistocene	700,000			Faults showing evidence of displacement during late Quaternary time.[3,4]
	Early Quaternary		2,000,000			Quaternary (undifferentiated) faults — most faults in this category show evidence of displacement during the last 2,000,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.
Pre-Quaternary	Pliocene		5,000,000			Fault showing evidence of no displacement during Quaternary time or faults without recognized Quaternary displacement.
	Miocene					

Base map is a composite of part the San Francisco 1:250,000 scale map (reference code 37 122–A1–TF–250–00, 1980) and the San Jose 1:250,000 scale map (reference code 37 120–A1–TF–250–00, 1969). For cartographic details, refer to these maps. Bathymetric information is not intended for navigational purposes.

Transverse Mercator Projection 10,000–meter Universal Transverse Mercator grid, zone 10.

Minor corrections and additions to culture by California Division of Mines and Geology 1987.

From: Bortugno & others (1991)

0 10 20 Miles

Scale 1:500,000

REGIONAL FAULT MAP
RUSSELL CITY ENERGY CENTER
Hayward, California

TRC Lowney

FIGURE 3

2289-1

Division of Mines and Geology, James F. Davis, State Geologist

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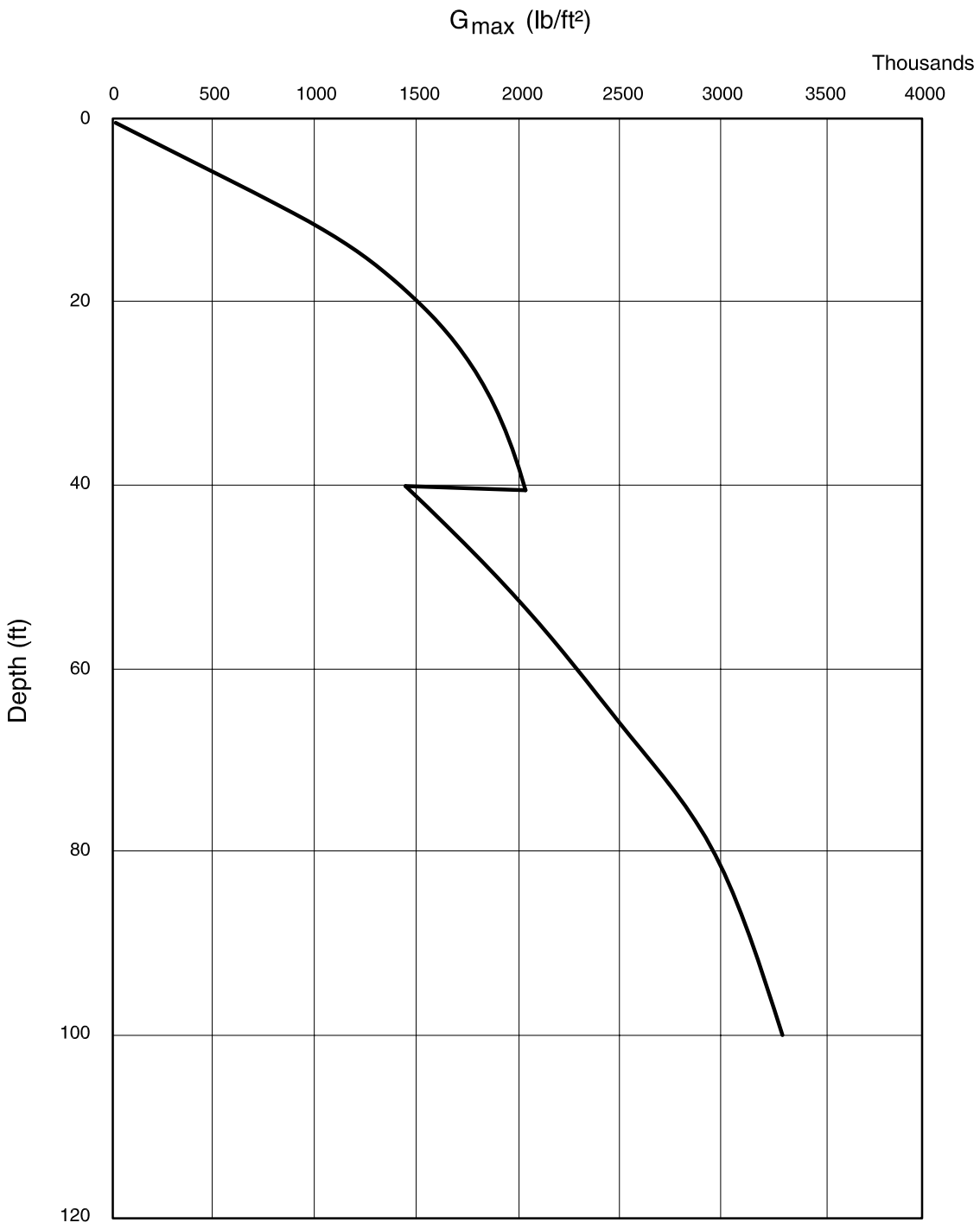

2/07*EB

SMALL STRAIN DYNAMIC SHEAR MODULUS (G_{max})

RUSSELL CITY ENERGY CENTER
Hayward, California

TRC Lowney

FIGURE 4
2289-1