APPENDIX C
LIQUEFACTION ANALYSIS

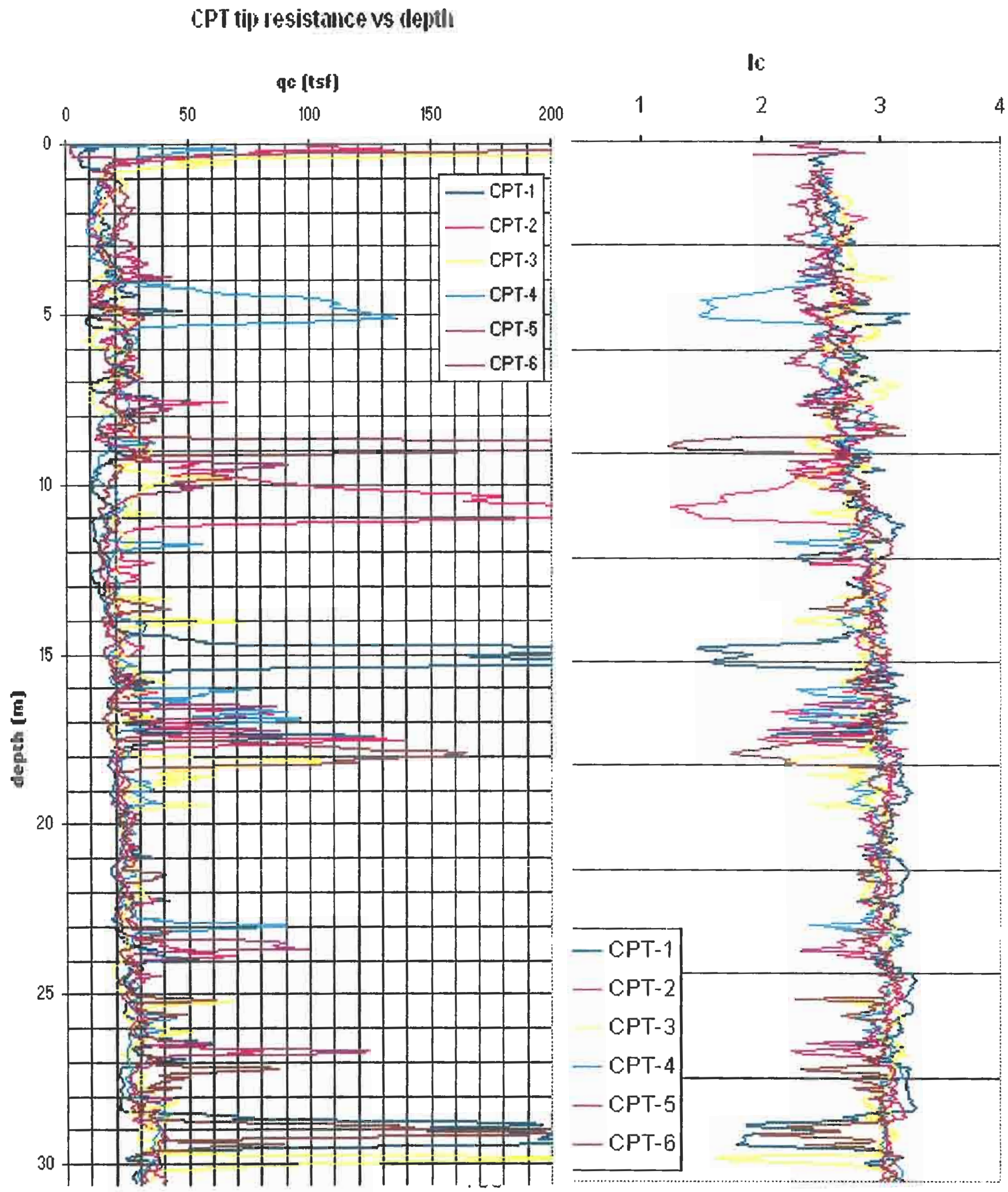
Liquefaction may occur in some soils during cyclic shaking. During shaking, soil is subjected to increased pore water pressure that may reduce the soil shear strength and resistance to cyclic and static shear stresses in the soil. This phenomenon is known as liquefaction. Liquefaction may cause loss of bearing capacity, sand boils, lateral spreading and post-earthquake ground settlement. It is generally accepted within the engineering community that loose to moderately dense, saturated, non-cohesive soils (sands and silts) are most susceptible to liquefaction. In general, soils with the highest liquefaction potential are generally found in loosely placed artificial fill and younger alluvial soils. The project site is overlain by significant thickness of alluvial deposits, and is located within an area zoned by the State of California as having potential for seismically induced liquefaction hazards.

