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5.1.2 Tectonic Setting

The tectonic setting of the Salton Trough is complex and is characterized by a transition along the Pacific-North American plate boundary from right-lateral strike-slip motion to ridge-transform divergent motion in the Gulf of California. Tectonic slip is funneled from the wide plate boundary zone in the northern part of the Salton Trough to the spreading centers in the Gulf of California to the south. This complex tectonic setting has created unique geomorphic and structural features in the region.

The Salton Trough marks a shift from a transform plate boundary consisting of the SAF and the other subparallel faults to the mid-ocean ridge-transform divergent plate boundary present in the Gulf of California to the south. This transition begins at the southern end of the Salton Sea (the northern end of the Imperial Valley) where nascent spreading is occurring between the southern end of the SAF and the northern end of the Imperial Fault (Figure 5-2). This spreading center (or pull-apart basin), known as the Brawley Seismic Zone, and the Cerro Prieto geothermal area in Mexico, represent the two northern-most in the series of small spreading centers offset by right-lateral transform faults which characterize the oblique spreading that is extending northward from the Gulf of California. The Brawley Seismic Zone is characterized by high levels of seismicity, as are the Elsinore, San Jacinto, and Imperial faults (Figure 5-1). The southern San Andreas fault has been seismically dormant in historical time, but has ruptured several times in the past 2,000 years (Bennett et al., 1996, Harden, 2004). The project site is located in the western portion of the Brawley Seismic Zone, which is characterized by geothermal and volcanic activity.

Relative motion between the Pacific and North American tectonic plates is accommodated primarily along the San Andreas fault (SAF) in California. However, the SAF is not the sole boundary between the two plates. Instead, relative plate motion is accommodated across a series of subparallel faults that form a broad boundary zone of deformation over 250 km wide. The relative motion between the Pacific and North American Plates in this boundary zone is approximately 50 millimeters per year (mm/yr) in a right-lateral sense (Bennett et al., 1996). The SAF, Elsinore, Imperial, Cerro Prieto, Laguna Salada, and San Jacinto faults are six of the subparallel faults that accommodate most of this motion (see Figure 5-2). While the displacement between the two tectonic plates occurs primarily along these plate-bounding faults, the tectonic regime also includes smaller faults and earthquakes associated with a change in the nature of the Pacific-North American plate boundary.

5.1.3 Stratigraphy

The considerable thickness of sediment that fills the Salton Trough has been deposited by the Colorado River at different locations that have varied through time. Today, the Colorado flows south into Mexico depositing sediments in a large delta at the northern tip of the Gulf of California. However, at least five times in the past 1,000 years, the Colorado River has flowed directly into the Salton trough, creating the freshwater Lake Cahuilla (Thomas and Rockwell, 1996). The lake would fill up to a level of about 13 meters above sea level before it would begin to flow out at the south end of the Salton Trough along the New River to the Gulf of California (Lippincott et al., 2008). The Colorado River would then migrate back to flowing south to the Gulf of California. During these periods, the lake dried up. This river avulsion sequence last occurred in about A.D. 1700, which was the last Lake Cahuilla highstand; the lake subsequently dried up by about 1750 (Lippincott et al., 2008). This change in flow and depositional patterns of the Colorado River is depicted in Figure 5-5. The approximate shoreline of Lake Cahuilla is shown on Figure 5-3: a large portion of the Imperial Valley, including the project site, was under water during lake highstands. This ancient shoreline is visible at several locations on the western side of Imperial Valley.

The Salton Trough was periodically inundated by smaller amounts of water until 1905, when severe flooding caused the Colorado River to overrun its banks and breach existing canal walls. The water flowed into and accumulated in the low-lying Salton Depression in the center of the Salton Trough. The river was not diverted back into its channel until 1907, and the Salton Sea was formed (Harden, 2004). The sea is maintained by water diverted from the Colorado River and by local inflow via the Alamo and other rivers. The Salton Sea highstand was in 1907; this is depicted on Figure 5-3. The Black Rock project site was under water at that time.

The presence of water in the Salton Trough is evident in the Holocene subsurface stratigraphy of the region. This stratigraphy is controlled by the presence or absence of the lake deposits. Therefore the alluvium in the region is a combination of fluvial, lacustrine, and deltaic sediments deposited during various stages of flow of the Colorado River. The fluvial and deltaic sediments are typically composed of sands and silts with a small amount of lean clay while the lacustrine sediments are composed primarily of clays (Thomas and Rockwell, 1996).

5.2 LOCAL GEOLOGIC SETTING

The project site is located adjacent to the southeastern side of the Salton Sea. As previously discussed, the Salton Sea once covered the project site, but has been slowly evaporating and becoming more saline. The surface of the Salton Sea has a current elevation of approximately -227 feet. The Salton Sea has no natural outlet, and its water level today is maintained primarily by the inflow of agricultural runoff from irrigation in the region via several sources including numerous small creeks, and the Alamo and New Rivers. Storm water runoff and effluent are a small component of inflow in addition to agricultural runoff. The Salton Sea shoreline is approximately a half-mile to the northwest of the site.

The Salton Buttes, an aforementioned volcanic feature of the Brawley Seismic Zone, are located to the west of the site along the Salton Sea. Obsidian Butte lies just west of the site and is the western-most of several small, rhyolite and obsidian domes that are arranged in a northeast trend. These domes are collectively called the Salton Buttes and are approximately 16,000 years old (Harden, 2004). Basalt erupted from mid-ocean ridges have been found in the domes, indicating that mid-ocean ridge magma is erupting and rifting is occurring beneath this portion of the Salton Trough (Harden, 2004).

The project site is underlain by late Holocene deposits associated with the presence or absence of ancient Lake Cahuilla. These deposits are typical of those found in the region as described above. The soils are composed of a combination of fluvial, lacustrine, and deltaic sediments. The fluvial and deltaic sediments consist of sands and silts, while the lacustrine deposits consist primarily of clays. Subsurface conditions are described in detail in the following section.

5.3 SUBSURFACE CONDITIONS

5.3.1 Earth Materials

The subsurface field investigation program consisted of rotary wash borings, HSA borings and CPT soundings, allowing the subsurface conditions of the site to be explored to a maximum depth of 85 feet below the ground surface (see Section 3.0 for details of the field investigation program). Subsurface conditions are described and illustrated in the boring and CPT logs (see Appendix A). Although the subsurface soils exhibit some variability both horizontally and vertically, for the purposes of this study they can be divided into layers that are generally identifiable across the site as depicted in the cross sections (Figures 5-6 through 5-11) and further described in the idealized soil profile discussed in Section 7 of this report and presented in Table 7.1. In summary, the project site is underlain by very soft to stiff clays with some interbedded silts and layers of very loose to dense silty sands.

5.3.2 Groundwater

Groundwater was observed in all of the HSA borings that were drilled to a depth of at least 10 feet. Free groundwater was initially encountered at a depth of approximately 8-9 feet across the majority of the site. The groundwater level was re-measured at four HSA boring locations approximately 36 hours after drilling was completed. The depth to groundwater was approximately 5 feet in HSA-14, HSA-3, and HSA-9, and was approximately 7 feet in HSA-20. Based on our observations at the time of the subsurface exploration program for this study, the depth to groundwater at the site generally appears to be approximately 5 feet below existing ground surface. However, variations in groundwater levels and soil moisture conditions can occur as a result of rainfall, runoff, and other factors. Fugro was informed that the alfalfa crop growing on site had not been irrigated for several weeks prior to the commencement of subsurface exploration, which likely had an effect on the observed water levels. Therefore, the soil moisture conditions and groundwater table elevations at this site should be assumed to fluctuate seasonally due to rainfall, on-site irrigation, local agricultural and industrial activities, changes in the water level of the Salton Sea and the adjacent drainage canal, and/or other factors not evident at the time of this study.

6.0 SEISMIC DESIGN CONSIDERATIONS

The proposed Black Rock Units 1, 2 and 3 project area is within a seismically active geologic setting in relatively close proximity to several faults and seismic zones as described in Section 5. Consequently, seismic shaking, fault rupture and liquefaction will need to be addressed as part of the project final design.

6.1 MINIMUM CODE REQUIREMENTS AND SEISMIC RISK

Fugro understands that the project will be designed in accordance with the 2007 California Building Code (CBC 2007), which went into effect on January 1, 2008, and is based on the 2006 International Building Code (IBC). The seismic design provisions of the 2006 IBC are based upon the ASCE 7-05 (ASCE, 2005) seismic design provisions. For seismic design of structures, CBC 2007 requires that the spectral lateral accelerations be based on mapped seismic parameters, site-specific procedures, or probabilistic methods.

The scope of this study was limited to providing recommendations based on mapped seismic parameters. Design spectral response acceleration parameters were assessed using the map-based acceleration parameters and site coefficients. The site coefficients were selected based on the site classification, which in turn were based on the soil properties in the top 100 feet at the site, including any special circumstances, such as the presence of liquefiable or soft soils.

Due to the presence of relatively loose sand layers underlying the project site, liquefaction is likely to occur during the design seismic event, as discussed below in Section 6.3. Therefore, based on the CBC 2007 requirements, the site is classified as Site Class F. However, the use of design spectral response acceleration parameters developed using the mapped parameters presented in CBC 2007 is limited to structures having a fundamental period of vibration (T) equal to or less than 0.5 seconds (ASCE 7-05). To develop design spectral response parameters for short-period structures (i.e., $T \leq 0.5$ seconds) the site coefficients can be assessed using site class as determined by CBC 2007 requirements without considering liquefaction, which for the project area is Site Class D. The mapped seismic parameters and the site coefficients for Site Class D for the current project area are summarized in Table 6-1.

For longer-period structures (i.e., $T \geq 0.5$ seconds), a site response analysis is required by CBC 2007 and ASCE 7-05 to assess the design spectral response acceleration parameters. However, a site response analysis is not part of the scope of this study.

It should be noted that the special requirements of CBC 2007 for Site Class F would not apply if liquefaction mitigation measures are implemented as part of the foundation design. For structures placed on improved soil mitigated to reduce liquefaction potential, the site classification will be Site Class D and the design spectral response acceleration parameters can be assessed using applicable seismic design parameters presented in Table 6-1.

Table 6-1. Summary of 2007 CBC Seismic Design Parameters

	Parameter	Value
Site Location	Latitude	33.1657
	Longitude	-115.6272
Occupancy Category (CBC Table 1604.5)		III
Mapped Acceleration Parameters[1]	S_S	1.5
	S_1	0.6
Site Classification[2]	Site Class	F / D [3]
Site Coefficients and Site Adjusted Acceleration Parameters for Site Class D	F_a	1.0
	F_v	1.5
	S_{MS}	1.5
	S_{M1}	0.9
Design Spectral Response Acceleration Parameters for Site Class D[4]	S_{DS}	1.0
	S_{D1}	0.6
Seismic Design Category		D

Notes:
[1] Coefficients estimated using the USGS calculator available at http://earthquake.usgs.gov/research/hazmaps/design/index.php for cited latitude and longitude.
[2] Site Classification based on the assessed shear wave velocity in top 100 feet.
[3] Site Class F if no liquefaction mitigation measures are implemented. Site Class D if liquefaction measures are implemented.
[4] Values applicable for Site Class F only for structures with fundamental period equal or below 0.5 seconds.

The ordinates for the recommended horizontal ground motion design spectra for 5 percent damping based on the CBC 2007 mapped acceleration parameters are included in Table 6-2 and presented graphically in Figure 6-1. The values for Site Class F are presented only for periods up to 0.5 seconds. A site response analysis using site-specific ground motion parameters is required by CBC 2007 and ASCE 7-05 to assess the spectral acceleration values for periods above 0.5 seconds for Site Class F.

Table 6-2. CBC2007 Recommended Horizontal Ground Motion Design Spectra

Period (sec)	Site Class F Spectral Acceleration (5% Damping)	Site Class D Spectral Acceleration (5% Damping)
PGA	0.400	0.400
0.05	0.675	0.675
0.10	0.900	0.900
0.12	1.000	1.000
0.15	1.000	1.000
0.20	1.000	1.000
0.30	1.000	1.000
0.50	1.000	1.000
0.60	A site response analysis using site specific ground motion parameters is required by CBC 2007 and ASCE-7-05 to assess the spectral acceleration values for periods above 0.5 seconds.	1.000
1.00		0.600
1.50		0.400
2.00		0.300
3.00		0.200
4.00		0.150
5.00		0.120

6.2 FAULT RUPTURE HAZARD

The proposed project site is located in the western portion of the Brawley Seismic Zone. This structural depression (or pull-apart basin) is a zone of transition between the northwest end of the Imperial fault, and the southwest end of the San Andreas fault (see Section 5). The zone's high seismicity is defined by microseismicity that appears to be due to magmatic intrusions caused by nascent spreading in the area, and aftershocks following earthquakes on nearby faults. The Brawley Seismic Zone is characterized by earthquake swarms, generally less than magnitude 3 or 4 (see Figure 5-2). Recent fault parameters characterize the Brawley Seismic Zone as a special case area of background seismicity, and not as a strike slip fault, so it is not modeled as a fault source by CGS (CGS, 2008). It is likely that the slip in this zone is being translated into regional subsidence and geothermal activity.

As discussed previously, the Brawley Seismic Zone is defined by epicenters of microseismicity or aftershocks following earthquakes on adjacent active faults rather than from geologic mapping of surface ruptures and geomorphic features. There are no known faults that reach the ground surface within the Brawley Seismic Zone. Thus, the site is not within an Alquist-Priolo Earthquake Fault Zone (Hart and Bryant, 1997). Although stress is being transferred to the Brawley Seismic Zone from nearby active faults, historic and microseismic records indicate the stress is released gradually through relatively constant earthquake swarm activity as described above. In addition, our aerial photographic review did not identify lineaments or other linear geomorphic features that trend toward the site, and the exploration we performed suggests relatively uniform stratigraphy that does not appear to have significant vertical offset. Therefore, based on the above information, we believe the potential for ground rupture at the site because of faulting in the Brawley Seismic Zone is considered to be low.

6.3 LIQUEFACTION HAZARD

Liquefaction is the loss of strength that can occur in loose, saturated sand during seismic shaking. The susceptibility of a granular soil to liquefaction is a function of the gradation, density, and fines content of the soil. Susceptibility to liquefaction decreases with respective increases in: a) distribution of grain size, b) soil density, c) fines content, d) clay-size fraction of the fines, and e) the age of the deposit. The soils under the area of the proposed facility are geologically recent, with extensive interbedded deposits of loose to medium dense, saturated sands that are susceptible to liquefaction.

There are a number of potential consequences when liquefaction occurs. When the shaking continues after the onset of liquefaction, liquefaction can produce a number of ground effects (e.g., sand boils, settlement, lurching, and lateral displacement). Liquefaction also can cause a loss of bearing capacity of shallow foundations, and lateral ground spreading. In general, the longer the duration of strong shaking after the initiation of liquefaction, the greater the consequences.

