<u>C A U T I O N!!</u>

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APPENDIX C(?)

LIQUEFACTION POTENTIAL ASSESSMENT AND EVALUATION OF EARTHQUAKE-INDUCED SETTLEMENTS

TABLE OF CONTENTS (Appendix C)

1.0 INTRODUCTION

Based on the attached map of California Seismic Hazard Zones-Whittier(?) Quadrangle-, the project site falls within an area ("green" zone) requiring an investigation to assess the potential for liquefaction. Therefore, we proceeded to perform analyses for this purpose.

For our liquefaction potential and seismic settlement analyses for the site, we performed 8 Cone Penetrometer Tests (CPTs) to a depth of approximately 50 feet to generate data necessary for the analyses. The CPTs were performed by Holguin-Fahan & Associates, Inc.. Copies of their CPT Interpretation files used in our analyses are included in **Appendix A.** Based upon the CPT data and our exploratory borings, subsurface soil conditions consist of [interbeds of loose to medium dense silty sands and relatively clean sands, and stiff to very stiff sandy silts and sandy clays within the upper ?? ft., underlain by dense sands and gravels to the depths explored. A state-of-the-practice computer program **EQLique&Settle"2" (2003/2005)** was used for our analyses. For assessment of liquefaction potential and earthquake-induced settlements, CPTs are advantageous over borings because provide a continuous soil profile with depth, are fast and therefore more locations can be explored thus providing a better coverage of information at the site, and overall reduce exploration costs because reduce the time and laboratory testing and interpretation costs. CPTs are very useful for these evaluations under high existing and/or historic-high groundwater conditions.

Analyses using CPT data depart from the premise that **N-Spt values obtained from CPT data are reasonable** (Robertson, February 2005 Seminar, Long Beach). ***This method of liquefaction evaluation is considered acceptable according to the NCEER report (Youd and Idriss, 1997)".*** Once N-Spt data are automatically obtained from CPT tip resistance, the rest of the analysis continues in the same manner as when blow counts are obtained from a boring. However, some data of soil types may be refined (reconciled) in the CPTs from data obtained from a boring drilled close to the CPT or a push-probe method of sampling used by the same CPT contractor. **EQLique&Settle"2"** has the capability to easily reconcile (at the option of the user) CPT data with actual test results at specific depths from a boring or push-probe near a CPT location. Also, the program has the capability to easily apply the Chinese Criteria at specific depths for analyses based on CPT data. For our analysis, CPT-01 was reconciled using soil type data from the nearby Boring B-1.

Analyses performed by **EQLique&Settle"2"** use simultaneously the historically highest and the existing groundwater levels. This capability is incorporated in the program because the existing water level is <u>also</u> needed for the calculation of the effective overburden pressure at <u>the time of the exploration</u> for the normalization (by the Cn factor) of the blow count that affects the Cyclic Resistance Ratio (CRR), and also to take into account that when the water is raised to the historic high level, changes in the analysis should be due only to <u>changes</u> in the effective overburden stress between the existing (or lower) and the historic high ground waters (per SCEC on DMG SP 117 – See a clarification on this aspect in the file "Introfeatures.doc" included in the "EQLique&Settle 2" CD). The effective overburden stress at any point (below the existing ground water, or between the existing and the historic ground waters, depending on the depth of analysis) is used to multiply the Cyclic Resistance Ratio (CRR) to obtain the Cyclic Resistance (CR), and also to divide the Cyclic Stress (CS) to obtain the Cyclic Stress Ratio (CSR) for the calculation of estimated earthquake-induced settlements.

2.0 DATA AND ASSUMPTIONS

Based on earthquake hazards analysis performed using Thomas Blake's computer programs (2003), a Site Earthquake Magnitude of 6.9 (?) generating an estimated peak horizontal ground acceleration (PGA) of 0.53g (?) (see attached chart), and a horizontal acceleration of 0.43g (?) corresponding to an earthquake with a reference magnitude of 7.5 were used for the liquefaction and earthquake-induced settlement analyses. These data are discussed in the main body of the report.

Basic information included soil types (Soil Behavior Type – SBT) and blow counts obtained from CPT data. For analysis, a historic high ground water depth of ?? feet was used. This depth was obtained from California State historically highest ground water maps (see attached). Existing ground water was encountered at a depth of approximately ?? feet at the time of the exploratory borings. As indicated above, soil types from CPT data were reconciled with data from Boring B-1, including a change due to a sandy fine-grained soil with a value of % passing the 5μ size less than 15 (application of the "Chinese Criteria"). Data of blow count and % Fines are plotted versus depth on **Figures Liq-1** and **Liq-2, graphs (a)** and **(b).**

3.0 ASSESSMENT OF LIQUEFACTION POTENTIAL

3.1 Method of Analyses

Liquefaction is a phenomenon whereby a saturated granular soil temporarily loses its strength because of the buildup of pore water pressure during seismic excitation. This loss of strength may cause structures founded on these soils to experience subsidence and/or lateral movement.