Liquefaction is a highly researched topic within the geotechnical community. Some recently published research indicates that some low plasticity clay soils may have undergone significant strength loss during the large 1999 Kocaeli earthquake in Turkey. Case studies of liquefaction after other large earthquakes have not indicated that clay liquefaction is a widespread phenomenon. Ongoing research indicates that the cyclic behavior of clayey soil is more complex than that of sand and silt, and depends on many factors including soil plasticity, shear strength, stress history, mineralogy, sensitivity, and other factors.

Subsurface Conditions Encountered

As discussed in the "Subsurface" section of our geotechnical report. The site subsurface profile generally consists of artificial fill over interbedded layers of sands, silts and clay alluvial soils. In general, most of the soils appear clayey. The sands and silts encountered were generally stiff, and the sandy soils ranged in consistency from medium dense to very dense. As demonstrated by the CPTs across the site, the tip resistances and soil behavior index (I_c) generally follow the same profile, indicating a generally uniform soil pofile, with the exception of a few interbedded sand layers, were the tip resistance and soil behavior index increase and decrease, respectively. Some soil layers encountered were determined to have potential for liquefaction, as discussed below.

As discussed in the "Ground Water" section of this report, we used a design ground water level of about Elevation 4 feet.



Methods of Analysis and Results

For our analysis soils were divided into soils with clay-like behavior and soils with sand-like behavior, based on Plasticity Index laboratory testing and CPT soil behavior index interpretations (I_c). In general, soils with a PI less than 7 or an I_c less than 2.4 were considered to behave more like a sand than a clay and were evaluated using sand guidelines. Soils with a PI of 7 or greater (Boulanger and Idriss, Report UCD/CGM-04/01, 2004) or an I_c greater than 2.6 were considered to behave as a clay. In general, for soils with an I_c between 2.4 and 2.6 are difficult to classify without site correlation. NCEER (2001) liquefaction guidelines recommend that "Because the relationship between I_c and soil type is approximate ... soils with an I_c of 2.4 or greater should be sampled and tested, to confirm soil type and test liquefiability using other criteria."

For soils encountered in the CPTs that have an I_c between 2.4 and 2.6, adjacent and nearby boring samples and laboratory test results were used to correlate clay-like and sand-like soil behavior, and evaluate liquefaction potential accordingly. Based on the generally a uniform clayey profile as discussed above, as illustrated by CPT data, and based on the data collected from our correlation borings RW-4 (performed adjacent to CPT-4) and H-11 (performed adjacent to CPT-5), we have judged that for this project site, soils are generally clayey for a CPT I_c greater than about 2.4.

Our liquefaction analysis was performed on soils that were judged to have sand-like soil behavior based on plasticity index testing and soil behavior index CPT interpretations. In general, it was judged that the site clay soils were generally too stiff to undergo cyclic failure, and were not likely to cause significant ground problems at the site, however, some clayey layers between the depths of about 35 and 40 feet were judged to be soft enough to be subject to cyclic strain softening. Deep foundation recommendations presented in this report include 5 feet of reduced strength soil to account for liquefaction and clay softening. Below, our discussion of our liquefaction analysis of sand-like soils is presented.