Evidence of liquefaction is known to have been observed in the general area of the site during past earthquakes. Potentially liquefiable soils are typically relatively loose sandy layers located within fine grained soils encountered underlying the site. There is no surficial evidence of historic occurrence of liquefaction at the site; however, due to the ongoing agricultural use of the site, such evidence would not be likely to remain visible for a significant length of time following any recent earthquakes.

Liquefaction analyses have been performed for this study using the CPT data for Design Level Earthquake (DLE). The DLE was characterized as having a peak ground acceleration (PGA) of 0.40g and moment magnitude of 6.5. Since the scope of this study did not include evaluation of ground motions at the site using site-specific deterministic or probabilistic hazard assessment, the selection of PGA was based on recommendations provided in ASCE 7-05. ASCE 7-05 recommends that a PGA equal to $S_{DS}/2.5$ be used for liquefaction triggering analyses in absence of site-specific ground motion study, where S_s is the mapped acceleration parameter evaluated from CBC 2007. The S_{DS} of 1.0 was evaluated for the project site as presented in Table 6-1. It should be noted that the CBC based PGA value of 0.40g is somewhat lower than the PGA value of 0.45g estimated using the CGS online interactive toolset (http://redirect.conservation.ca.gov/cgs/rghm/pshamap/pshamain.html) for the Black Rock project location and Site Class D for the 475-year return period presented. The DLE values based on CBC 2007 are similar to the 475-year return period events; however, small differences are to be expected since the CBC-based DLE is not directly based on the 475-year return period event.

The moment magnitude of 6.5 was selected based on the most recent evaluation of the expected maximum magnitude for Brawley Seismic Zone as presented by the CGS (2008). Although the Brawley Seismic Zone has been characterized as a zone of earthquake swarms or microseismicity based on observed recent seismic activity, it is considered capable of producing larger seismic events than just the earthquake swarms that have been observed in recent times. Furthermore, the site is also subject to ground accelerations generated by major seismic events on other faults in the area, for example the San Andreas fault. It is important to note that the source of the seismic activity that generates the ground acceleration experienced at a site need not be located at, or even in close proximity to, the site.

The liquefaction analyses were performed per the NCEER procedure (Youd , Iriss, 2001) over the entire depth of exploration assuming the groundwater table was at about 5 feet below the existing ground surface. Ground settlement due to liquefaction was calculated for the DLE using volumetric strain relationships by Ishihara and Yoshimine (1992). The results of these analyses for selected CPTs are presented graphically on Figures 6-2 through 6-6 (Subsurface Liquefaction Cross Sections) corresponding to the geologic cross-section locations shown on Figure 5-6. A key with definitions of symbols for the CPT liquefaction sections is presented on Figure 6-2. The results of the liquefaction triggering analysis for all CPTs are presented in Appendix C.

Figures 6-3 through 6-6 indicate that a significant portion of sandy materials in the upper 30 to 40 feet will likely liquefy during the design level event. These figures also indicate that some of the deeper sandy layers could also potentially liquefy during the DLE. Figures 6-2 through 6-5 also present the estimated liquefaction-induced settlements. According to Seed (1979), at most of the sites where some surface evidence of liquefaction has been observed in the field, the critical layer in which liquefaction is believed to have occurred has been located at depths of less than 45 feet and the depth of the groundwater table has been less than 15 feet. However, Seed (1979) states that this should not be construed to indicate that liquefaction cannot be induced at larger depths due to earthquake shaking.

In the context of this reasoning and published recommendations by the Southern California Earthquake Center (Martin and Lew, 1999), ground settlement due to liquefaction has been calculated using the CPT data for the design level earthquake event using volumetric strain relationships by Ishihara and Yoshimine (1992). Estimated total settlements for the design level event are presented in Table 6-3, with respect to CPT locations with at least 50 feet of penetration.

Table 6-3. Summary of Liquefaction Settlement Estimates

CPT	Estimated Total Settlement (inches)[1]	CPT Depth (feet)	Proposed Nearby Structures
C-102	5.5	50	Injection Area - Unit 3
C-104	5.0	75	Miscellaneous Equipment - Unit 3
C-105	6.0	75	Cooling Tower - Unit 3
C-106	5.0	50	Control Building
C-107	6.0	75	Turbine/Generator - Unit 3
C-108	6.5	75	Cooling Tower - Unit 3
C-109	6.5	75	Miscellaneous Equipment - Unit 2
C-110	6.5	75	Miscellaneous Equipment - Unit 1
C-111	8.5	75	Transformer - Unit 2
C-112	6	75	Turbine/Generator - Unit 2
C-113	7.5	75	Cooling Tower - Unit 2
C-114	7.5	75	Cooling Tower - Unit 2
C-115	6.5	75	Transformer - Unit 1
C-116	6.5	75	Turbine/Generator - Unit 1
C-117	6.5	75	Cooling Tower - Unit 1
C-118	6.5	75	Cooling Tower - Unit 1
C-120	5	50	Injection Area - Unit 1
C-121	6	50	Storm Water Retention Basin

Notes:
[1] Estimated settlements rounded up to the nearest half-inch.

Observations of liquefaction-induced settlements from past earthquakes throughout the world have shown that settlement from liquefaction is difficult to predict and can vary significantly over relatively short distances. Current SCEC guidelines recommend that differential settlement from liquefaction should be conservatively estimated as one-half to two-thirds the total settlement from liquefaction where sites have variable stratigraphy. Because the site is relatively flat and subsurface conditions are relatively consistent, we recommend that differential settlements for design of structures around the site be estimated as about one-half of the total settlement from liquefaction.

6.4 LATERAL SPREADING POTENTIAL

Estimating lateral movements resulting from seismic events is highly uncertain. Youd and Bartlett (2002) have developed empirical procedures for estimating lateral movements. Their empirically-derived procedures for estimating lateral movements depend on earthquake magnitude, distance between the site and seismic event, thickness of liquefied layer, ground slope or ratio of free-face height to distance between free-face and structure, fines content, the average particle size of the material forming the liquefied layer, and the SPT N-value. The proposed project site is relatively level and is not adjacent to any significant depressions (e.g. deep channels, river basins, etc.). Although subsurface conditions at the site consist of silty and sandy soils that are susceptible to liquefaction, we believe the risk of lateral movement at the site during a significant seismic event is very low.

6.5 SURFACE EFFECTS FROM LIQUEFACTION

Liquefaction is often accompanied by the development of sand boils and fissures, herein termed surface effects. Ishihara (1985) produced an empirical procedure to estimate the thickness of the overlying non-liquefiable layer to prevent level-ground liquefaction-related damage from surface effects. This procedure was later validated in a study by Youd and Garris (1995), where it was concluded that the procedure is not appropriate for assessing surface affects from liquefaction at level ground sites subject to lateral spreading. However, preliminary field data from the recent earthquakes (1999 Kocaeli [Turkey], 1999 Chi-Chi [Taiwan]) suggest that the Ishihara procedure may not always be capable of predicting the occurrence of surface effects.

Since the potential for lateral spreading at the project site is very low (see Section 6.4), the Ishihara procedure was used to estimate the potential for surface effects occurring at the site as a result of liquefaction. Review of subsurface conditions throughout the project area and application of the Ishihara procedure suggests that the site has a potential of experiencing surface effects if liquefaction at the site were to occur. Surface manifestations of liquefaction could include sand boils or ground fissures.

7.0 FOUNDATION RECOMMENDATIONS

7.1 INTRODUCTION

Each unit of the proposed facility will consist of a Generating Area and a Production Area. The structural and mechanical components of each respective area will be identical. Based on preliminary equipment loading data provided by CalEnergy, we understand that the largest structural loads will be in the Generating Areas at the cooling tower and the turbine/generator structures. Although plans showing the dimensions of all the structures and foundations had not been prepared at the time of this report, Fugro has assumed that the various mechanical equipment and structures will be grouped as required and supported on reinforced concrete footing and/or mat foundations. The three units will also share a number of facilities that include a control building, fire water and purge water storage tanks, and a storm water retention pond.

Fugro understands that it is planned to raise existing grades in the proposed structure and road areas a minimum of 1.5 feet above existing grade in order to achieve proper site drainage conditions. This will create a pad of engineered fill material beneath most structures and pavements and will help to mitigate some of foundation design and construction issues associated with the soft ground conditions at this site.

Recommendations are presented below for shallow and deep foundations together with estimated settlements under static and seismic (i.e., liquefaction) conditions. The structural designers should select the appropriate foundation type based on the sensitivity of the structures to the estimated settlements and the level of risk that CalEnergy is willing to accept in the performance of the facility.

7.1.1 Design Requirements and Considerations

In general, for satisfactory foundation performance, the selected foundation design must meet the following criteria:

1. Applied structural loads transmitted to the soils through shallow or deep foundations should not exceed the ultimate bearing capacity (which is a function of the shear strength) of the foundation soils. Moreover, the applied bearing pressures should not exceed an allowable bearing pressure determined by dividing the ultimate bearing capacity by an appropriate factor of safety.
2. The settlements due to compression and consolidation of the underlying soils must be within tolerable limits of the structure.

Our assessments of and recommendations for bearing capacity and settlement are presented in the following sections. Any major relocation of equipment or any significant increase in structural loads or foundation dimensions could result in a revision of these recommendations. Such changes should be reviewed by Fugro prior to finalizing design or implementing construction.

7.1.2 Foundation Loads

The expected foundation loads for major structures, such as cooling towers and turbine/generator structures, were estimated based on the preliminary information provided by CalEnergy. Based on the available data, the highest foundation loads are expected to be at the generator/turbine structures, which are expected to weigh about 1,000 kips each and exert an average pressure of about 600 pounds per square foot (psf) over a foundation area of 60 feet by 30 feet, and the cooling towers, which are expected to weigh about 7,600 kips each and exert an average pressure of about 500 psf over a foundation area of 282 feet by 54 feet.

For large-area foundations, the foundation stresses will be transferred relatively deep into underlying soils. To evaluate the subsurface stress distribution and estimated settlement under large-area foundations, the computer program UNISETTLE (Unisoft Ltd., 2002) was used. The Boussinesq elastic stress distribution option in UNISETTLE was selected for the stress analysis. The Boussinesq theory assumes the subsurface is an isotropic, homogeneous, linear elastic half-space.

7.1.3 Idealized Soil Profile

Foundation recommendations presented in this section were developed using the idealized subsurface conditions for major soil groups as listed in Table 7-1.

Table 7-1. Idealized Subsurface Profile for Foundation Recommendations Evaluation

Soil Unit	Depth to the Top of Layer (feet)	Total Unit Weight (pcf)	Apparent Cohesion / Undrained Shear Strength (psf)	Friction Angle (degrees)	Undrained Residual Strength of Liquefiable Soils (psf)
New Compacted Fill	0	120	100	35	-
Shallow Interbedded Clays, Silts and Sands	3	115	800	0	-
Loose to Medium Dense Silty Sands	10	120	0	30	250
Clays and Silts	15	120	1,500	0	-
Loose to Medium Dense Silty Sands	18	120	0	32	550
Clays and Silts	25	115	1,000	0	-
Medium Dense to Dense Silty Sands	35	125	0	35	-
Clays and Silts Interbedded with Sands	47	115	1,000 to 1,500	0	-

The shear strength parameters for the idealized subsurface profile were assessed using the data collected during the field exploration (CPT resistance values and SPT blowcounts), and results of the shear strength laboratory testing.

7.2 SHALLOW FOOTINGS

Proposed lightly-loaded ancillary buildings and selected equipment may be supported on shallow foundations. The use of shallow foundations is contingent on the assumption that the risks of the anticipated differential static and liquefaction-induced seismic settlements are acceptable and can be accommodated in the design of the structures. We recommend minimum footing widths of 24 inches for individual square footings and 18 inches for continuous strip footings. The footing thickness should be determined by a structural engineer, but should not be less than 12 inches.

As discussed in subsequent sections of this report, overexcavation and replacement of existing surficial soils will be performed in all foundation areas of the site and grades will be raised by at least 1.5 feet above existing elevations. Consequently, all footings should be founded on at least 12 inches of compacted structural fill at a minimum embedment of 24 inches, relative to the adjacent finished grade or slab elevation, whichever is lower. Where necessary, areas of overexcavation should be deepened to achieve the minimum recommended depth of embedment and thickness of structural fill beneath footings.

Existing surficial soils at the site exhibit a moderate expansion potential; however, the proposed overexcavation and replacement of existing surficial soils, together with proper site drainage, is expected to mitigate the potential for expansive soil damage to shallow foundations.

7.2.1 Allowable Bearing Pressures

Shallow footings should be proportioned for dead load plus probable maximum live load so that the maximum net bearing pressure does not exceed the maximum allowable net bearing pressure. The maximum allowable net bearing pressure of 1,500 psf should be used for foundation design, and provides a factor of safety greater than 2. A one-third increase can be applied to maximum bearing pressures for wind-loads. Because of the potential for liquefaction of the soil below the footings, the recommended maximum bearing pressures should not be increased for seismic loads.

7.2.2 Estimated Settlement

7.2.2.1 Static Settlement

Settlement estimates were based on the data provided by CPTs located in the project area and the idealized soil profile presented in Section 7.1.3 using standard settlement calculations. Estimated total settlements are presented in Table 7-2. Approximately two-thirds of the estimated static-load settlement is expected to occur during construction as loads are applied. The remaining estimated static-load settlement is expected to occur within 3 to 6 months of the application of the load. These estimates are based on the recommended minimum thickness of new structural fill below the bottom of the footings.

Table 7-2. Summary of Settlement Estimates for Shallow Footings

Foundation Type and Size	Contact Pressure (psf)	Estimated Total Static Settlement (inches)	Estimated Differential Static Settlement[1] (inches)	Estimated Total Liquefaction Induced Settlement (inches)	Estimated Differential Liquefaction Induced Settlement [1] (inches)
Strip footings (width 2 to 3 feet)	500	0.5 - 1	< 0.5	6 - 8	3 - 4
	1,000	0.8 - 1.2	< 0.8		
	1,500	1 - 1.5	0.5 - 1		
Individual footings (less than 5 feet square)	500	0.5 - 1	< 0.5	6 - 8	2 - 3
	1,000	1 - 1.5	0.5 - 1		
	1,500	1.5 - 2	0.5 - 1		
Individual footings (5 to 15 feet square)	500	1 - 1.5	0.5 - 1	6 - 8	3 - 4
	1,000	1.5 - 2	0.5 - 1		
	1500	2 - 2.5	1 - 1.5		

Notes: [1] Differential settlements are estimated over a distance of 30 feet

Differential settlements between footings may result from variations in subsurface conditions, differences in footing size, and variations in loading conditions. The differential settlements were estimated at about one-third to one-half the total settlement, depending on the size of the foundation.

Estimated total and differential settlements for shallow footings are based on the assumption that the foundations are not immediately adjacent to the cooling towers, turbine/generator structure or other heavy loads. If the subsurface soils beneath the footings are also subjected to stresses from adjacent heavy structures, then settlements beneath the buildings may be greater than the estimated settlements presented in the preceding paragraphs.