Seed et al. (1984) originally presented design curves based on <u>field performance</u> during earthquakes (cases where evidence of liquefaction has been observed), which provides a higher level of confidence in the analyses results. After 1984, liquefaction analysis procedures have evolved in recent years based on additional research. These state-of-the-practice procedures were used for this report by the computer program. Liquefaction potential curves distinctly separate liquefaction and non-liquefaction. Therefore, liquefiable zones in this analysis are based upon the premise that liquefaction potential exists if the resisting stress (Strength) against liquefaction is less than the earthquake-induced stress; this is to say, a stress ratio or factor of safety less than unity. However, reviewing agencies may require a higher safety factor for identification of liquefiable layers; hence, for analysis, the computer program adds a safety factor greater than one with a single click (a value of 1.25 is a reasonably high safety factor). Also, the earthquake-induced settlement analysis considers zones that may settle somewhat even though the factor of safety may be greater than one (i.e. a theoretically "non-liquefaction" condition).

As mentioned above, liquefaction potential analyses were performed using the state-of-the-practice computer program **EQLique&Settle"2"**, which incorporates the latest developments in the analysis procedures (CDMG, 1997; SCEC/DMG SP117, 1999; Youd, Idriss, et al, ASCE

October 2001; Youd & Idriss, NCEER 1997. See specific references at the end of this Appendix). Per these procedures, the earthquake-induced stresses at any depth are estimated and compared with empirically-based stresses (strength) for sites where liquefaction has occurred. Some data provided by the liquefaction analyses are used for the evaluation of earthquake-induced settlements.

The calculation of strength against liquefaction is based on existing soil data such as effective overburden pressure, blow count and percent fines. A surcharge (psf) on the surface may also be incorporated in an analysis. Corrected SPT blow count data are utilized to determine the strength/overburden pressure ratio for 7½ magnitude earthquakes (Youd & Idriss, 2001). Correction of blow count for normalized overburden pressure is provided by Liao, et al. (1986) by the equation $Cn=SQRT(1/\sigma'_0)$, where σ'_0 = effective overburden pressure in TSF (see "INTRODUCTION"**:** Cn is determined for conditions at the time of the exploration). Based on data by Liao and recommendations by Youd and Idriss (2001), we used for analysis upper and lower limits of Cn of 1.7 and 0.4, respectively. The corrected SPT "N" is : N1=CnxN. [NOTE**:** the analysis considers a hammer/energy ratio of 60%. Thus, for analyses using borings, a correction factor CE is used to account for variability of hammers. Table 5.2 (SCEC/DMG SP117, March '99) and Table 2 (Youd, Idriss, et. al., Oct. '01) present recommended values of CE. Blow counts may be obtained using an automatic trip hammer and without a "donut" hammer. For the automatic trip hammer, a value of CE = 1.25 is the average value from Table 5.2].

Correction for fines is provided by Youd & Idriss (NCEER, 1997) and SCEC (1999). This correction provides equivalent blow count data for clean sand, which is ultimately used to assess strength against liquefaction. Lower limit, conservative values of % Fines are used for SBTs from CPT data. For more accuracy, the program "EQLique&Settle2" has the capability to easily reconcile values of % Fines at specific depths from actual test results on samples from a correlation boring drilled next to one of the CPTs. Also, based on hydrometer test data, reconciliation can readily be performed in the program if the Chinese Criteria applies. Other correction factors presented by Youd, Idriss, et al., (2001) are incorporated in the computer program.

The earthquake-induced stress is calculated per the following equation by Seed (1984):

Earthquake-induced stress = $\tau_{av} = 0.65.(a_{max}/g).\ \sigma_0.r_d$

Where: a_{max}/g = ratio of peak ground acceleration and acceleration of gravity.
σ_0 = Total overburden pressure
r_d = Stress reduction factor to account for soil deformability (Seed & Idriss, 1970, and Youd, Idriss, et al., 2001)

Theoretical factors of safety against liquefaction are calculated using corrected values of N-SPT.