Liquefaction Analysis for Sand-Like Soil Behavior

Methods for evaluation of liquefaction potential for soils with sand-like behavior are relatively well established. Our liquefaction analyses followed the methods presented by the 1998 NCEER Workshops (Youd, et al., 2001) in accordance with guidelines set forth in CDMG Special Publication 117 (CDMG, 1997). The NCEER methods for SPT and CPT analyses update simplified procedures presented by Seed and Idriss (1971). The analysis method compares the cyclic resistance ratio (CRR) with the earthquake-induced cyclic stress ratio (CSR) at different depths due to the estimated earthquake ground motions. The relationship for CSR is presented as follows:

$$CSR = 0.65\ (a_{max}/g)(\sigma_{vo}/\sigma'_{vo})r_d$$

where a_{max} is the peak horizontal acceleration at the ground surface generated by an earthquake, g is the acceleration of gravity, σ_{vo} and σ'_{vo} are total and effective overburden stresses, respectively, and r_d is a stress reduction coefficient. CRR is a function of the soil density and grain characteristics.

The factor of safety (FS) against liquefaction is expressed as the ratio of the cyclic resistance ratio (CRR) to the cyclic stress ratio (CSR). If the FS is less than 1.0, the soil is considered to be liquefiable during seismic shaking.

$$FS = CRR/CSR$$

We evaluated the liquefaction potential of the sand-like soils using a peak ground acceleration of 0.53g and a design earthquake magnitude of 7.5Mw. This is consistent with our site response analysis for the site and maps published by the California Geologic Survey for the area. The ground shaking parameters used are estimated to be representative of a seismic event having a 10 percent chance of exceedance in 50 years.

We corrected the field SPT blow counts from our boring for overburden, stress reduction versus depth, fines content, hammer energy ratio, boring diameter, rod length and sampling method (SPT sampler without liners). Our CPT tip pressures were corrected for overburden and fines content. The CPT method utilizes the soil behavior type index (I_C) and the exponential factor "n" applied to the Normalized Cone Resistance "Q" to evaluate how likely a layer is to contain significant plastic fines and have a low liquefaction potential. Cyclic Resistance Ratios (CRR) were calculated for both SPT and CPT methods using normalized "N" values and CPT tip pressures corrected to clean sand values and the SPT and CPT clean sand base curves presented in the NCEER method. The CRRs were then corrected for the design ground water level and magnitude scaling factors. Estimates of volumetric change and settlement were determined by the Ishihara and Yoshimine (1990) method. As discussed in the SCEC report, differential movement for level ground, deep soil sites, are estimated to be on the order of half the total estimated settlement.

Our analysis includes estimation of liquefaction based on rotary wash SPT and the CPT data. As discussed in the NCEER conference proceedings liquefaction evaluation techniques have not been verified below a depth of about 15 meters, due to limited case study information. Our sand-liquefaction analysis, includes evaluation of the entire soil profile. However, liquefaction that occurs below 50 feet is not judged to be able to cause significant ground deformation due to the thin and discontinuous layering. However it is our opinion that some strength loss in these deeper layers could occur. Below, we have summarized the results of our analysis.

Table C-1. Results of Liquefaction Analyses – SPT Method

Boring No.	Depth to Top of Layer (feet)	Thickness of Layer (feet)	*SPT (N_{160CS})	Factor of Safety	Potential for Liquefaction	Design Effects
RW-1	75	4	13 to 21	0.5	Likely	Some soil strength loss could occur in about 4 feet of soil
RW-2	62	5	28	0.9	Likely	Some soil strength loss could occur in about 5 feet of soil
RW-3	134	3	23	0.9	Likely	Some soil strength loss could occur in about 3 feet of soil
RW-4	14	4	21	0.5	Likely	1½-inches of total ground settlement could occur, Some pile downdrag could occur
RW-5	No liquefiable soils encountered					
RW-6	118	3	18	0.6	Likely	Some soil strength loss could occur in about 3 feet of soil
RW-7	No liquefiable soils encountered					
RW-8	32	3	26	0.5	Likely	¾-inches of total ground settlement could occur
RW-9	No liquefiable soils encountered					
RW-10	No liquefiable soils encountered					