7.2.2.2 Liquefaction-Related Settlement

Although large settlements are not expected to occur due to static loads, additional settlements may occur if loose sand layers liquefy during a large earthquake. As summarized in Section 6.0, the submerged granular soils are susceptible to liquefaction and additional settlement may occur due to liquefaction during an earthquake. Seismically induced total settlements could be as much as 6 to 8 inches over the site, if earthquake-induced ground accelerations produce extensive liquefaction. Those settlements could be quite variable and create significant differential settlements over limited distances; therefore, the settlements due to liquefaction should be expected to vary laterally. The differential settlements can be estimated at about one-half of the total settlement. Estimated total and differential liquefaction induced settlements for the shallow footings are presented in Table 7-2.

7.2.3 Lateral Sliding and Passive Resistance

The sliding resistance generated through a soil/concrete interface can be computed by using an allowable coefficient of friction of 0.25 and the applicable structural load allowed by the

2007 CBC. A one-third increase may be applied to the allowable coefficient of friction for short-term loading.

For foundation elements bearing against compacted fill, the allowable passive earth resistance (neglecting the upper 1-foot) may be estimated using an equivalent fluid weight of 150 pcf. The allowable passive pressure may be used in combination with the frictional resistance. A one-third increase may be applied to the allowable passive pressure for short-term loading.

7.3 MAT FOUNDATIONS

Larger facilities such as the cooling towers and turbine/generator structures may be supported on mat foundations. The use of mat foundations is contingent on the assumption that the risks of the anticipated differential static and liquefaction-induced seismic settlements are acceptable and can be accommodated in the design of the structures. To limit the total and differential liquefaction induced seismic settlements, ground improvement measures can be implemented, as discussed in more detail in Section 7.5.

As discussed in subsequent sections of this report, overexcavation and replacement of existing surficial soils will be performed in all foundation areas of the site. Consequently, all mat foundations should rest on at least 12 inches of compacted structural fill at a minimum embedment of 24 inches, relative to the adjacent finished grade or slab elevation, whichever is lower. Where necessary, areas of overexcavation should be deepened to achieve the minimum recommended depth of embedment and thickness of structural fill beneath mat foundations.

7.3.1 Allowable Bearing Pressures

Mat foundations should be proportioned for dead load plus probable maximum live load so that the maximum net bearing pressure does not exceed the maximum allowable net bearing pressure. A maximum allowable net bearing pressure of 1,500 psf may be used for foundation design, and provides a factor of safety greater than 2. A one-third increase can be applied to maximum bearing pressures for wind-loads. Because of the potential for liquefaction of the soil below the footings, the recommended maximum bearing pressures should not be increased by one-third for seismic loads.

7.3.2 Estimated Settlement

7.3.2.1 Static Settlement

Estimated total settlements are presented in Table 7-3. Settlement estimates were based on soil profiles and data as interpreted from the CPTs located in the proposed cooling tower and turbine/generator locations and consolidation test results from samples obtained from the borings. Estimated contact pressures are based on structural loads and assumed foundations dimensions as interpreted from data provided by CalEnergy and described in Section 7.1.2 of this report. About one-third of the estimated static-load settlement is expected to occur during construction as loads are applied. The remaining estimated static-load settlement is expected to occur within six to nine months of the application of the load.

Table 7-3. Settlement Estimates for Selected Major Structures on Mat Foundations

Mat Foundation Location	Estimated Contact Pressures[1] (psf)	CPTs Used in Analysis	Estimated Total Static Settlement (inches)	Estimated Differential Static Settlement (inches)[2]	Estimated Total Liquefaction Induced Settlement (inches)	Estimated Differential Liquefaction Induced Settlement[3] (inches)
Cooling Tower - Unit 1	500	CPT-117 CPT-118	2.5 - 3	1.5 - 2	6.5 - 7	3 - 3.5
Cooling Tower - Unit 2	500	CPT-113 CPT-114	2.8 - 3.2	1.8 - 2.2	8 - 8.5	4 - 4.5
Cooling Tower - Unit 3	500	CPT-105 CPT-108	2.5 - 3	1.5 - 2	6 - 6.5	3 - 3.5
Turbine/Generator Unit 1	600	CPT-116	2 - 2.5	1 - 1.5	6.5	3.5
Turbine/Generator Unit 2	600	CPT-112	2 - 2.5	1 - 1.5	6	3
Turbine/Generator Unit 3	600	CPT-107	2 - 2.5	1 - 1.5	6	3

Notes:
[1] Estimated based on preliminary information provided by CalEnergy (Figure 1-3)
[2] Estimated differential settlement by considering the variation in soil conditions and stress distribution between the center and the edge of the mat foundation
[3] Differential settlement between the center and the edge of the mat foundations is estimated as one-half of the total liquefaction induced seismic settlement.

Differential settlements may result from variations in subsurface conditions and variations in loading conditions. The differential settlements were estimated by comparing the loading conditions underneath the center point and the corner point of the mat foundation.

7.3.2.2 Liquefaction-Related Settlement

Although large settlements are not expected to occur due to static loads, additional settlements may occur if looser sand layers liquefy during a large earthquake. As summarized in Section 6.0, the submerged granular fills are susceptible to liquefaction. Thus, additional settlement may occur due to liquefaction during an earthquake. Seismically induced total settlements could be as much as 6 to 8 inches over the site, if earthquake-induced ground accelerations produce extensive liquefaction. Those settlements could be quite variable and create significant differential settlements over limited distances; therefore, settlements due to liquefaction should be expected to vary laterally. The differential settlements can be estimated at about one-half of the total settlement. Estimated total and differential liquefaction induced settlements for the shallow footings are presented in Table 7-2.

7.3.3 Lateral Sliding and Passive Resistance

The sliding resistance generated through a soil/concrete interface can be computed by using an allowable coefficient of friction of 0.25 and the applicable structural load allowed by the

2007 CBC. A one-third increase may be applied to the allowable coefficient of friction for short-term loading.

For foundation elements bearing against compacted fill, the allowable passive earth resistance (neglecting the upper 1-foot) may be estimated using an equivalent fluid weight of 150 pcf. The allowable passive pressure may be used in combination with the frictional resistance. A one-third increase may be applied to the allowable passive pressure for short-term loading.

7.4 GROUND IMPROVEMENT

As discussed in preceding sections of this report, static settlements in the range of 2 to 3 inches were estimated for mats supporting structures such as the cooling tower and generator/turbine and 6 to 8 inches of total settlement due to liquefaction were estimated for the design seismic event, with possible differential settlements on the order of 3 to 4 inches. Therefore, seismic loading will probably control foundation design of the structures. For structures that are unable to tolerate settlements of this magnitude, various methods of deep ground improvement, such as stone columns or Cement Deep Soil Mixing (CDSM), could be used to mitigate liquefaction beneath the foundations. A treatment depth of approximately 50 feet below existing ground surface is considered appropriate for this site.

Stone columns are a ground improvement technology involving the replacement of weak soils with columns of compacted gravel. The columns are typically about 3 feet in diameter and constructed in a grid pattern at a spacing of about 7 to 8 feet on-center, although the spacing can be increased or decreased to suit site conditions and project requirements. The treatment area would extend horizontally outside the perimeter of the foundation a distance of approximately one-third to one-half of the vertical depth of treatment. The primary use of stone columns is to densify loose granular soils and increase their strength. Therefore, they are best suited for use at sites where foundation soils are predominantly loose to medium dense sands or silty sands with relatively low fines content (i.e., silt and clay content) and no significant silt or clay strata. As observed during the subsurface exploration for this study and described previously in this report, the stratigraphy of the Black Rock site is predominantly clay with interbeds of loose to medium dense sands. Although stone columns will help mitigate the liquefaction potential under proposed structures at this site, they could experience vertical deformation (i.e., compression) during an earthquake due to lateral compression of soft fine-grained soils, which would result in surface settlement for structures resting on the columns. However, CalEnergy has used stone columns on previous geothermal plant projects in the area and is therefore familiar with the cost and performance to date of this technology in ground conditions similar to those at the Black Rock site.

CDSM is a ground improvement technology that consists of drilling a series of overlapping borings in which cement slurry is blended in situ with the subsurface soils to create columns of soil-cement having higher strength and lower compressibility than the untreated native soils. Rebar can be placed in the soil-cement columns to transfer lateral loads from structure foundations. In general, a network of interconnected soil-cement columns resulting from the CDSM process is structurally superior to a network of stone columns in that it is stiffer, less compressible, and offers superior lateral load transfer for foundations. This technology has been used successfully for over 30 years in ground conditions similar to those at the Black Rock site.

For CDSM, the treatment area would extend horizontally outside the perimeter of the foundation a distance equal to about one-third of the vertical depth of treatment. The effective treatment volume beneath a structure is a function of the treatment area and the depth of treatment. Multiplying the treatment area by the treatment depth gives the total volume, from which the effective treatment volume is obtained by taking approximately 25 to 30 percent of the total volume. The distribution of ground improvement points (i.e., the treatment pattern) within the treatment area is performed according to one of a variety of possible geometric patterns based on site conditions, structural support requirements, and other design criteria. The actual treatment pattern should be determined by consultation between the project structural and geotechnical engineers during the design phase.

7.5 DEEP FOUNDATIONS

If any of the anticipated settlements, either static, liquefaction-induced seismic, and/or differential, are not acceptable for sensitive equipment, a deep foundation system, such as driven piles, may be used for support of such structures. Alternatively, ground improvement methods may be employed to mitigate the liquefaction potential as described in Section 7.4. Pile capacities and special considerations associated with use of pile foundations at the site are presented in the following section.

7.5.1 Axial Capacity

Axial capacities were based on methods presented in the American Petroleum Institute (API, 2000) as coded into the program APile Plus, Version 4.0 (Ensoft, 2005). The idealized subsurface profile presented in Section 7.1.3 was used in developing axial pile capacity recommendations.

Axial Capacity under Static Loads. Figure 7-1 presents the ultimate axial capacity curves developed for 12-, 14- and 16-inch square driven concrete piles. The axial capacities presented in the figure are derived primarily from the frictional resistance of the subsurface materials. End bearing will also contribute to the axial load capacity if the pile tips are founded in a sand layer. Typically, factors of safety of 2.0 are applied to design dead loads to assess the required ultimate axial pile capacity. A minimum depth of embedment can then be assessed using the curves presented in Figure 7-1. Selection of minimum embedment in this manner should limit the vertical movement of pile head under design load to less than one-half inch.

Uplift Capacity. Driven piles can be used to resist intermittent uplift loads using skin friction. The allowable uplift resistance of the piles was estimated based on 50 percent of the frictional capacity of the pile. The allowable uplift resistance curves for 12-, 14- and 16-inch square driven concrete piles are presented in Figure 7-2.

Seismic Considerations and Down-drag Loads. The presence of liquefiable soils within the zone of pile embedment will have two effects: 1) reduction of axial pile capacity during seismic loading due to the loss of strength in liquefiable soils; and 2) downdrag loads applied on the pile as a result of liquefaction-induced soil settlements developing following the seismic loading.

To avoid significant loss of axial capacity during seismic loading, the pile tips should not be placed in liquefiable soils, i.e., the pile tip elevation should be below about elevation -265 feet. The medium dense to dense silty sands extending from about elevation -260 to -275 feet are generally not expected to fully liquefy; however, some development of excess pore pressure may occur. The liquefaction triggering analyses indicate that limited sand zones below elevation -265 feet might be susceptible to liquefaction and there is a potential for limited liquefaction-induced additional settlements on the order of 1 to 2 inches to occur even for structures on pile foundations.

As the excess pore pressures generated during seismic loading in liquefiable soils start to dissipate, settlement will occur and the soil overlying the liquefiable soil zone will move downward relative to the piles. This downward movement will result in down-drag forces (i.e., negative axial capacity) on the piles due to the skin friction between the piles and the soil, which in turn will cause a reduction in the load-carrying capacity of the piles. Therefore, to account for both the down-drag forces and the loss of positive axial capacity along the same portion of the pile, the ultimate axial pile capacities should be reduced by approximately 110, 130, and 150 kips for 12-, 14- and 16-inch square piles, respectively.

Because liquefaction-related settlement and the resulting down-drag forces occur in a matter of minutes to hours after the design seismic event, it is appropriate to consider these forces as static loads, i.e., there is no need to combine down-drag forces with the inertial forces caused by ground shaking.

7.5.2 Lateral Capacity

Lateral load pile evaluations were performed using the computer program LPile Plus Version 5.0 (Ensoft, 2008) which is based on a soil resistance-pile deflection model (p-y analysis). LPile Plus was used to estimate the lateral load capacity versus head deflection and maximum moment. To account for potential strength loss in soils during the design level earthquake, a reduction factor for lateral soil resistance was applied in the form of p-multipliers. The evaluated reductions include the loss of soil resistance due to development of excess pore pressures, soil liquefaction, and loss of soil strength due to cyclic degradation.

The relationships between lateral load capacity and maximum moment versus pile head displacement are presented in Figures 7-3 and 7-4 for fixed-head and free-head piles, respectively. The fixed-head pile case will apply when pile head is connected to a mat foundation of sufficient size and stiffness to limit the pile head rotation. The free-head pile case will apply for conditions where no such restraint will be provided by the pile head connection. The minimum pile embedment depth recommended to achieve the presented lateral pile capacities is about 35 to 40 feet.

No factor of safety has been applied to the estimated loads or deflections. Due to the interbedded distribution of sand within the predominantly clayey stratigraphy in the upper 60 feet, the presence of thin liquefiable layers was neglected for design purposes. Depth of fixity calculations are presented in Appendix E.

If additional lateral support is required, the piles may be augmented with stone columns or CDSM to depths of approximately 20 feet around the foundation perimeter. If such ground improvement measures are implemented, lateral load capacities for driven piles may be evaluated from the data presented in Figures 7-5 and 7-6 for fixed-head and free-head piles, respectively.

7.5.3 Pile Spacing and Pile Group Effects.

For closely spaced piles, the interaction between individual piles may result in a reduction of both axial and lateral pile capacity. Therefore, it is recommended that the piles are not spaced closer than 5 pile diameters on-center. If piles are to be spaced closer than 5 times the pile section diameters, Fugro should review the recommended pile capacity curves and lateral pile displacements curves and provide additional recommendations, as needed.

7.6 MISCELLANEOUS

7.6.1 Drilled Shaft Foundations

Drilled shafts (i.e., piers) may be used for foundation support of above-ground piping. Because no information as to the size of the piers and required capacities were available at the time of the preparation of this report, we are providing generalized recommendations for evaluation of stability of drilled piers under axial and lateral loads.

The allowable axial capacity was evaluated by taking into account skin friction around the pile perimeter and can be estimated from the following expression:

Allowable Axial Capacity (kips) = $0.1\ z^{2}D$

where: z = depth of embedment below final grade in feet
D = pier diameter in feet

This expression incorporates a safety factor of 2 and is based on the assumption that the drilled piers will be embedded no greater than 15 feet below final grade.