3.2 Results of Liquefaction Potential Assessment

In order to assess a reasonable range of possible results, a **sensitivity** ("worst, remote case" condition) analysis or an extreme reconciliation for CPT-01 location was performed assuming the remote possibility that thick layers of sands exist as interpreted based on spaced sampling of Boring B-1 (large sample intervals in boreholes miss interlayers of fine-grained soils detected by a CPT sounding). This very conservative reconciliation of the CPT data was performed for CPT-01 which resulted in the liquefiable layers shown at the left of the "Induced Stress" plot on Figure Liq-1R1, Graph (c), and in a seismic settlement of approximately 2 inches (Figure Liq-1R1, Graph (d)).

Another reconciliation analysis was performed changing in the CPT data percent fines (and corresponding soil behavior type descriptions) only at specific depths of samples obtained and for actual sample thicknesses from Boring B-1. This resulted in liquefiable layers shown on Figure Liq-1R2, Graph (c), and a seismic settlement of 0.66(?) inch (≈0.7"?) (Figure Liq-1R2, Graph (d)). **Thus, it can reasonably be deduced that potential seismic settlements fall within the range of 0.7(?) inch and 2(?) inches.**

By inspection of CPT raw data, other locations were not analyzed since they are considered to be no more critical than the location that was analyzed.

To assist reviews by regulatory agencies or others, the induced stress is shown on the graphs for a factor of safety (FS) of 1, as well as a FS of 1.25 (this factor may be changed by the user with a single click). Liquefiable layers for either FS can be identified when the resistance to liquefaction is less than the induced stress. For this site, by examination of the graphs and for the sensitivity analysis case, it is interpreted that significant liquefiable layers occur between ?? feet and ?? feet, and between ?? feet and ?? feet, and a significant non-liquefiable layer occurs at depth between ?? feet and ?? feet.

4.0 ESTIMATION OF EARTHQUAKE-INDUCED SETTLEMENTS

4.1 Method of Analysis

Earthquake-induced settlements were estimated using procedures presented by Tokimatsu and Seed (1987) for dry/moist soils (above the water table) and saturated sands. Settlements analyses were performed for the same location analyzed for liquefaction potential to estimate the maximum possible cyclic settlement to be expected at the project site. As indicated previously, **EQLique&Settle"2"** (2005) computer program was used to calculate cyclic settlements and presents them on a graph of cumulative settlements, including settlements above and below historic high ground water depths.

Volumetric strains for soils above the water table were estimated using blow count data and cyclic shear strain (Tokimatsu and Seed, 1987). **The volumetric strain was then doubled to account for multidirectional effects** (the volumetric strain data were originally obtained from one-dimensional laboratory testing). Cyclic settlement was obtained by multiplying the thickness of the soil layer by the calculated volumetric strain.

Cyclic settlements for saturated sands were estimated using blow count data corrected for fines content (i.e. blow counts for equivalent clean sand were used) and other factors used for liquefaction analyses (Youd & Idriss/NCEER, 1997 and SCEC, 1999). The referenced procedure applies only to saturated clean sands.

Volumetric strain for saturated sands was estimated using the calculated earthquake-induced cyclic shear stress and corrected blow count (Tokimatsu and Seed, 1987). The cyclic settlement was obtained by multiplying the thickness of the liquefied soil layer by the volumetric strain.

4.2 Results of Earthquake-Induced Settlements Analyses

As previously mentioned, the results of computerized cyclic settlement analyses are presented on **Figures Liq-1R1, Liq-1R2, Graphs (d).** In this case, actual seismic settlements are expected to be less than 2(?) inches due to built-in conservatism in the procedures and the rigorous conditions analyzed in the sensitivity analysis. A maximum cumulative total cyclic settlement ranging between 0.7(?) inch and 2(?) inches has been estimated at the analyzed location. Considering procedural conservatism and the above results [optional if it applies**:** and that subsurface soil conditions are generally consistent across the site], a maximum differential cyclic settlement of 1.5(?) inches [or for the option**:** 1 inch] in a span of 30 feet is estimated (SCEC/DMG SP117, 1999).

[Optional if it applies**:** settlements at greater depths were computed and apparently correspond to some liquefiable layers (by comparison of graphs (c) and (d)). These deep liquefiable layers are expected to be mitigated by upper non-liquefiable layers (per Ishihara, 1985, procedures), and the settlements at the corresponding depths, although are expected to occur, they should not propagate to the surface owing to the bridging effects from the non-liquefiable layers. However, settlements at shallower depths should be mitigated by special measures, such as**:** removal and recompaction; by-passing with piles; compaction grouting; a structural mat foundation; post-tensioned slabs; or combinations of these or other measures].

