10 Mar 1991, 1:00 pm - 3:00 pm

Liquefaction and Surface Settlement in the Marina District

D. Rosidi
ENGEO Incorporated, Concord, California

W. B. Wigginton
ENGEO Incorporated, Concord, California

Proceedings: Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, March 11-15, 1991, St. Louis, Missouri, Paper No. LP19

Liquefaction and Surface Settlement in the Marina District

D. Rosidi
ENGEO Incorporated, Concord, California

W.B. Wigginton
ENGEO Incorporated, Concord, California

SYNOPSIS This paper evaluates the liquefaction potential of sands within the Marina District. Three types of underlying potentially liquefiable soil deposits are studied: (A) Hydraulic fill, (B) Artificial fill, and (C) Strawberry Island and other modern beach deposits. A liquefaction analysis for each of these deposits is conducted. It is suggested that a Peak Ground Acceleration (PGA) of approximately 0.1g is needed in the Marina District to initiate liquefaction. Relations between PGA and thickness of liquefiable soil, as well as potential surface settlement are presented. These relationships are useful for the study of expected performance of the Marina District in future earthquakes. The results can also be used to evaluate possible ground modification methods, utility design, or structural retrofit designs.

INTRODUCTION

Liquefaction, manifested by sand boils and ground settlement, was prevalent in the Marina District of San Francisco during the 1989 Loma Prieta quake. Studies by USGS and UC Berkeley Earthquake Engineering Research Institute have shown that the widespread damage is associated with the liquefaction. Settlement, structural distress to buildings, pavement damage, and damage to the lifeline utilities were observed and well documented. The Marina District of San Francisco provides a unique opportunity for engineers to better assess the problem of seismically-induced ground failures.

This paper attempts to quantify for various microzones the severity of liquefaction and the accompanying ground settlement. The earthquake shaking parameter is represented by the Peak Ground Acceleration (PGA), although other better quantitative parameters such as Root Mean Square (RMS) or spectral acceleration can be used.

SUBSURFACE CONDITIONS

Detailed investigations of the subsurface soil conditions at the Marina District are documented elsewhere (Bonilla, 1990; Bennett, 1990). In general the sandy soils beneath the district can be grouped into four categories: Hydraulic fill, Artificial fill, Strawberry Island (modern beach) deposits, and Dune sand (native soil).

Historic records for the 1906 San Francisco and the 1989 Loma Prieta earthquakes do not indicate liquefaction occurred within the Dune sand areas which are typically at higher elevations. Previous studies on liquefaction susceptibility for the Dune sand deposits surrounding the Marina District have shown little damage potential for this relatively dense natural deposit (Roth and Kavazanjian, 1984). Therefore, Dune sand deposits will not be considered further in this study.

Within the damage affected area (see Figure 1), the subsurface sandy soils are divided into three zones which reveal a varying resistance to seismic loading: (A) Hydraulic fill, (B) Artificial fill, and (C) Strawberry Island (modern beach) deposit. A representative profile for each soil type used for the analysis is shown on Figure 2 along with mean grain size and typical SPT blow-counts.

LIQUEFACTION ANALYSIS

Liquefaction is a phenomenon where soil becomes a liquid-like material as the result of pore water pressure build-up. The effects to the ground and structures built upon it depend upon the thickness and depth of liquefiable layers. For example, if liquefaction occurs within a limited zone deep below the ground surface, the effect to structures built on or near surface may not be significant. For such an event, sand boils and surface settlement often are not observable.

An assessment of liquefaction, therefore, should involve the determination of the thickness of potentially liquefiable soil, and its location in respect to the ground surface. Analysis for a specific point or depth, as is commonly done in practice, does not offer enough information for a complete assessment.

In this study, a procedure proposed by Iwasaki, et al. (1978) was employed. Liquefaction Resistance Factor, FL (equivalent to the factor of safety) is defined as the ratio of "shear resistance" to the "induced shear stress" caused by earthquake shaking. The "induced shear stress", L can be approximated as (Seed and Idriss, 1982):

$$L = a_{max}\frac{\sigma}{\sigma'}r_d \qquad (1)$$

Where a_{max} is the PGA in g, σ and σ' are the total and effective overburden pressures, respectively, and r_d is the stress reduction (correction) factor for the rigid body assumption.