The minimum embedment depth required to maintain the lateral stability should be estimated using requirements set forth in CBC (2007) Section 1805.7.2.1 and 1805.7.2.2, and an allowable lateral bearing pressure of 200 psf/foot below grade. If equipment supported by the drilled piers is sensitive to lateral displacement allowable lateral bearing pressure should be reduced by one half. An increase of allowable lateral bearing pressure of one-third is permitted for short term loads.

The drilled piers are not expected to experience significant settlement if static loads do not exceed the recommended allowable axial capacity. The total settlement of drilled piers as a result of liquefaction occurring during the design level earthquake is expected to be on the order of 6 to 8 inches, if no ground improvement is used. The differential liquefaction-induced settlements can be estimated at about 2 to 4 inches over 30 feet.

7.6.2 Concrete Slabs On-grade

Minimum Slab Thickness and Reinforcement. We recommend that all concrete slabs be reinforced. Slab thickness and reinforcement should be designed by the project structural engineer to resist structural loading and to satisfy pertinent code, temperature, and shrinkage requirements. As a minimum, we suggest that all slabs be at least 6 inches thick and be reinforced with No. 4 reinforcing bars (Grade 40) spaced at 14 inches on-center each way.

Reinforcement should be placed at mid-thickness of the slab with means to ensure that the reinforcement remains in place during construction and concrete placement.

Existing surficial soils at the site exhibit a moderate expansion potential; however, the proposed overexcavation and replacement of existing surficial soils, together with proper site drainage, will create a layer of non-expansive material that is expected to mitigate the potential for expansive soil damage to concrete slabs on-grade.

Vapor Barrier. When moisture sensitive flooring is expected, interior floor slabs should be protected against moisture vapor penetration with a continuous impermeable membrane. The impermeable membrane should be at least a 6-mil-thick polyethylene sheet or similar commercial vapor barrier that is placed midway within 4 inches of sand placed directly beneath the slab. In descending sequence, slabs with moisture-sensitive flooring should be underlain by:

- Two inches of sand;
- The vapor barrier membrane;
- Two inches of sand; and
- Four inches of crushed stone.

The crushed stone beneath the 4 inches of sand is to act as a capillary break. This layer should consist of poorly graded pea gravel or crushed rock. A material conforming to ASTM C33, Grade 67 is suggested with sizes ranging mostly between one-quarter and one-half inch. The lower sand layer is to act as a protective layer against penetration of protrusions through the vapor layer. The top sand layer, which is to protect the vapor barrier from construction activities, should be moistened slightly prior to placing concrete. Those layers should consist of clean sand with less than 5 percent passing the No. 200 sieve.

Both the sand and crushed stone layer should be lightly vibrated with four to five passes of a base plate on a walk-behind, self propelled vibrator.

7.6.3 Berm Stability Evaluation

An existing berm borders the entire length of the west side of the project site and separates the site from a drainage canal immediately to the west. An unpaved access road runs along the crest of the berm, which has an elevation of about -221 feet (NAVD88). The height of the east side of the berm (i.e., facing the project site) varies between about 4 feet at the south end to about 7 feet at the north end. Based on information provided by CalEnergy, the existing berm crest will be raised to elevation -220 feet (NAVD88) and the east face of the berm will be regraded to a constant slope of about 2:1 (horizontal to vertical) or flatter. We have assumed that the berm crest will be about 12 feet wide to accommodate vehicle traffic.

The stability of a 2:1 berm was evaluated assuming subsurface soil properties similar to idealized soil profiles used for developing foundation recommendations as presented in Section 7. The soil properties of berm material were estimated by review of the data collected by CPTs and HSA borings advanced along the berm crest.

Berm stability was evaluated for static and dynamic loading conditions. Static loading conditions included water on the west side of the berm up to Elevation -223 feet (i.e., three feet of freeboard) and vehicle loading on top of the berm equal to a distributed load of 250 psf over the berm crest. Dynamic loading conditions were evaluated using pseudo-static analyses taking into account the potentially liquefiable soils underlying the berm.

The static stability analysis resulted in a factor of safety of 1.9 as presented in Figure 7-7, for the conditions described above, indicating that the berm is expected to remain stable under the assumed static loading conditions. The stability of the berm under seismic conditions was evaluated using a pseudo-static analysis by estimating the displacement expected to occur during a design level earthquake. Based on the results of the pseudo-static analysis, also presented in Figure 7-7, a yield coefficient of about 0.13g was assessed for a berm underlain by liquefiable sand materials. Based on the calculated yield coefficient, coupled with our understanding of seismic demand at the site and the seismic displacement assessment recommendations as presented by Bray and Travasarou (2007), a seismic horizontal displacement of less than 6 inches was estimated for the design level earthquake. The results indicate that, while some displacement is likely to occur (estimated up to 6 inches), the berm is not expected to suffer global failure, i.e., the berm will still remain functional. However, some regrading and slope repair might be required following the design level earthquake to return the berm surface elevations to pre-earthquake levels

New materials used for the improvement of the berm should be compacted to a minimum of 95 percent relative compaction per ASTM D1557. Unless otherwise specified, all fill shall be placed in accordance with the recommendations provided in Section 8 of this report.

7.7 FOUNDATION DESIGN SUMMARY AND CONCLUSIONS

Based on the subsurface exploration, laboratory testing, and engineering analysis phases of the geotechnical study, Fugro finds that the project site is suitable for development of the proposed Black Rock Units 1, 2, and 3 geothermal power plant. Due to the preliminary stage of development of the facility, detailed plans showing the dimensions of all the structures and foundations were not available at the time this report was prepared; however, recommendations for various foundation options were presented above, assuming that the various mechanical equipment and structures will be grouped as required and supported on reinforced concrete mat foundations or shallow footings where appropriate. Recommendations for ground improvement and deep foundations were also presented for consideration. The optimum foundation system must be selected by CalEnergy and its design team based on a number of criteria over which Fugro has limited control. In general, these criteria include performance, cost, and risk. The foundation system should provide the required level of performance at the lowest cost and at a level of risk acceptable to the owner.

7.7.1 Shallow Footing and Mat Foundations

For structures capable of withstanding the estimated settlements presented in this report, and that have a fundamental period of vibration of 0.5 seconds or less, it is the opinion of Fugro that shallow footing and mat foundations, designed and constructed in accordance with the recommendations of this report, are suitable for use at this site and will provide the lowest cost foundation system; however, CalEnergy must be prepared to accept any risk of damage to, or loss of use of, the facility resulting from static settlement or seismically induced settlements or ground shaking. For example, if the facility is required to either act as an emergency backup facility or be operational immediately after an earthquake, the time required to make repairs may be unacceptable. Moreover, the cost of the repairs alone may be unacceptable, regardless of any facility downtime considerations.

Structures that have a fundamental period of vibration greater than 0.5 seconds may also be supported on shallow footings or mat foundations but will require a site response analysis as required by CBC 2007 and ASCE 7-05 to assess the design spectral response acceleration parameters. This analysis would not be required if liquefaction mitigation measures, such as the ground improvement methods discussed previously, are implemented as part of the foundation design.

7.7.2 Ground Improvement

For structures supported on shallow footings or mats and that are not capable of withstanding the estimated settlements, or where the repair costs or facility downtime factors create an unacceptable level of risk, ground improvement may be performed to mitigate the liquefaction potential and reduce both static and seismically-induced settlements. A treatment depth of approximately 50 feet below existing ground surface is considered appropriate for this site. The treatment area will depend on structural loading requirements and the selected ground improvement technology. Ground improvement technologies suitable for use at this site include stone columns and Cement Deep Soil Mixing. Fugro understands that CalEnergy has used stone columns on previous geothermal plant projects in the area and is therefore familiar with the cost and performance of this technology in ground conditions similar to those at the Black Rock site; however, Fugro considers that CDSM would provide a structurally superior, and potentially less expensive, alternative to stone columns. Fugro recommends that CalEnergy and their design team evaluate both these ground improvement technologies with respect to cost, performance, and time of construction and select the optimum alternative.

7.7.3 Deep Foundations

If any of the anticipated settlements, either static, liquefaction-induced seismic, and/or differential, are not acceptable for sensitive equipment or structures, and ground improvement alone is not considered feasible for reasons of cost or performance, a deep foundation system consisting of driven piles may be used for support of such structures. Recommendations for design and installation of various sizes of driven precast concrete piles were presented in this report. As discussed previously in this report, a pile foundation system would have to be supplemented with stone columns or CDSM to depths of approximately 20 feet around the foundation perimeter in order to provide additional lateral support during the design seismic event. Fugro recommends that CalEnergy consult with their design team and equipment manufacturers to determine if a deep foundation system is a requirement for any component of the proposed facility.

8.0 SITE DEVELOPMENT AND GRADING

8.1 SITE PREPARATION

Soil containing debris, organics, pavement, abandoned utilities, or other unsuitable materials, should be stripped from all proposed foundation, structure, and pavement areas and discarded offsite. With the exception of the perimeter access roads, the majority of the project site consists of cultivated farm land that was planted with alfalfa at the time of Fugro's October 2008 field investigation. The surface of the cultivated area consists of rows of slightly raised planted beds separated by lower furrows for irrigation. In order to remove the root zone and other organic material, a minimum of 18 inches of existing soil, as measured from the top of the existing planted areas, should be stripped from the site in areas where roads, equipment, and structures will be constructed. This material is not considered suitable for re-use as structural fill at the site. Additional removal may be required depending on the conditions observed at the time the grading is performed.

Fugro understands that it is planned to raise existing grades in the proposed structure and road areas a minimum of 1.5 feet above existing grade in order to achieve proper site drainage conditions. Placement of compacted fill will require a stable grade on which to operate compaction equipment; however, the high-moisture content, fine-grained soils present across the site will likely prevent obtaining the specified compaction requirements and create difficult working conditions for the earth moving equipment anticipated for the project. If compaction cannot be achieved because of wet conditions and pumping soil, stabilization of the excavation bottom will be necessary. Recommendations for stabilization of the excavation bottom are presented in the following sections.

The exposed surface of all excavation areas should be observed by Fugro prior to processing or placing fill. Excavation bottoms should be deepened, as needed, to remove loose or soft materials, artificial fill, or other deleterious material where encountered.

8.2 SPECIAL SUBGRADE STABILIZATION MEASURES

Special stabilization measures will likely be required if moist, soft or pumping subgrade is encountered during construction. These measures will be required to provide a firm and unyielding subgrade surface on which to place fill and perform construction activities. Special subgrade stabilization measures that have been used successfully for other projects, and that are considered suitable for this site consist of:

- Deepening the excavation bottom by about 1 to 2 feet, followed by placing a layer of geotextile, such as Geolon HP570, or the equivalent, on the excavation bottom, followed by the placement of about 2 feet of 4-inch minus crushed rock over the fabric. A filter fabric such as Mirafi 180N, or equivalent, should be placed on top of the rock layer prior to placing fill in order to reduce the potential for migration of fines into the rock; or
- Lime or Portland cement treatment of the fine-grained subgrade, followed by placement of compacted structural fill. Depending on the type of fill soil available, additional lime or cement treatment of the structural fill may be warranted.

The measure required will depend on the condition of the subgrade at the time of construction, the type of structural fill material that will be placed, the nature of the construction activities (e.g., vibratory compaction equipment, number of equipment passes), and the availability/cost of materials.

Fugro understands that imported fill soils will likely be similar to the on-site fine-grained materials. These soils are generally very sensitive to moisture content and can be difficult to compact. Therefore, consideration should be given to use of lime or Portland cement throughout the fill layer beneath all structures and roads. This procedure will allow for greater ease of compaction and also create a much stronger structural layer that will be more resistant to changes in moisture content over time.

8.3 DRAINAGE

Positive drainage should be developed and maintained away from all structures, foundations, and any exterior improvements. Hardscape areas should be maximized where possible adjacent to foundations to reduce the potential for water infiltration. Roof and surface runoff should be collected and conveyed away from structures and on-grade improvement areas. Water should not be allowed to accumulate or pond near structure foundations or on-grade improvements.

8.4 FILL PLACEMENT AND COMPACTION

Fill placement and grading operations should be performed according to the grading recommendations of this report. Fugro recommends that, unless otherwise noted, all fill materials be compacted to at least 95 percent relative compaction based on the maximum dry density determined from ASTM D1557.

Imported soils used as compacted fill should be placed and compacted at a moisture content of between -1 and +3 percent of the optimum moisture content. Each layer should be spread evenly and should be thoroughly blade-mixed during the spreading to provide relative uniformity of material within each layer. Soft or yielding materials should be removed and be replaced with properly compacted fill material prior to placing the next layer.

Rock, gravel, and other oversized material (greater than 4 inches in diameter) should be removed from the fill material prior to being placed. Rock less than 4 inches in diameter should not be nested, and voids caused by inclusion of rock in the fill should be filled with sand or other approved material.

When the moisture content of the fill material is below that sufficient to achieve the recommended compaction, water should be added to the fill. While water is being added, the soil should be bladed and mixed to provide relatively uniform moisture content throughout the material. When the moisture content of the fill material is excessive, the fill material should be aerated by blading or other methods. Fill should be spread in loose lifts no thicker than about 8-inches prior to being compacted. Fill and backfill materials may need to be placed in thinner lifts to achieve the recommended compaction depending on the equipment being used.

8.5 MATERIALS

8.5.1 Imported Fill

Imported fill should be free of organics (i.e., roots, vegetative matter, etc.), oversize material (i.e., rocks greater than 4 inches in diameter), trash and debris, and other deleterious material. Organics should be removed from the soils to be used as fill so that fill soils have an organic content of less than 3 percent by weight. All imported fill materials should have an

Expansion Index (EI) of less than 20, which should be verified during grading. Imported fill meeting these requirements may be used as backfill in foundation and road overexcavation areas, except as noted otherwise in this report.

All fill of any origin proposed for use at the site should be observed and tested by Fugro prior to import to, or use at, the site.

8.6 GEOTECHNICAL OVERSIGHT

Proper geotechnical observation and testing during construction are imperative in allowing the geotechnical engineer the opportunity to verify assumptions made during the design process. Therefore, all overexcavation and fill placement activities should be performed under the observation and testing of the geotechnical engineer of record for the project.

The Conditions of Certification will require that the project owner assign California-registered civil and geotechnical engineers, and a California-certified engineering geologist to the project to provide the necessary oversight and inspection. Therefore, we recommend that Fugro be retained during site grading and foundation construction to observe compliance with the design concepts and geotechnical recommendations, and to allow design changes in the event that subsurface conditions or methods of construction differ from those anticipated.

9.0 FLEXIBLE PAVEMENT DESIGN

9.1 DESIGN SECTION AND MATERIALS

Pavement thickness design depends on the strength of the subgrade soils, the type of construction materials, and on the traffic loading to which the pavement will be subjected. Subgrade strength was evaluated by Resistance-value (R-value) tests performed on three samples of subgrade soils obtained from proposed paving areas. Based on the results of the R-value tests and the general soil conditions at the site, an R-value of 4 is considered appropriate for design of pavement both on-site and on adjacent McKendry and Boyle Roads where pavement improvements will also be made. It is assumed that on-site subgrade will most likely be treated with lime or Portland cement and off-site subgrade (i.e., along McKendry and Boyle Roads) may not be treated.