[Optional if it applies**:** earthquake-induced settlements above and below the historic high ground water level are very small or negligible, or not significant, as to require any special mitigation].

5.0 CONCLUSIONS

1. Some potential for liquefaction and earthquake-induced settlements are expected at this site. Although a maximum total settlement of 2(?) inches is expected to be the possible highest (due to the rigorous conditions used for the sensitivity analysis), the results reflect that settlements mostly occur below a depth of 20(?) feet and, from the seismic point of view, the piles recommended for the foundation should be deeper than 40(?) feet to by-pass the settlement-

prone layers. However, downdrag of the piles may occur during shaking due to loss of support by the liquefiable layers. Downdrag effects are incorporated in the pile capacity chart provided in this report.

2. A maximum seismic differential settlement of 1.5(?) inches in a distance of 30 feet may be incorporated in the design of foundations alternative to piles (e.g. Mat foundation or Post-Tensioned Slabs in conjunction with a removal and recompaction of 10 feet and/or compaction grouting).

3. Even based on the results from the rigorous reconciliation analyses, and per Ishihara's procedures (1985), no potential for surface manifestation (e.g. sand boils or significant ground fissures) as an effect from movement of layer(s) below the historic high groundwater is expected at this site. This is considering that significant liquefiable layers only start at a depth of 19(??) feet and a thick non-liquefiable layer is present immediately below a depth of 26(??) feet with a thickness greater than 6(?) feet, to an approximate maximum thickness of 9(?) feet.

4. Based on SCEC (1999) guidelines, a **potential for loss of bearing capacity** due to liquefaction is not expected at the site since there is not an upper potentially liquefiable layer at a depth shallower than the estimated depth where the induced vertical stress in the soil is less than 10% of the bearing pressure imposed by the proposed alternative foundation systems (a mat or PT slabs). Furthermore, these systems are designed to dissipate structural loads. For the piles, this is not an issue, and no loss of bearing capacity is expected for grade beams or lightly loaded slabs-on-grade.

5. **In significant conformance with Youd, Hanson, and Bartlett** (ASCE Geotechnical Jr. April 1995, and Lecture by Youd on July 7, 1999), **<u>no lateral spreading</u>** due to liquefaction is expected at this site due to the following reasons:

   - Alluvial subsurface soils are essentially horizontally layered.
   - There is not a free-face toward which liquefied soils could move laterally.
   - [Optional if it applies to the case of minor liquefiable layers and minor settlements**:** liquefaction potential and associated settlements are considered to be not significant at the site. It should be noted that the settlement calculations include multi-directional effects in the volumetric strains].
   - [Optional if it applies**: in the liquefaction spreadsheet (a part of the computer program)** it can be observed that at an analyzed location, saturated **liquefiable** sands with values of **N1(60) <15** within the upper 10 meters (30 feet) have a cumulative thickness (**T_{15})** less than 1 meter (3 ft), or at other locations where T_{15} is greater than 1 meter, these locations are scattered or have minimal occurrence throughout the site, and cannot reasonably be <u>connected</u> to form a hypothetical "continuous" line of significant length that could reasonably be expected to "exit" on a slope or a free-face, or move significantly below the gentle slope of the site].
   - [Optional if it applies**:** a layer identified as potentially prone to lateral spreading is at a depth (measured from the top of the free-face) greater than two times the height of the free-face. This layer is not expected to shear to cause significant lateral spreading (Youd at the 7/7/99 Lecture).

6.0 RECOMMENDATIONS

Measures to mitigate soil settlements are discussed in the main text of this report, which consist of piles, or a mat foundation or PT slabs, or removal/recompaction, and/or compaction grouting. For the mat or the PT slabs, a minimum removal and recompaction of 10(?) feet is recommended, and a maximum differential seismic settlement of 1.5(?) inches [or 1(?) inch] in a distance of 30 feet may conservatively be used for foundation design. Since a 10(?)-foot cap of recompacted soils is recommended, the total static settlement is expected to be essentially uniform, and the differential static settlement non-significant or negligible. Thus, for final foundation design, a maximum combined (seismic and static) differential settlement of 1.5(?) inches [or 1(?) inch] in a distance of 30 feet may be used. [or in the case of piles**:** a chart for pile design by the structural engineer is included in this report].

[Optional if it applies**:** owing to the low magnitude of estimated conservative earthquake-induced differential seismic settlement, special measures to mitigate cyclic settlements are considered to be unnecessary].

7.0 REFERENCES SPECIFIC TO THESE ANALYSES (see at the end of this report)

ATTACHMENTS: