# LIQUEFACTION POTENTIAL STUDY FOR A POWER PLANT SITE

SIU-MUN WOO
Associate & Senior Geotechnical Engineer
Moh and Associates, Taipei, R.O.C.

ZA-CHIEH MOH
Principal, Moh and Associates

and

CHIN-DER OU
Associate and Manager
Moh and Associates, Singapore

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by

SIU-MUN WOO
Associate & Senior Geotechnical Engr.
Moh and Associates
Taipei, Taiwan, R.O.C.

ZA-CHIEH MOH
Principal
Moh and Associates
Taiwan, Hong Kong and Singapore

CHIN-DER OU
Associate & Manager
Moh and Associates
Singapore

## ABSTRACT

This paper presents the results of a soil liquefaction potential study for a proposed power plant site. Soil liquefaction potential analyses were carried out by means of semi-empirical methods, analytical method and effective stress analysis. Evaluation of results of analyses show that localized liquefaction could occur at the proposed site when the ground motion acceleration is equivalent to or higher than 0.13 g. Compaction sand piles were recommended to be used to densify the sand deposit. The results of densification of the loose soil after installation of compaction sand piles are also reported.

## INTRODUCTION

When a saturated fine loose sand is subjected to an earthquake or other vibratory dynamic motion, the sand may lose its strength and behave like a liquid tempdrarily. This phenomenon is the so-called "liquefaction". Liquefaction of loose fine sand may be attributed to the tendency of contraction of soil volume and the consequent rapid increase of pore water pressure. Hence, soil liquefaction has caused settling, tilting and failure of many civil engineering structures, and has drawn the attention of many geotechnical engineers in the last decade.

A thermal power plant of the Taiwan Power Company is being constructed at the Hsin-Ta-Kong site in the south-western part of Taiwan. The island of Taiwan is located in the active seismic zone of the Pacific Ocean and has experienced over thousand earthquakes every year. Since the site has a layer of loose sand near the ground surface which may be susceptible to liquefaction during earthquakes, a detailed soil liquefaction potential investigation at the site was carried out. Prior to this study, it was not an uncommon practice of evaluating soil liquefaction potential for sites of major projects in Taiwan. However, practically all the previous liquefaction potential evaluations were made on the basis of empirical methods (i.e. SEED and IDRISS, 1967). For the present study, liquefaction potential of the soil deposit was evaluated by using a number of different approaches, including empirical methods based on Standard Penetration Test results and analytical methods employing dynamic soil testings and effective stress model analysis. Methods of soil improvement for reducing the liquefaction potential of the site were evaluated. The effectiveness of the recommended remedial measure is discussed.

## SEISMICITY OF THE SITE

According to a study by TSAI et al (1977), the area of Taiwan can be separated into 3 seismic zones, namely the North-East Seismic Zone (NESZ), the East Seismic Zone (ESZ) and the West Seismic Zone (WSZ). The boundary lines between the 3 zones are shown in Fig. 1. The Hsin-Ta-Kong site is located in the southern part of the West Seismic Zone. Figure 1 also shows the epicenter distribution of earthquakes with magnitude larger than 7.0 on the Richter scale occurred during the period of 1897 to 1977. It can be seen that there were 3 earthquakes with epicenters in the ESZ and 12 earthquakes in the ESZ. The earthquake epicenter nearest to the Hsin-Ta-Kong site was located only about 60 km away. In addition, many minor earthquakes were detected within a distance of 20 km to the site. Table 1 lists some of the seismic information relating to the site.

The maximum ground acceleration at a site depends on the magnitude of earthquakes, the location of hypocenter (epicenter distance and focal depth), and the geological structure of the site. For the site concerned, the maximum ground accelerations for a return period of 50 years, as suggested by HSU (1975), MAU et al (1978), and TSAI et al (1977), are listed in Table 2. An average value of 0.13 g was adopted for analysis in this study.

## SITE INVESTIGATION AND LABORATORY TESTINGS

The plant site which is about 1 sq km in area is situated on an alluvial beach parallel to the coastline of the Hsin-Ta-Kong. The topography of the site at the time of investigation varied from El. + 5.5 m to El. + 0.3 m above mean sea level. Field exploration and sampling work were carried out before a 4 m hydraulic fill was placed over the site. A total of 7 boreholes was drilled, of which 3 boreholes of 30 m to 50 m depth were used to establish the soil profile and to perform Standard Penetration Tests (SPT). The other four boreholes were used for taking undisturbed soil samples. Extreme care was exercised in performing the SPT in order to obtain accurate blow counts. Since the upper layer of the sand deposit at the site is in a loose state, sampling by ordinary method, if possible, would have undesirable disturbance on the unstable soil structure. In this study, a special hydraulically opened piston sampler called Osterberg sampler was utilized to obtain undisturbed samples. In order to avoid soil disturbance during transportation to the laboratory for testing, a quick freezing technique was used. After the free water in the sampler was completely drained, the tube containing soil sample was immersed into liquid nitrogen for freezing. The frozen samples were then kept in a sample box packed with dry ice while being transported. Various laboratory testings were performed on the samples to determine the physical properties and engineering characteristics of the soil deposit. In addition to the conventional direct shear tests, cyclic triaxial tests were performed on the undisturbed samples to determine the pore pressure development of the soils during cyclic loading. A total of 4 cyclic triaxial tests was carried out in the Soils Laboratory of the University of Tokyo in Japan. The saturation and consolidation procedures were the same as those described by ISHIHARA et al (1978). All tests were conducted with a back pressure of 1.5 kg/cm$^2$ and an effective confining pressure of 1.0 kg/cm$^2$. Cyclic load was applied until the specimen deformed to a peak to peak axial strain of 10%. During the cyclic loading, chamber pressure was kept constant and the axial load, axial deformation and change in pore water pressure were monitored with time.