Design traffic loading conditions were not available at the time of this study; therefore, Traffic Indices of 5, 6, 7, and 8 were assumed to be representative of the range of traffic loading conditions that will occur in road and parking areas and were used to develop pavement sections for the project. The project civil engineer should select the appropriate Traffic Index (T.I.) for each pavement area based on the design traffic loading conditions. If design T.I. values are different from the assumed values, Fugro should be notified for reevaluation of pavement section thickness.

Pavement sections were developed using the Caltrans design method for flexible pavement based on a 20-year design life and are presented in Tables 9-1 and 9-2. The sections are based on the parameters discussed above for both treated and untreated subgrade conditions.

Table 9-1. Recommended Pavement Sections for Pavement Supported on at least 12 inches of Soil-Cement or Lime Treated Subgrade

Traffic Index (TI)	5	6	7	8
Asphalt Concrete (ft)	0.25	0.25	0.35	0.40
Class II Aggregate Base (ft)	0.35	0.35	0.35	0.35

Table 9-2. Recommended Pavement Sections for Pavement with no Soil-Cement or Lime Treated Subgrade

Traffic Index (TI)	5	6	7	8
Asphalt Concrete (in.)	0.25	0.25	0.35	0.40
Class II Aggregate Base (in.)	0.80	1.15	1.25	1.50

9.2 CONSTRUCTION CONSIDERATIONS

Subgrade. Roadway areas should be prepared as described in Section 8 of this report. The areas to receive pavement should be stripped and excavated to the proposed subgrade elevation or entirely through any existing asphaltic-concrete or base. The upper 12 inches of all pavement subgrade, treated or untreated, should be moisture conditioned to within 2 percent of optimum moisture content and compacted to at least 95 percent of the maximum dry density as determined by the latest revision of ASTM D1557. All subgrade preparation activities should be performed under the observation and testing of a representative of the project geotechnical engineer.

R-value tests should be performed on subgrade materials near the completion of rough grading and on potential import soils in order to confirm pavement design sections. The samples for the confirmatory R-value tests should be obtained from the upper 3 feet of pavement subgrade soils.

Aggregate Base. Aggregate base should have a minimum R-value of 78 and conform to the requirements of California Class II Aggregate Base. Aggregate base material should be compacted in lifts not exceeding 6 to 8 inches in thickness, to at least 95 percent of the maximum dry density determined from ASTM D1557, latest revision. As-compacted moisture contents for the aggregate base materials should be within 2 percent of the optimum moisture content, as determined from ASTM D1557.

Drainage. Proper drainage of the paved and surrounding unpaved areas is essential. Grades should be established to expedite runoff away from the pavements and reduce moisture infiltration in the base and subgrade.

10.0 CORROSION AND CHEMICAL DATA

Soil corrosion potential for buried metal and concrete was estimated by performing water-soluble sulfate, chloride, pH, and electrical resistivity tests. Results of these tests for near-surface soils are provided in Appendix B and are summarized in Table 10-1.

Table 10-1. Summary of Chemical Test Results

Location	Depth (feet)	Material	Sulfates (ppm)	Chlorides (ppm)	Resistivity at 100% Saturation, ohms-cm	pH	Redox (mV)
HSA-3	0 - 3	Clay	2,800	1,500	270	8.2	470
HSA-10	0 - 3	Clay	1,300	800	280	7.9	460
HSA-11	0 - 3	Clay	1,200	630	310	7.9	470
HSA-14	0 - 3	Clay	1,200	970	400	7.9	460
HSA-20	0 - 3	Clay	1,600	970	260	8.4	460

ppm - parts per million; mV - millivolt

Electrical resistivity is a measure of soil resistance to the flow of electrical current. The electrical resistivity of a soil decreases primarily as its chemical and moisture contents increase. The corrosion potential for ferrous metals is generally higher in soils with low electrical resistivity. A commonly accepted correlation between electrical resistivity and corrosivity for buried ferrous metals is presented below in Table 10-2.

Table 10-2. Soil Corrosion Potential Correlation

Electrical Resistivity (Ohms-cm)	Corrosion Potential
Less than 1,000	Severe
1,000 to 2,000	Corrosive
2,000 to 10,000	Moderate
Greater than 10,000	Mild

Results of electrical resistivity tests indicate values ranging between 260 and 400 ohms-cm for the near-surface soils. Based on this limited data, near-surface soils at the Black Rock site appear to have a severe corrosion potential for buried ferrous metals. This potential should be considered in design of underground metal pipes.

Based on the results of the sulfate tests presented above, the surficial soils appear to have a moderate to severe degree of corrosivity to concrete. Concrete in contact with site soils should be designed and constructed in accordance with the requirements of the American Concrete Institute (ACI) 318, Section 4.3 as specified in Section 1904A.3 of the 2007 CBC. The results of the tests presented above indicate this will require the use of Type V Portland cement and a maximum water-cement ratio of 0.45 for concrete in contact with soils. Appropriate testing should be performed at the completion of rough grading to confirm the corrosion resistance design requirements for concrete to be placed in contact with site soils.

Based on the results of the chloride tests presented above, the surficial soils appear to have a moderate to severe degree of corrosivity to ferrous metals. Reinforcement in concrete should be protected from corrosion and exposure to chlorides in accordance the requirements of the American Concrete Institute (ACI) 318, Section 4.4 as specified in Section 1904A.4 of the 2007 CBC. Appropriate testing should be performed at the completion of rough grading to confirm the corrosion resistance design requirements for concrete to be placed in contact with site soils.

In general, corrosive site soils should be assumed in estimating the design life of underground utility lines and buried structures at the Black Rock site. Fugro recommends that a corrosion engineer be consulted to determine the most appropriate corrosion protection measures for all buried utilities and structures at the site, including pile foundations.

11.0 REFERENCES

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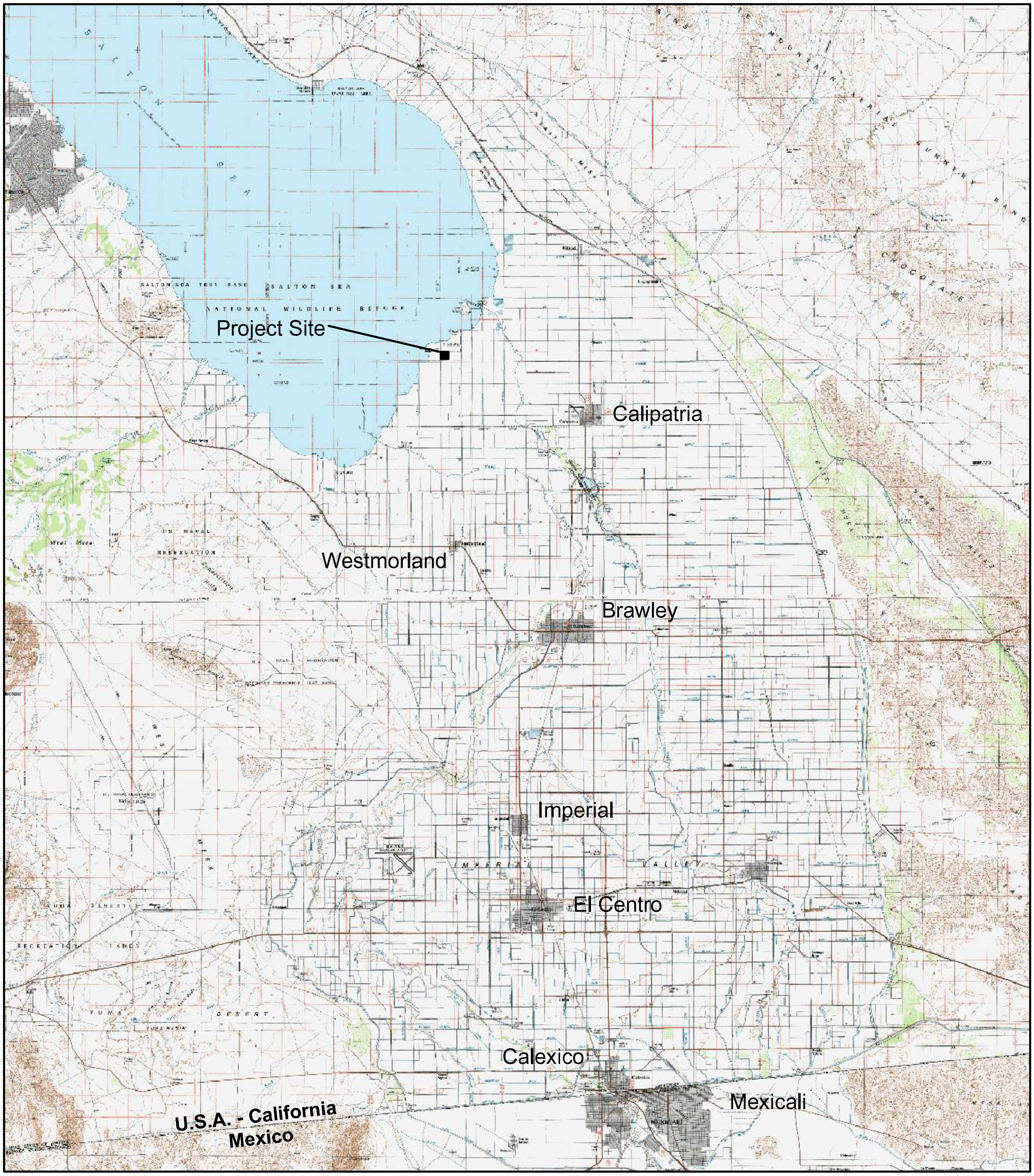
FIGURES



VICINITY MAP
Black Rock Units 1, 2 & 3
Calipatria, California

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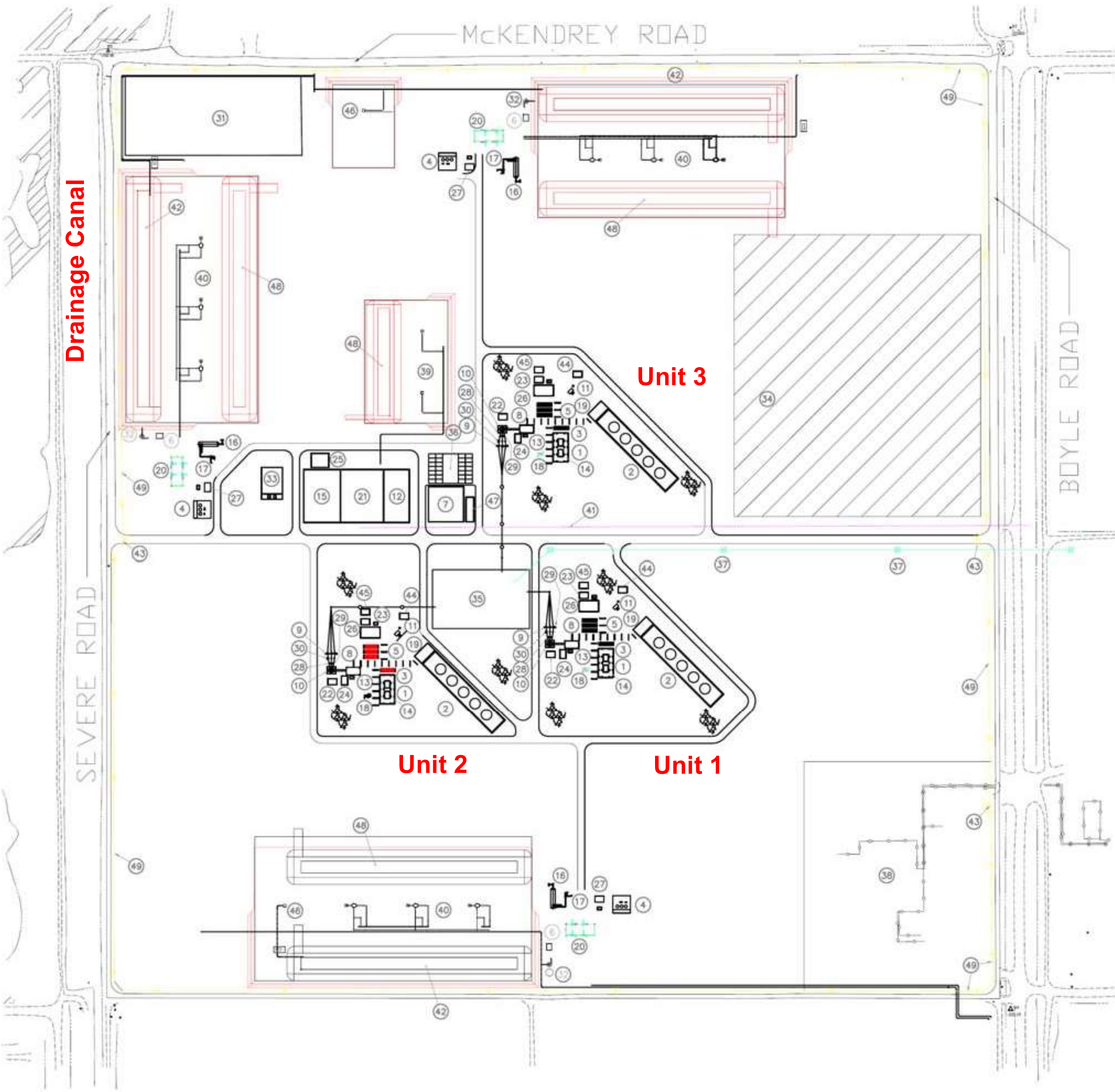
FIGURE 1-1



SITE LAYOUT
Black Rock Units 1, 2 & 3
Calipatria, California

FIGURE 1-2

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EP-07-11
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by: J. Reverente

BLACK ROCK 1, 2 and 3

Major Equipment Dimension & Weights			
EQUIPMENT DESCRIPTION	**WEIGHTS**	**DIMENSION or SIZE**	**REMARKS**
Equipment	*Pounds*		
HP Separator	350,000	12' ID x 54' T/T	Horizontal
HP Steam Scrubber	62,000	5' ID x 28' T/T	Vertical
HP Steam Demister	75,000	8' ID x 18' T/T	Vertical
Turbine	273,500	49' long x 16' wide x 10' high	Turbine/Generator combined
Generator	208,006		dimensions for TG set
Main Condenser	505,019		Under the turbine/generator set
First Stage Steam Ejector, 1st Train		26" x 26" Ejector with	
First Stage Ejector Nozzle, 1st Train		Multi-Nozzles	
First Stage Inter-Condenser, 1st Train	15,000	36" Shell Diameter	
First Stage Steam Ejector, 2nd Train		26" x 26" Ejector with	
First Stage Ejector Nozzle, 2nd Train		Multi-Nozzles	
First Stage Inter-Condenser, 2nd Train	15,000	36" Shell Diameter	
First Stage Steam Ejector, 3rd Train		26" x 26" Ejector with	
First Stage Ejector Nozzle, 3rd Train		Multi-Nozzles	
First Stage Inter-Condenser, 3rd Train	15,000	36" Shell Diameter	
Cooling Tower (fiber glass)	7,572,000	48.5' width x 276.5' length	Includes concrete basin + water
Cooling Tower Basin (Reinforced concrete)	2,800,000	54' width x 282' length	Water = 3,800,000 Lbs.
Rock Muffler (Reinforced concrete)		16' wide x 20' long x 24' high	
Production Test Unit	60,000	15' ID x 38' top to cone bottom	307,000 lbs with full of water
NCG Knock-Out Pot		36" Shell Diameter, 15' T/T	
NCG Oxidizer/Heat Exchanger	24,000	2,500 ACFM	
NCG Quench Tank & Scrubber + vent stack	6,500		
36% Acid Tank A	11,000 gallons		
36% Acid Tank B	11,000 gallons		
2.5% Acid Tank	38,000 gallons		
Brine Holding Pond (Earthen pond with liners)	1,100,000 gallons	600' long x 50' wide x 7' deep	with 2 feet freeboard
Common Holding Pond (Reinforced Concrete)	2,585,088 gallons	360' long x 120' wide x 10' high	rectangular structure with 12" wall
Power Distribution Control 1		42' x 27' x 10'	Metal Building
Power Distribution Control 2		40' x 28' x 10'	Metal Building
Power Distribution Control 3		42' x 27' x 10'	Metal Building
Control Building		100' x 100'	For Black Rock 1, 2 and 3
Emergency Diesel Generator, 1.5 MW	31,131	232.09"L x 99.9"W x 108.25"H	
Emergency Diesel Generator, 1.0 MW	17,738	183.74"L x 80.49"W x 86.7"H	
Step-up Transformer	212,374	223"H x 267"Width x 213"Depth	

Based on information provided by CalEnergy in September 2008.

PRELIMINARY EQUIPMENT SIZE AND WEIGHT LIST
Black Rock Units 1, 2 & 3
Calipatria, California

FIGURE 1-3a

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EP-07-11
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by: J. Reverente

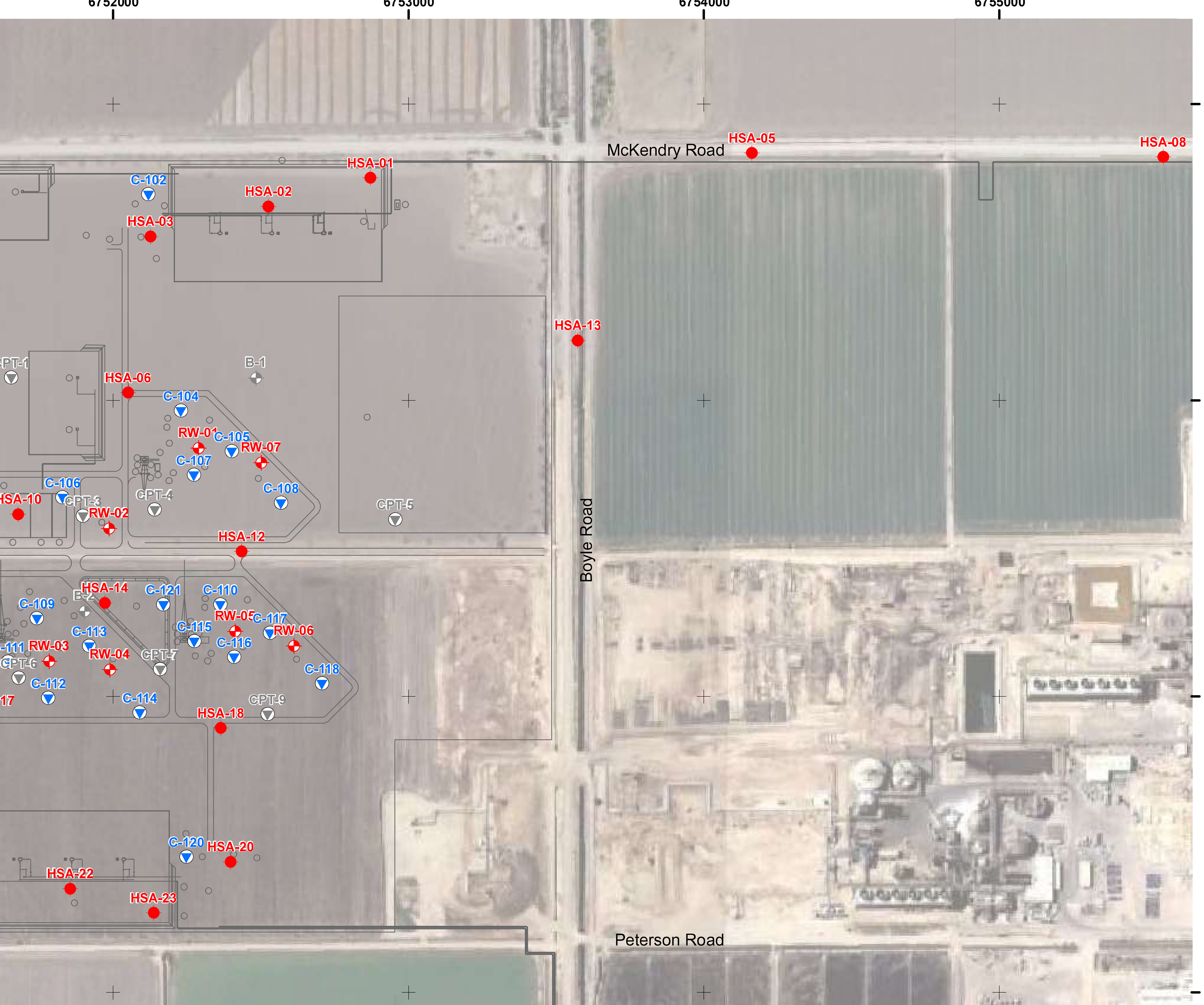
BLACK ROCK 1, 2 and 3

Major Equipment Dimension & Weights			
EQUIPMENT DESCRIPTION	**WEIGHTS**	**DIMENSION or SIZE**	**REMARKS**
Equipment	*Pounds*		
Pumps and Motors			
Brine Booster Injection Pump - A	4,030	14" x 16" x 26"	11,000 gpm flow capacity
Brine Booster Injection Pump - B	4,030	14" x 16" x 26"	11,000 gpm flow capacity
Brine Booster Injection Pump Motor - A	8,600	1,000 HP	
Brine Booster Injection Pump Motor - B	8,600	1,000 HP	
Brine Main Injection Pump - A	4,650	12" x 16" x 26"	11,000 gpm flow capacity
Brine Main Injection Pump - B	4,650	12" x 16" x 26"	11,000 gpm flow capacity
Brine Main Injection Pump Motor - A	10,000	2,000 HP	
Brine Main Injection Pump Motor - B	10,000	2,000 HP	
Aerated Brine Injection Vertical Pump - Toyo	900	8" x 6" x 84"	Cantilever Pump
Aerated Brine Injection Vertical Pump Motor - Toyo		75 HP	
Aerated Brine Main Injection Pump + motor	4,289	5" x 4", 400 HP	600 gpm Flow Capacity
Condensate Injection Pump - A + motor 400 HP	4,500		
Condensate Injection Pump - B + motor 400 HP	4,500		
Hot Well Condensate Pump - A	4,572	14DXC, 10" Discharge	2,100 gpm Flow Capacity
Hot Well Condensate Pump - B	4,572	14DXC, 10" Discharge	2,100 gpm Flow Capacity
Hot Well Condensate Pump Motor - A	1,836	125 HP	
Hot Well Condensate Pump Motor - B	1,836	125 HP	
Vacuum Pump - A, 1st Train + motor 350 HP			
Vacuum Pump - B, 2nd Train + motor 350 HP			
Vacuum Pump - C, 3rd Train + motor 350 HP			
Air Compressor - A + motor, 200 hp + dryer	7,572	826 cfm @ 150 psi	102" x 79" x 70"
Air Compressor - B + motor, 200 hp + dryer	7,572	826 cfm @ 150 psi	102" x 79" x 70"
Air Receiver	4,138 gallons	6' OD x 18' T/T	
Circulating Water Pump - A	20,683	44GHXC, 36" Discharge	
Circulating Water Pump - B	20,683	44GHXC, 36" Discharge	
Circulating Water Pump - C	20,683	44GHXC, 36" Discharge	
Circulating Water Pump Motor - A	15,000	1,000 HP	
Circulating Water Pump Motor - B	15,000	1,000 HP	
Circulating Water Pump Motor - C	15,000	1,000 HP	

Based on information provided by CalEnergy in September 2008.

PRELIMINARY EQUIPMENT SIZE AND WEIGHT LIST
Black Rock Units 1, 2 & 3
Calipatria, California

FIGURE 1-3b



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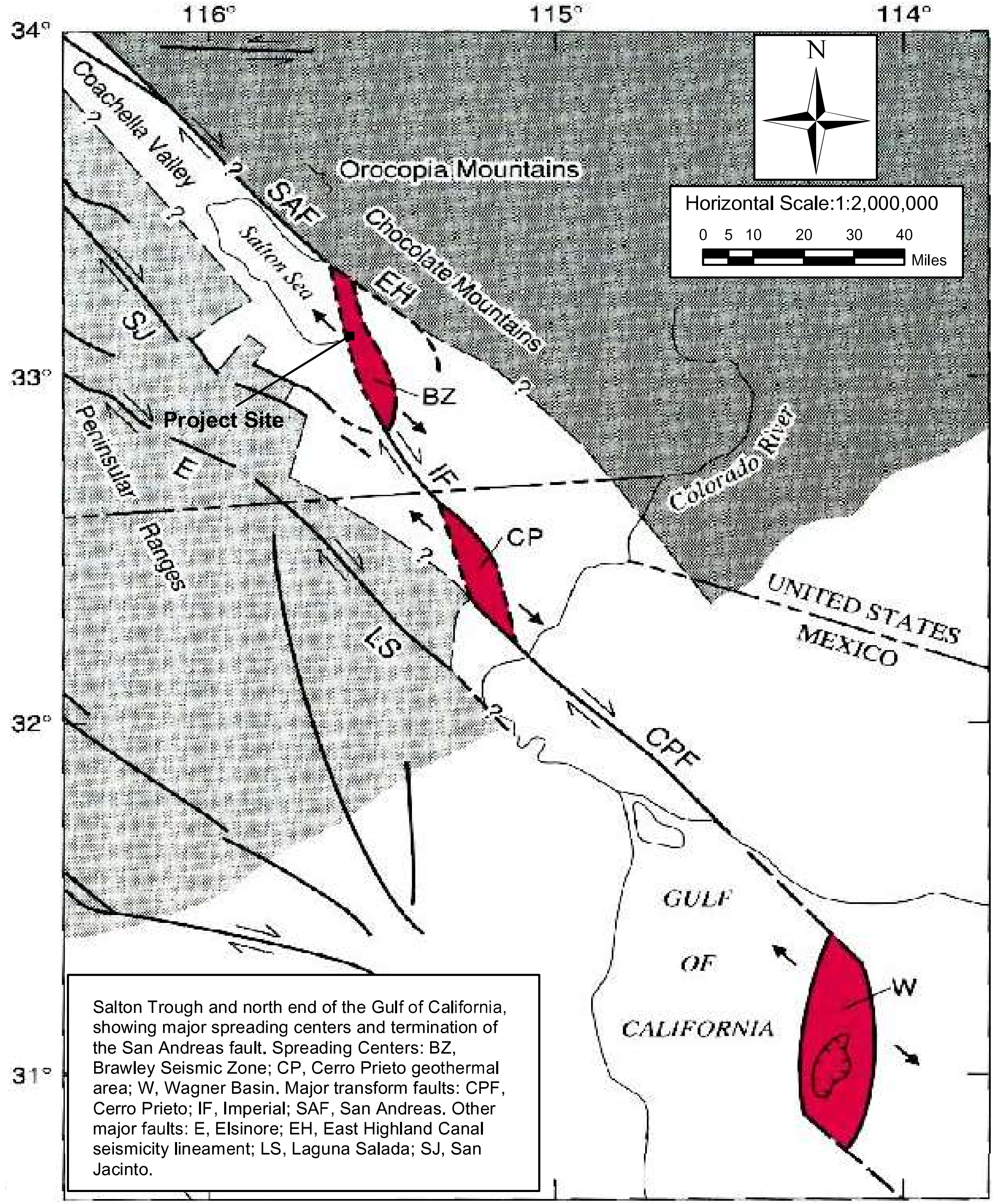

Salton Trough and north end of the Gulf of California, showing major spreading centers and termination of the San Andreas fault. Spreading Centers: BZ, Brawley Seismic Zone; CP, Cerro Prieto geothermal area; W, Wagner Basin. Major transform faults: CPF, Cerro Prieto; IF, Imperial; SAF, San Andreas. Other major faults: E, Elsinore; EH, East Highland Canal seismicity lineament; LS, Laguna Salada; SJ, San Jacinto.

SOURCE: Wallace, 1990

TECTONIC SETTING

Black Rock Units 1, 2 & 3
Calipatria, California

FIGURE 5-1

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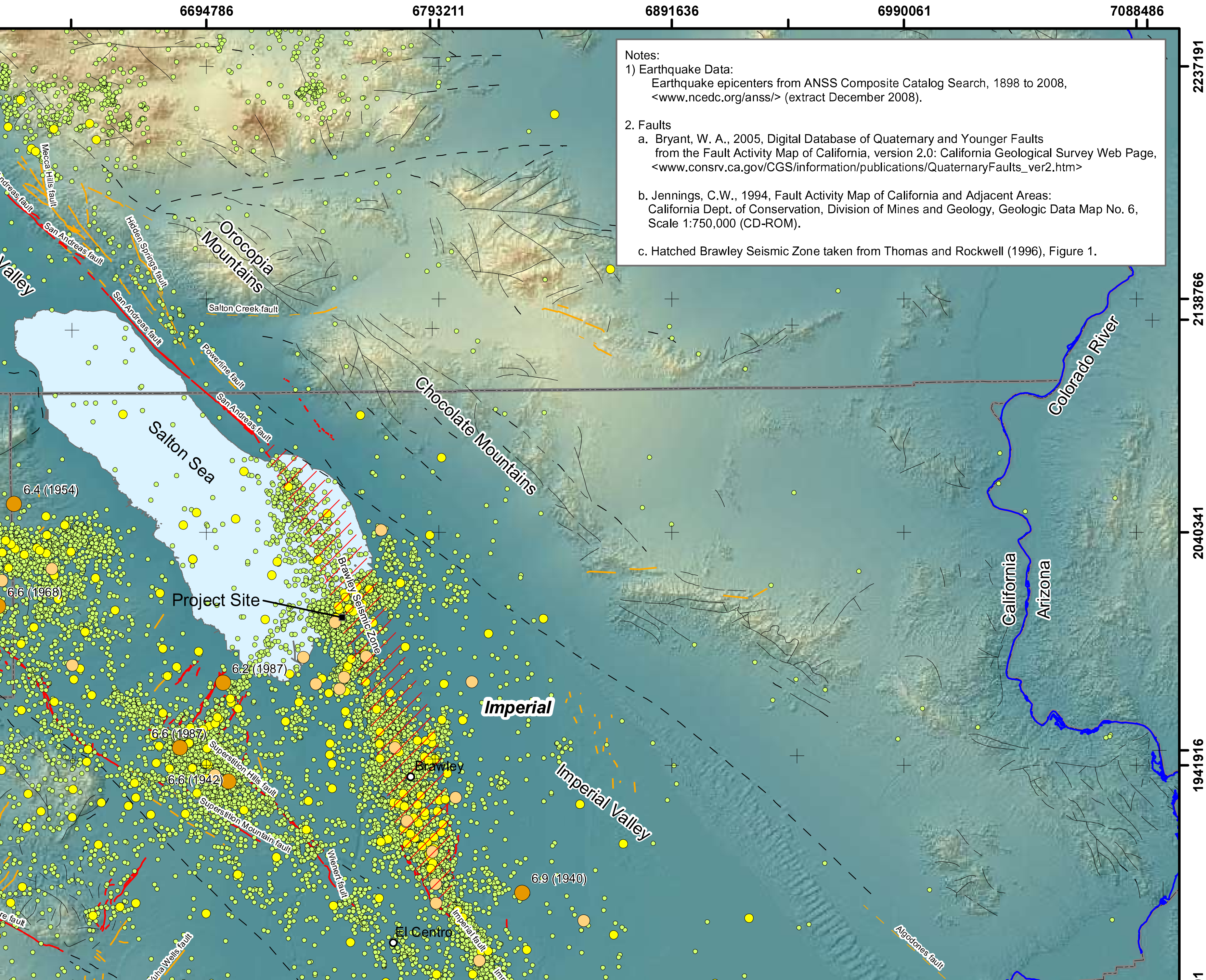

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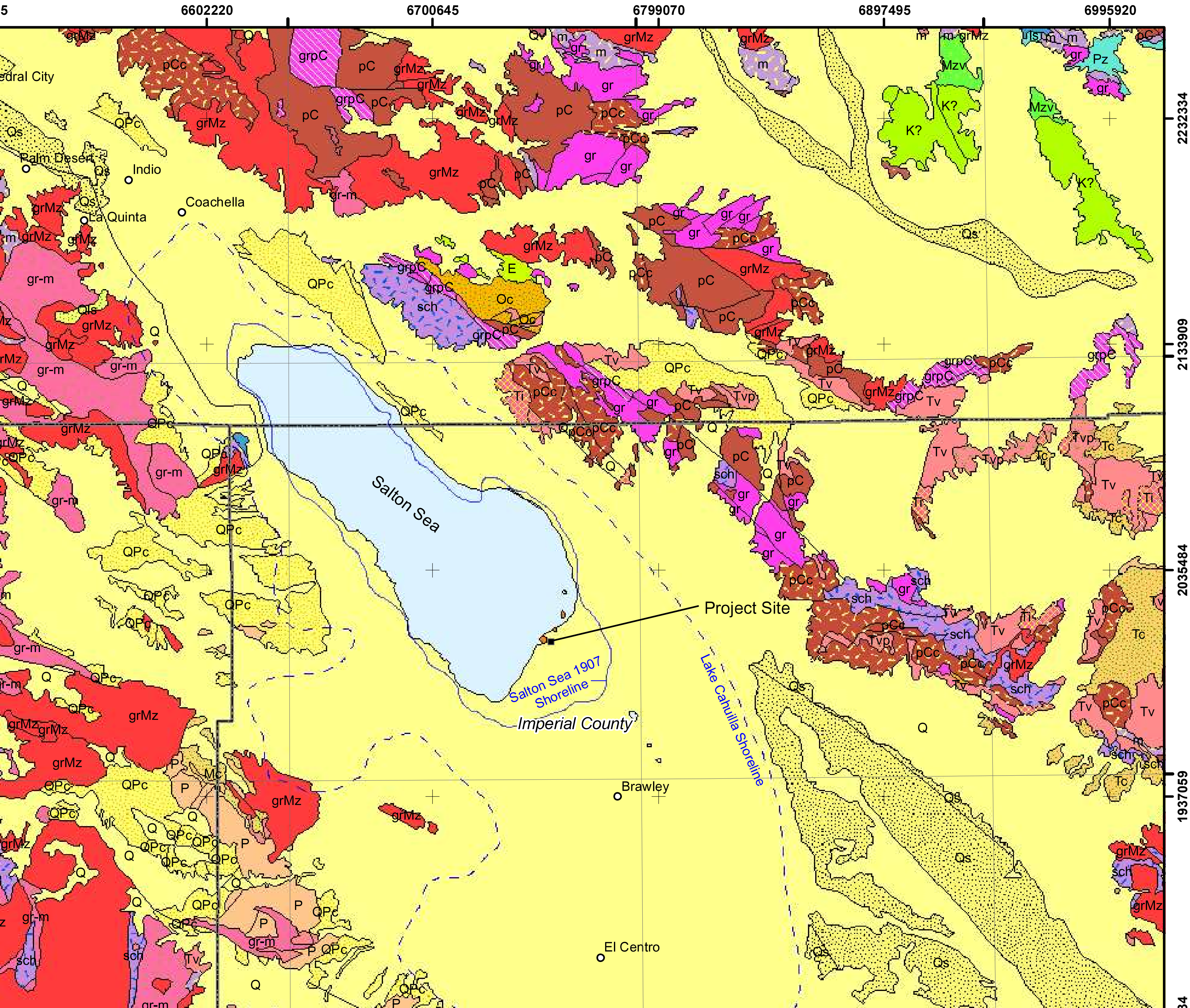

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Notes:
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Figure 3.

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aternary age, includes ponded sediments.

e deposits of probable Pliocene age.

and slope deposits of late Pleistocene age.

e with the San Pedro formation.

and sedimentary rocks of Quaternary and Tertiary (Pliocene and Miocene) age.

osits of Quaternary and late Tertiary (?) age

rocks of Pliocene age.

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Onshore Geologic Units

C	C, Carboniferous marine
Ca	Ca, Cambrian marine
D	D, Devonian marine
E	E, Eocene marine
Ec	Ec, Eocene nonmarine
Ep	Ep, Paleocene marine
J	J, Jurassic marine
K	K, Cretaceous marine undivided(in part nonmarine)
KJf	KJf, Franciscan Complex
KJfm	KJfm, Franciscan melange
KJfs	KJfs, Franciscan schist
Kl	Kl, Lower Cretaceous marine
Ku	Ku, Upper Cretaceous marine
M	M, Miocene marine
Mc	Mc, Miocene nonmarine
Mzv	Mzv, Mesozoic volcanic and metavolcanic rocks; Franciscan volcanic rocks
O	O, Oligocene marine
Oc	Oc, Oligocene nonmarine
P	P, Pliocene marine
Pm	Pm, Permian marine
Pz	Pz, Paleozoic marine, undivided
Pzv	Pzv, Paleozoic metavolcanic rocks
Q	Q, Alluvium (mostly Holocene some Pleistocene);Quaternary nonmarine; Quaternary marine
QPc	QPc, Plio-Pleistocene nonmarine; Pliocene nonmarine
Qg	Qg, Glacial deposits

Qls	Qls, Selected large landsli
Qrv	Qrv, Recent (Holocene) vo
Qrvp	Qrvp, Recent (Holocene) p
Qs	Qs, Extensive sand dune d
Qv	Qv, Quaternary volcanic flo
Qvp	Qvp, Quaternary pyroclast
SO	SO, Silurian and/or Ordovi
TK	TK, Tertiary-Cretaceous Co
Tc	Tc, Tertiary nonmarine, und
Ti	Ti, Tertiary intrusive rocks
Tr	Tr, Triassic marine
Tv	Tv, Tertiary volcanic flow ro
Tvp	Tvp, Tertiary pyroclastic ro
gb	gb, Mesozoic gabbroic roc
gr	gr, Undated granitic rocks
gr-m	gr-m, Granitic and metamo
grCz	grCz, Cenozoic (Tertiary) g
grMz	grMz, Mesozoic granitic ro
grPz	grPz, Paleozoic and Perm
grpC	grpC, Precambrian granitic
ls	ls, Limestone of probable P
m	m, Undivided pre-Cenozoic
mv	mv, Undivided pre-Cenozo
pC	pC, Precambrian rocks, un
pCc	pCc, Precambrian igneous
sch	sch, Schist of various types
um	um, Ultramafic rocks, chief
	water

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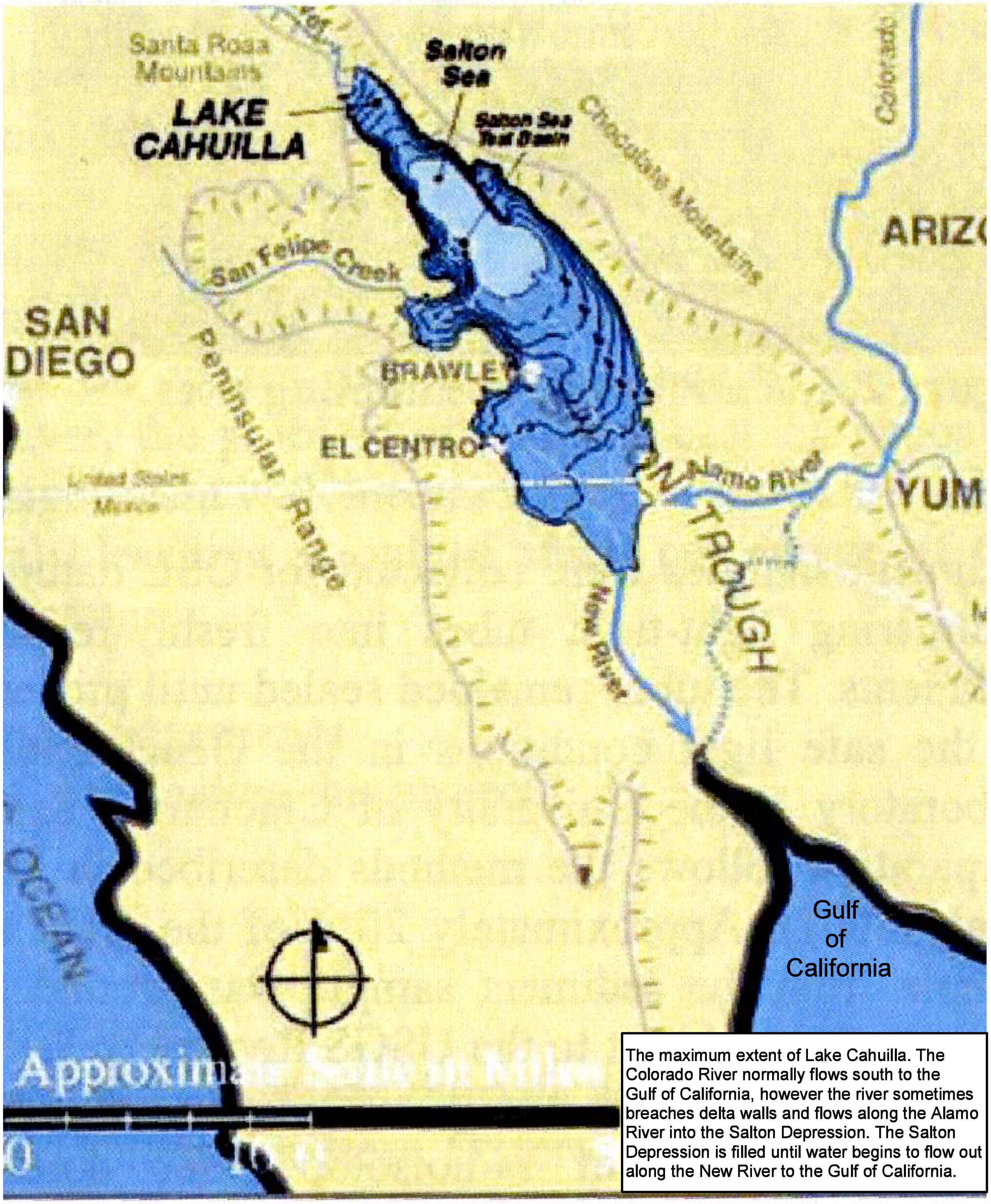

The maximum extent of Lake Cahuilla. The Colorado River normally flows south to the Gulf of California, however the river sometimes breaches delta walls and flows along the Alamo River into the Salton Depression. The Salton Depression is filled until water begins to flow out along the New River to the Gulf of California.

SOURCE: Lippincott et al., 2008

FORMATION OF LAKE CAHUILLA AND FLOW PATTERN CHANGES OF THE COLORADO RIVER

Black Rock Units 1, 2 & 3
Calipatria, California

FIGURE 5-5

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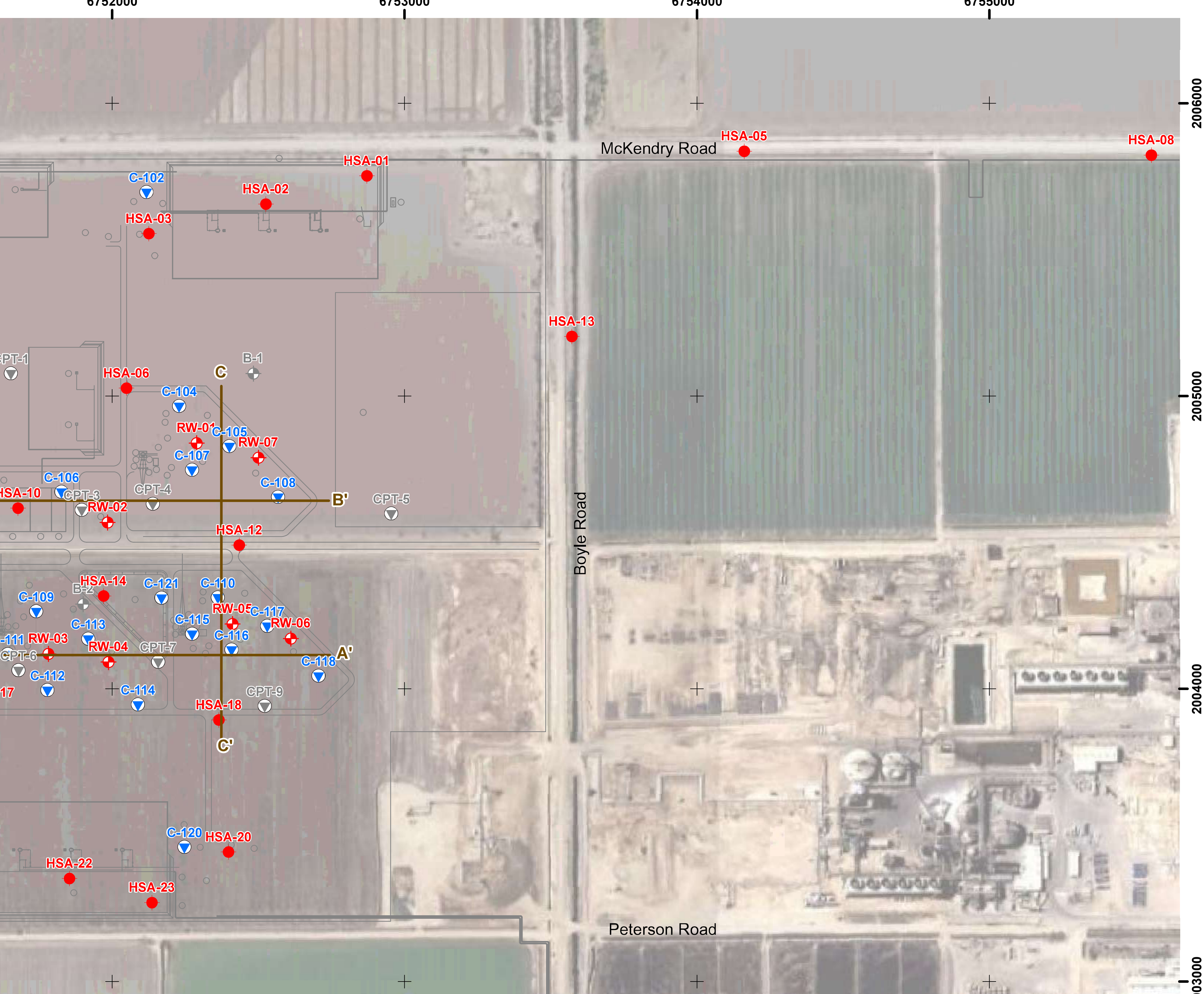

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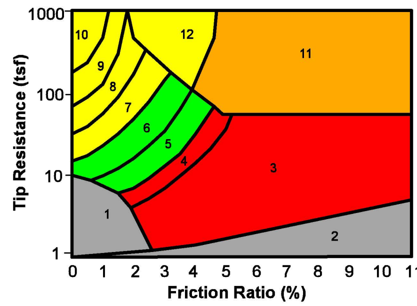
SOIL TYPES

- Well graded GRAVEL (GW)
- Poorly graded GRAVEL (GP)
- GRAVEL with sand (GP or GW)
- GRAVEL with clay (GP or GW)
- Clayey GRAVEL (GC)
- GRAVEL with silt (GP or GW)
- Silty GRAVEL (GM)
- Well graded SAND (SW)
- Poorly graded SAND (SP)
- SAND with gravel (SP or SW)
- SAND with clay (SP-SC)
- Clayey SAND (SC)
- Silty SAND (SM)
- SAND with silt (SP-SM)
- Fat CLAY (CH)
- Sandy fat CLAY (CH)
- Lean CLAY (CL)
- Sandy lean CLAY (CL)
- Silty CLAY (CL-ML)
- Elastic SILT (MH)
- SILT (ML)
- Sandy SILT (ML)
- Clayey SILT (ML/CL)
- Highly plastic ORGANICS (OH)
- Low plasticity ORGANICS (OL)
- SANDSTONE (Rx)
- SILTSTONE (Rx)
- CLAYSTONE (Rx)
- Conglomerate (Rx)
- Interbedded Rock Strata (Rx)
- Pavement

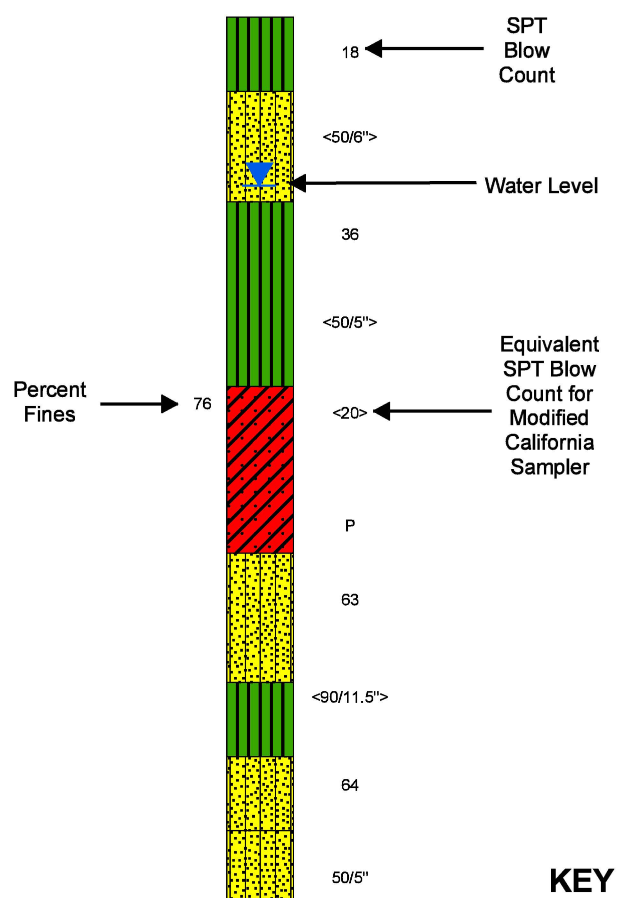


Zone	Soil Behavior Type	U.S.C.S.
1	Sensitive Fine-grained	OL-CH
2	Organic Material	OL-OH
3	Clay	CH
4	Silty Clay to Clay	CL-CH
5	Clayey Silt to Silty Clay	MH-CL
6	Sandy Silt to Clayey Silt	ML-MH
7	Silty Sand to Sandy Silt	SM-ML
8	Sand to Silty Sand	SM-SP
9	Sand	SW-SP
10	Gravelly Sand to Sand	SW-GW
11	Very Stiff Fine-grained *	CH-CL
12	Sand to Clayey Sand *	SC-SM

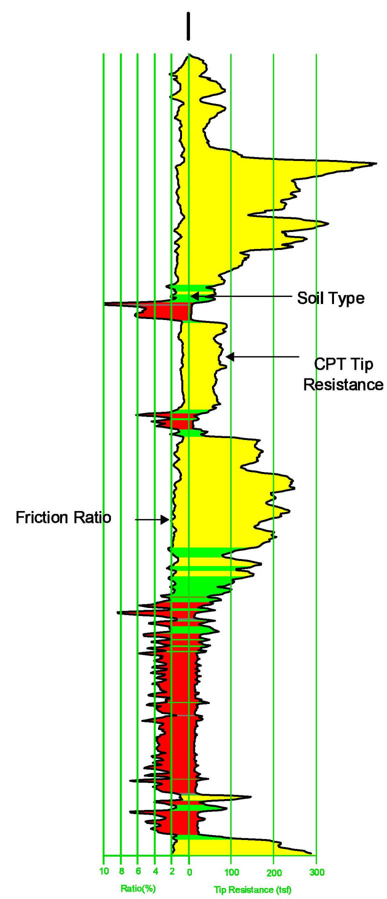
*overconsolidated or cemented

CPT CORRELATION CHART

(Robertson and Campanella, 1988)



SOIL BORING LITHOLOGY

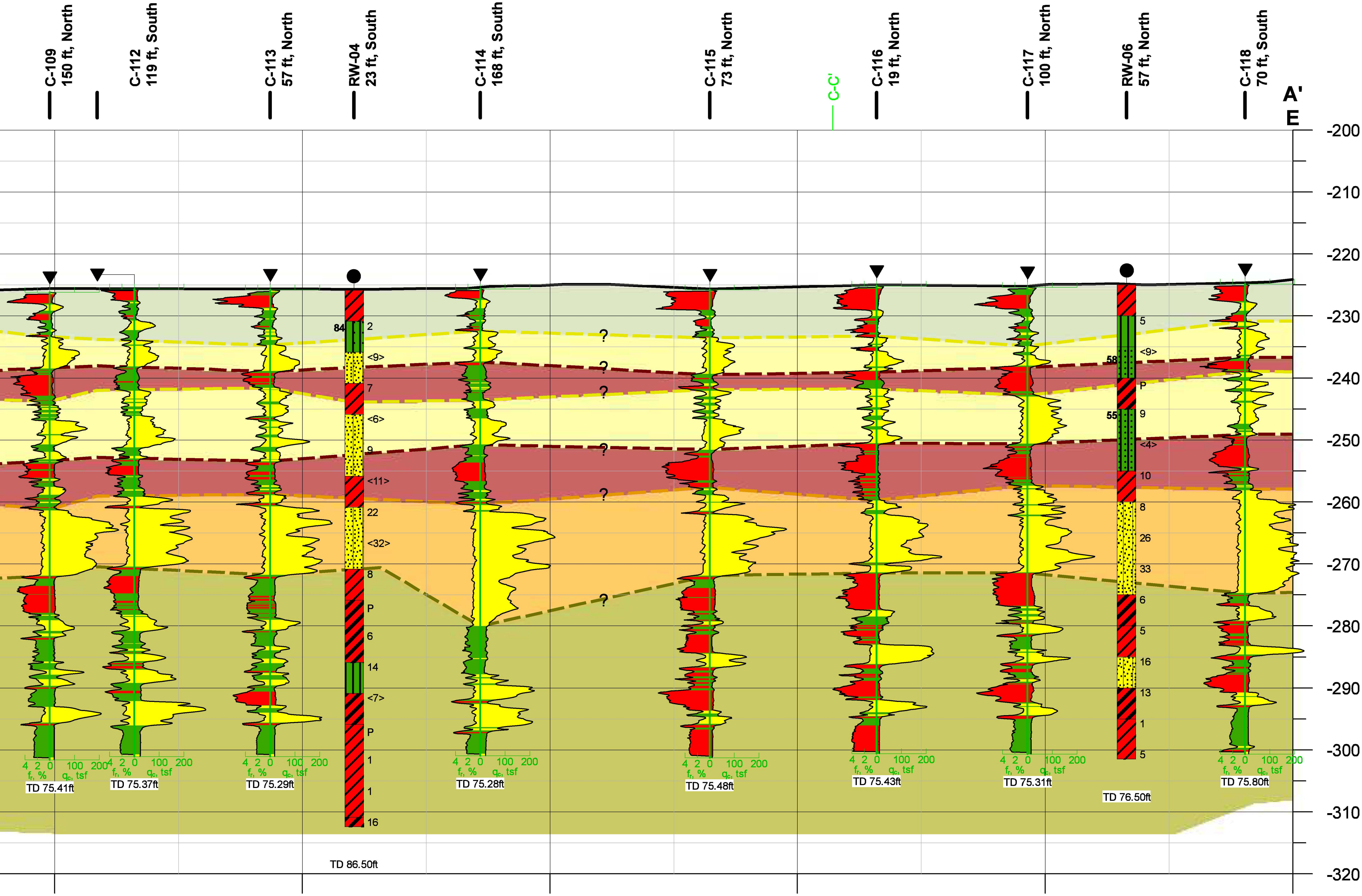


CPT SOUNDING WITH INTERPRETED LITHOLOGY

KEY TO CROSS SECTIONS
Black Rock Units 1, 2 & 3
Calipatria, California

FIGURE 5-7

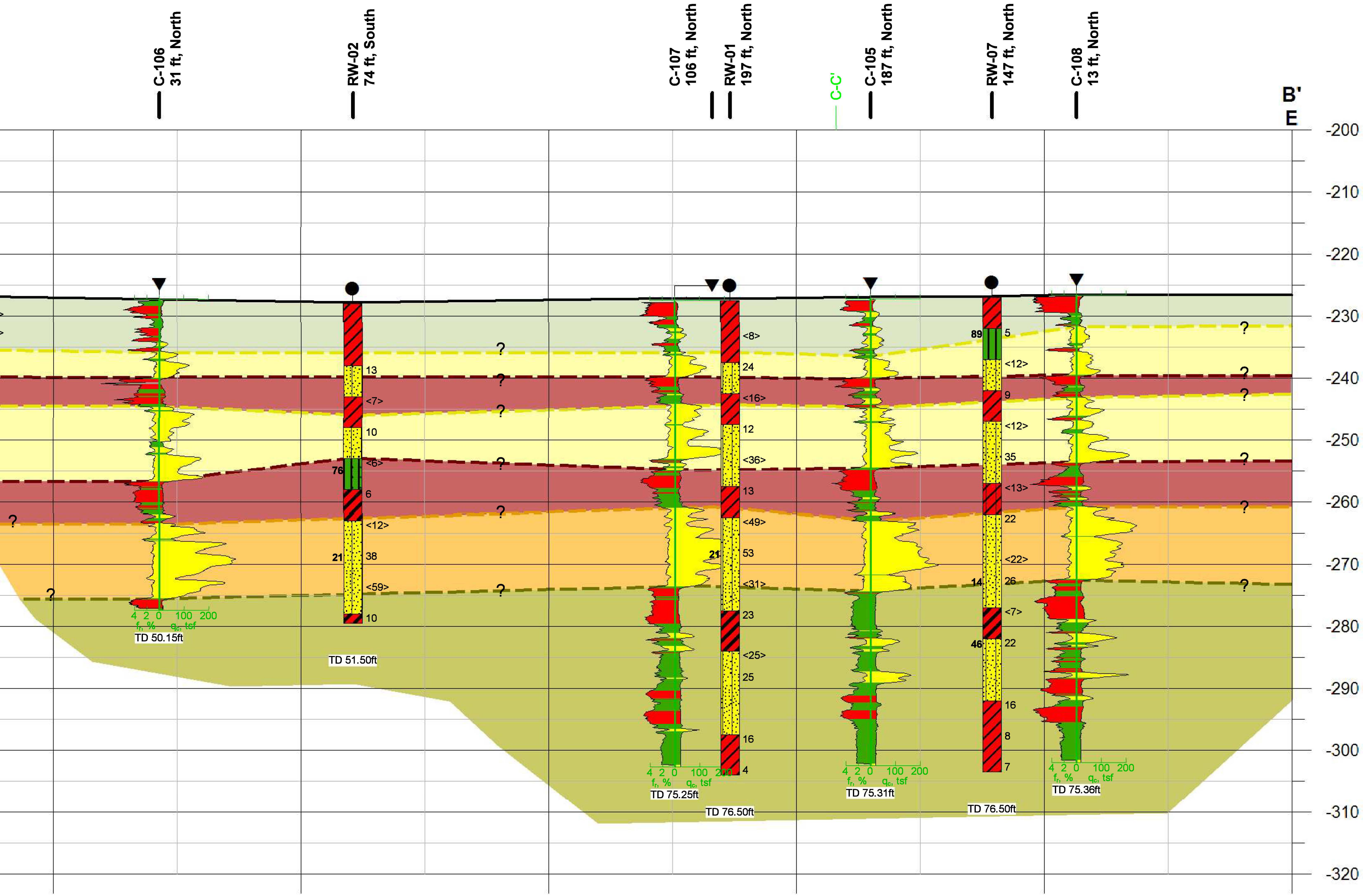
N:\Projects\3652_CalEnergy\3652-001_Black_Rock\Outputs\Final_Report\mxd\Fig5-7_Xsectkey.mxd, 12/18/08, dpase



nterbedded Clays, Silts and Sands

Medium Dense to Dense Silty Sands

SUBSU



nterbedded Clays, Silts and Sands

Medium Dense to Dense Silty Sands

Medium Dense Silty Sands

Clays and Silts Interbedded with Sand Layers

SUBSU