* SPT blow counts corrected for overburden, sampling methods, and fines content

Table C-2. Results of Liquefaction Analyses – CPT Method

CPT	Depth to Top of Layer (feet)	Layer Thickness (feet)	q_{C1N-cs}	Soil Behavior Index (I_c)	Potential for Liquefaction	Design Effects
CPT-1	15.9	1/3	111	2.4	Likely	About 1¼ inches of total ground settlement could occur,
	24.3	1	109	2.4	Likely	
	39.7	¾	35	2.4	Likely	
	49.0	¾	153	1.9	Likely	
	50.4	1/3	140	1.9	Likely	Some strength loss could occur in about 3 feet of soil
	56.9	½	118	2.2	Likely	
	94.3	2/3	133	2.0	Likely	
	95.5	1½	128	2.0	Likely	
CPT-2	24.8	<¼	141	2.4	Likely	About ¾-inches of total ground settlement could occur
	31.3	½	114	2.4	Likely	
	32.2	1 2/3	145	2.2	Likely	
	34.0	2/3	150	1.7	Likely	
	36.3	½	131	2.0	Likely	
	54.6	½	109	2.3	Likely	Some strength loss could occur in about 1 ½ feet of soil
	55.9	<¼	99	2.4	Likely	
	57.3	2/3	126	2.1	Likely	
CPT-3	28.7	1	72	2.4	Likely	About 1-inch of total ground settlement could occur
	31.7	1½	96	2.4	Likely	
	45.9	½	101	2.3	Likely	
	59.2	¾	122	2.3	Likely	Some Strength loss could occur in about 2 feet of soil
	61.0	<¼	100	2.4	Likely	
	63.8	<¼	92	2.4	Likely	
	97.6	¾	132	1.9	Likely	
[1]CPT-4	14.0	3¾	109	2.1	Likely	About 1 ¼ inches of total ground settlement could occur some pile, downdrag may occur
	38.6	½	73	2.3	Likely	
	52.7	2/3	103	2.4	Likely	Some strength loss could occur in about 2 feet of soil
	54.6	½	134	2.4	Likely	
	55.3	½	125	2.3	Likely	
	75.3	<¼	134	2.4	Likely	
[2]CPT-5	13.7	2¾	46	2.4	Likely	About 1½ inches of total ground settlement could occur, some pile downdrag may occur
	30.7	<¼	141	2.2	Likely	
	54.1	<¼	135	2.4	Likely	Some strength loss could occur in about 2 feet of soil
	56.4	½	95	2.3	Likely	
	77.6	1/3	119	2.4	Likely	
	86.8	<¼	80	2.4	Likely	
	87.4	½	127	2.3	Likely	
CPT-6	28.4	<¼	97	1.9	Likely	About ½ inch of total ground settlement could occur
	29.7	½	110	1.9	Likely	
	57.8	2 1/3	124	2.1	Likely	Some strength loss could occur in about 3½ feet of soil
	82.5	<¼	72	2.3	Likely	
	89.2	<¼	106	2.4	Likely	
	94.7	1/3	136	2.3	Likely	
	95.5	1/3	136	2.0	Likely	

[1] RW-4 performed adjacent to CPT-4 and was used to correlate CPT interpretations with laboratory test results and visual observations.
[2] H-11 performed adjacent to CPT-5 and was used to correlate CPT interpretations with laboratory test results and visual observations

Summary of Results

Or analysis indicates that up to approximately 1½-inches of post earthquake liquefaction settlement could occur at the ground surface due to liquefaction. With up to ¾-inch of differential settlement due to seismically-induced liquefaction. Where thicker sand layers appear somewhat continuous such as at depths of about 15 feet in CPT-4, and CPT-5, we judge that downdrag may develop on piles due to post-earthquake seismically induced liquefaction settlement. Some loss of strength of soils below about 50 feet may also occur, and are included in allowable pile capacities. Details for design are presented in the "Foundations" section of this report. The surficial non-liquefiable layers at the site are judged to be thick enough to prevent ground rupture.