The "resistance" of the soil to shaking, R, is given by the following equations which were developed based on laboratory cyclic loading tests:

$$for,\ 0.02 \leq D_{50} \leq 0.6mm$$

$$R = 0.0882\sqrt{\frac{N}{\sigma'+0.7}} + 0.225\log_{10}\frac{0.35}{D_{50}} \qquad (2)$$

$$for,\ 0.6 < D_{50} \leq 2.0mm$$

$$R = 0.0882\sqrt{\frac{N}{\sigma'+0.7}} - 0.05 \qquad (3)$$

N is the penetration blow-count, and D_{50} is the mean diameter of soil particle in mm. The units of R and σ' are in kg - force/cm^2.

This procedure has been shown to provide good predictive capability when compared to field observations on several earthquake occurrences in Japan (Iwasaki, 1986). It is appropriate, then, that analysis using this procedure should use the Japanese Standard blow-count, N_J. Seed, et al. (1985) have shown that the Japanese blow-count has an average of 67% energy efficiency. Moreover, there are also differences in the borehole diameter and the rate of hammer drops. The following relation was used to convert any U.S. blow-count to N_J:

$$N_J = \frac{E_{f1}}{67}\frac{10}{9}N_{f1} \qquad (4)$$

Where E_{f1} is the energy efficiency in percent of the sampling operation, and N_{f1} is the corresponding blow-count encountered. The factor of 10/9 is the correction for the hole diameter and hammer drop rate. The corrected blow-counts for each of the typical soil profiles are shown on Figure 2.

PGA VS. THICKNESS OF LIQUEFIABLE LAYER

It is suggested by our study that a PGA between 0.05 to 0.1g is needed to initiate the process of liquefaction, with approximately 0.1g needed to produce any observable damage at the surface.

Relations between the thickness of liquefiable sand layer and PGA are shown on Figure 3. The Hydraulic fill appears to have liquefaction of its total thickness at the lowest PGA of the three profiles studied. The entire thickness of saturated sand can liquefy under a PGA of approximately 0.20g, 0.23g, and 0.30g for the Hydraulic fill, Strawberry Island, and Artificial fill, respectively. It should be noted that a PGA of 0.21g was measured for the 1989 Loma Prieta earthquake 1½ miles to the southwest of the Marina District in the Presidio. This exceeds a PGA of 0.13g measured 3 miles to the southwest of the Marina District in Golden Gate Park during the 22 March 1957 Daly City Earthquake, the largest nearby post-1906 earthquake before the 1989 Loma Prieta event.

Since the computations do not take into account earthquake duration as represented by an equivalent number of uniform cycles, the short duration of 15 seconds for the Loma Prieta earthquake could explain the lack of liquefaction of the full thickness of the Hydraulic fill. A short 5-second duration of strong motion for the 1957 Daly City earthquake might also explain the lack of surface liquefaction expression or damage in the Marina District for that earlier 5.3 earthquake.

SEISMICALLY-INDUCED SETTLEMENT

Seismically-induced settlement is one of the important factors that one has to consider for any seismic foundation retrofit or ground modification, especially for ground treatment related to underground structures. The damage to structures often depends on the amount of settlement as much as on the extent of liquefaction.

Based on a procedure introduced by Tokimatsu and Seed (1987), total seismically-induced settlement for each of the subsurface soil conditions at the Marina District is calculated at different levels of PGA. These results are shown on Figure 4. For settlement analysis, standard blow-count energy efficiency of 60%, N_{60} was used.

Actual field measurements of the "post earthquake" settlements at the nearest locations adjacent to the boreholes from which the representative soil profiles were obtained, show a surface settlement of 11 mm for the Artificial fill, approximately 25 to 30 mm for the Strawberry Island deposit and 96 to 143 mm for the Hydraulic fill (Bennett, 1990).

Contrary to the computed results shown on Figure 4, a larger average "post earthquake" settlement was observed, as a result of the 1989 earthquake, for the Artificial fill than for the Strawberry Island deposit. This discrepancy might be caused by the horizontal soil layer assumption in the analysis, whereas the actual subsurface soil layers slope towards the hydraulically-filled lagoon (Area A in Figure 1). The larger observed settlement of the Artificial fill may be a result of lateral spreading of liquefied soil and/or the variation of the subsurface conditions within this zone. As a consequence, more damage occurred to structures founded on the Artificial fill (Area B) than might be expected from the analysis used in this study.

CONCLUSIONS

Relations between PGA and dynamic settlement as well as the thickness of liquefiable sand layers have been presented. These relations show that the Hydraulic fill has the highest potential of liquefaction and surface settlement, followed by Strawberry Island and Artificial fill. A maximum seismically-induced settlement of about 175 mm is computed for the Hydraulic fill, compared to about 80 mm for the Strawberry Island, and about 50 mm for the Artificial fill. The greater damage observed for areas underlain by the Artificial fill as compared to damage observed within the Strawberry Island area may have been a result of other factors, for instance, possible lateral spreading. Secondary factors warrant further investigation. It is also suggested by Figures 3 and 4 that the saturated sandy deposits under the Marina District were not fully liquefied by the 1989 quake. Greater damaging effects to the structures can be expected during large earthquakes in the future. A PGA of about 0.1g is suggested as the threshold acceleration level necessary to produce damage in the Marina District during future earthquakes.

REFERENCES

Bennett, Michael J., "Ground Deformation and Liquefaction of Soil in The Marina District" in Effects of the Loma Prieta Earthquake on the Marina District, San Francisco, California, USGS, Open File Report 90-253, 1990

Bonilla, M. G., "Natural and Artificial Deposits in the Marina District" in Effects of the Loma Prieta Earthquake on the Marina District, San Francisco, California, USGS, Open File Report 90-253, 1990.

Iwasaki, T., et al., "A Practical Method for Assessing Liquefaction Potential Based on Case Studies in Various Sites in Japan" Proc. of Fifth Japan Symposium on Earthquake Engineering, Nov. 1978.

Roth, R. A. and E. Kavazanjian, Jr., "Liquefaction Susceptibility Mapping for San Francisco, California", Bull. Association of Engineering Geologists, Vol. XXI, No. 4, 1984.

Seed, H. B. and I. M. Idriss, "Ground Motions and Soil Liquefaction During Earthquake" EERI Monograph, 1982.

Seed, H. B., et al., "Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations" J. of Geotechnical Engineering, Vol. III, No. 12, Dec. 1985.

Seed, H. B., et al., "Preliminary Report on the Principal Geotechnical Aspects of the October 17, 1989 Loma Prieta Earthquake", Report #90/05 EERC, U.C. Berkeley, April 1990.

Tokimatsu, K. and H. B. Seed, "Evaluation of Settlements in Sands due to Earthquake Shaking" J. of Geotechnical Engineering, Vol. 113, No. 8, August 1987.

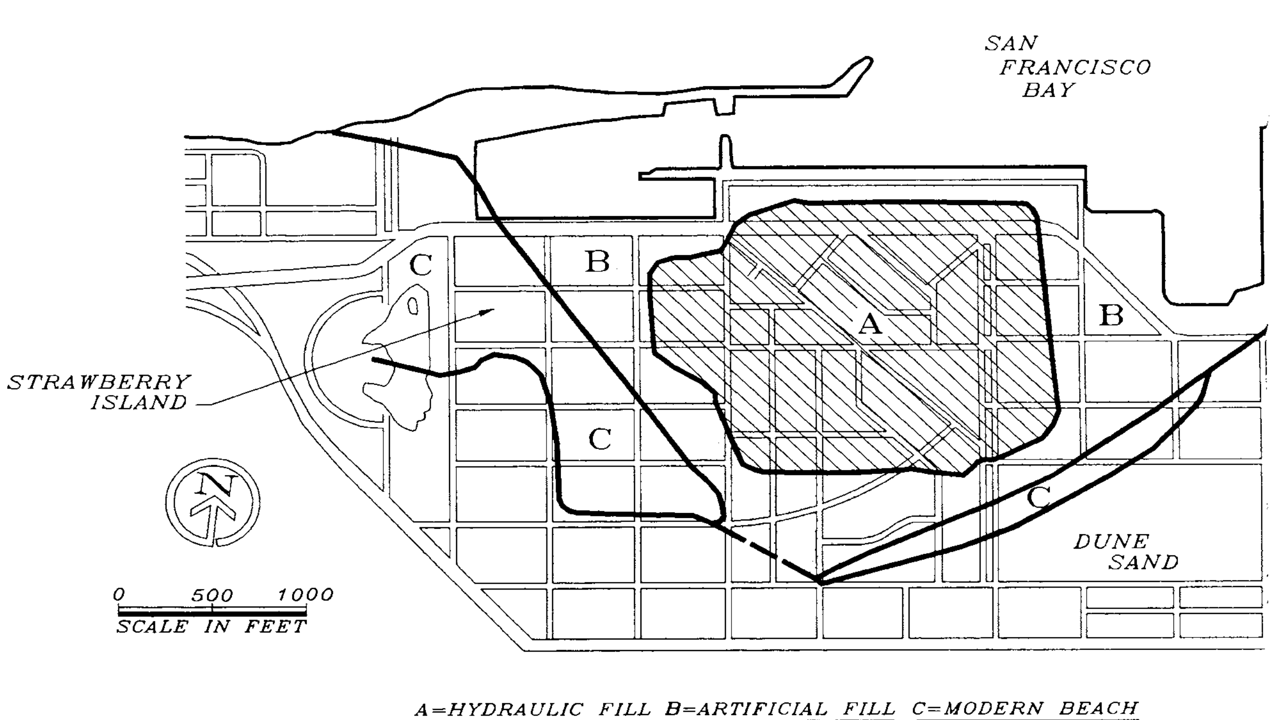

FIGURE 1 – MARINA DISTRICT ZONATION

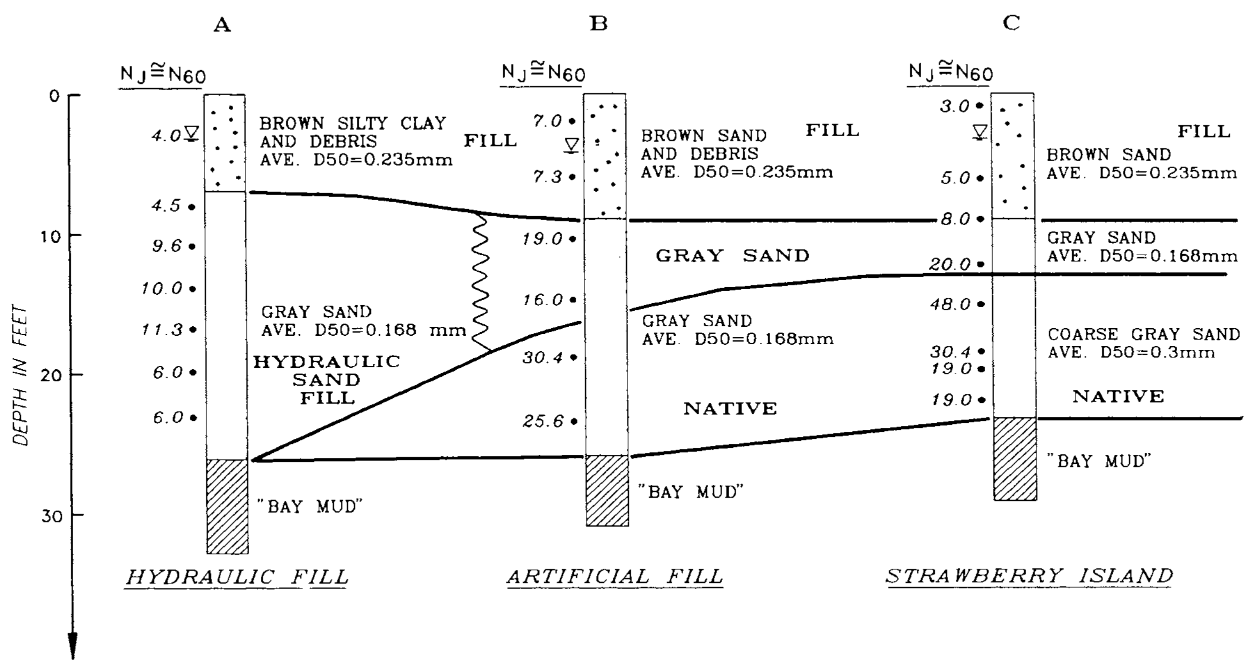

FIGURE 2 – TYPICAL SOIL PROFILES

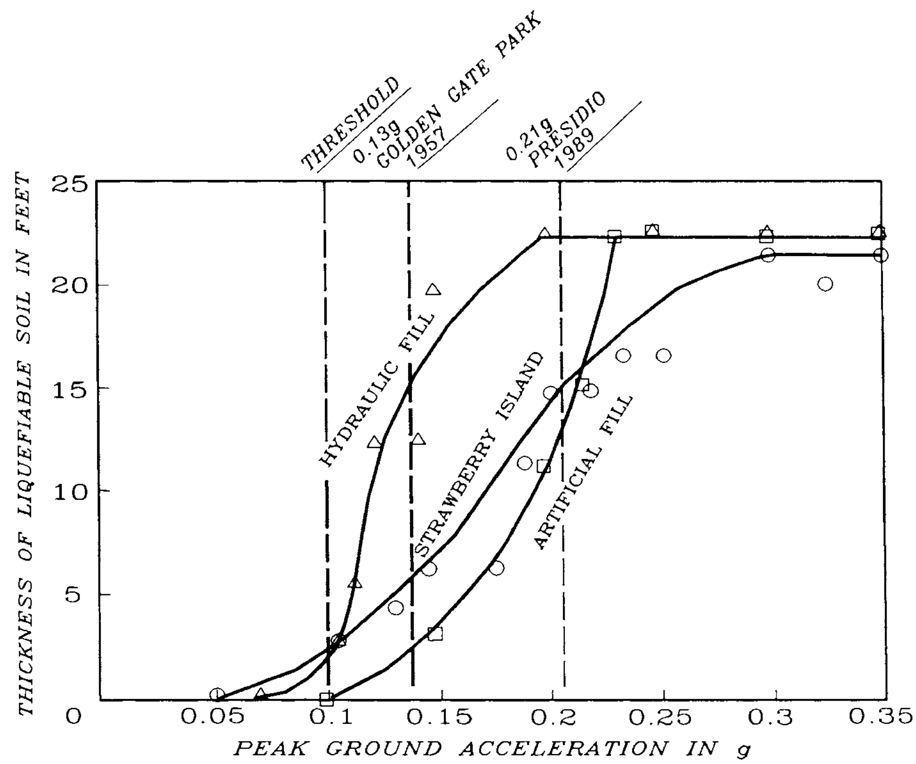

FIGURE 3 –
RELATION BETWEEN THICKNESS OF LIQUEFIABLE SAND LAYER AND PEAK GROUND ACCELERATION

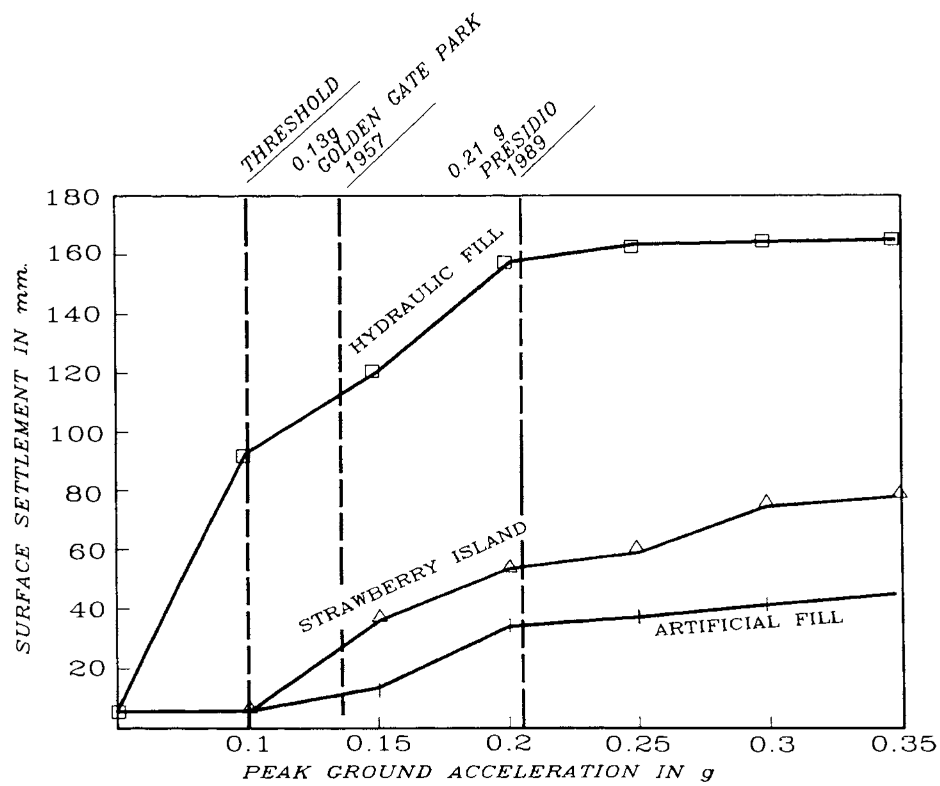

FIGURE 4 –
RELATION BETWEEN PEAK GROUND ACCELERATION AND SEISMICALLY INDUCED SETTLEMENT