According to the information obtained from the subsurface exploration, the subsoils at the site can be divided into the following strata:

(1) Silty sand layer — From the ground surface to a depth of approximately 3 m, the soil is a layer of gray silty sand, loose to medium dense with Standard Penetration Resistance N values varying from 3 to 16. The mean grain size ($D_{50}$) ranges from 0.19 to 0.32 mm. The amount of fine-grained soils, i.e. minus No. 200 sieve, is about 12%. Since the soil grains are in a loose state, liquefaction problem may occur in this layer.

(2) Silty clay layer — Underlying the silty sand is a layer of gray silty clay of about 14 m thick. The silty clay is of very soft consistency with Liquid Limit of 43% and plasticity index of 23%. The Liquidity Index of this soil is about 1.0 or larger.

(3) Silty sand layer — Immediately below the silty clay layer is a thick layer of gray silty sand extending to the bottom of boring which is 50 m below the existing ground surface. The silty sand is of medium dense to very dense with N values varying from 20 to 80 and increasing with depth. The mean grain size ($D_{50}$) ranges from 0.075 to 0.180 mm, and the fines content is about 20%.

Since the site is located right beside the sea, the groundwater table as measured during the exploration period was about El. + 0.00 m to El. + 0.50 m.

## LIQUEFACTION POTENTIAL EVALUATIONS

### Analysis by Empirical Methods

In this analysis, the semi-empirical SEED's method (SEED, 1976) and the method suggested by NISHIYAMA et al (1977) were adopted for the evaluation of soil liquefaction potential based on N values. In SEED's method, the cyclic shear stress which would develop during an earthquake is determined by the relationship:

$$\left(\frac{\tau_{av}}{\sigma_o'}\right) = 0.65 \frac{a_{max}}{g} \frac{\sigma_o}{\sigma_o'} \gamma_d \tag{1}$$

where $\tau_{av}$ is the average horizontal shear stress induced by earthquake; $\sigma_o'$ and $\sigma_o$ are the effective and total overburden pressure respectively on sand layer under consideration; $a_{max}$ is maximum acceleration at ground surface; g is acceleration of gravity; $\gamma_d$ is a stress reduction factor which is a function of the soil depth.

The Standard Penetration Resistance, N values obtained in the field are corrected to values of $N_1$ which are penetration resistance corresponding to an effective overburden pressure of one ton per sq ft, by using the relationship:

$$N_1 = C_N \; N \tag{2}$$

in which $C_N = 1 - 1.25 \log (\sigma_o' / \sigma_1')$. Where $\sigma_o'$ is the effective overburden pressure in $t/ft^2$, where the penetration resistance has the value N; $\sigma_1'$ is the effective overburden pressure of 1 $t/ft^2$. The corrected N values (i.e., $N_1$) are plotted against the shear stress ratio ($\tau_{av} / \sigma_o'$). The values of which are compared with the stress ratio required to cause cyclic liquefaction, the so-called lower bound line. The lower bound line was established by SEED on the basis of a comprehensive collection of data. To express the result in terms of factor of safety, the following expression is used:

$$F.S. \quad = \quad \frac{\text{Stress ratio required to cause liquefaction}}{\text{Average stress ratio induced by earthquake}} \tag{3}$$

Based on SEED's method with $a_{max} = 0.13$ g, the results of analysis are shown in Fig. 2. It is obvious that in some areas in the top sand layer, the factors of safety against liquefaction are below or close to 1, indicating high liquefaction potential. After the site is raised by placing a 4 m hydraulic fill, increase of the overburden pressure will reduce the liquefaction potential. In the analysis, it was assumed that the hydraulic fill would be in a loose state. Consolidation of the in situ sand due to placement of fill was not considered. Figure 2 shows that the factors of safety against liquefaction do increase due to placement of the fill. However, at some locations, the liquefaction potential remains to be high.

Empirical method of predicting sand liquefaction suggested by NISHIYAMA et al (1977) was also adopted. In this method, thickness of the soil layer, particle size, and the N values were taken into account. The results of analysis are similar to that obtained by using SEED's method.

## Analysis by Analytical Method

There are a number of approaches to analyze soil liquefaction potential by means of analytical method, for example, FINN et al (1971), SEED and PEACOCK (1971), CASTRO (1975), and ISHIHARA (1976). The basic principle of all these approaches is similar. The liquefaction potential is evaluated on the basis of laboratory testing data and external forces estimated from simplified response analysis. The major difference among the various approaches is the correction or modification applied to the laboratory testing data in order to simulate the in situ conditions. For the present study, since the soil samples were tested in the Soils Laboratory of the University of Tokyo, the analysis suggested by ISHIHARA (1976) appears to be more appropriate to be followed.

The actual stress conditions under which an in situ soil element is subject to is quite complex. It is rather difficult to exactly duplicate the in situ stress condition. The laboratory determined soil strength would therefore be somewhat different from that in the field, corrections or modifications would be needed. If the laboratory determined value $\sigma_d$ / $\sigma_o$' were used as the reference, then the externally applied stress condition must be modified. The laboratory cyclic triaxial tests were performed under isotropic stress condition of $K_o = 1$ and the maximum stress ratio caused by earthquake can be related to the laboratory determined stress ratio by the following equation:

$$\frac{\tau}{\sigma_o'} \quad = \quad 0.55 \quad \left(\frac{3}{1 + 2K_o}\right) \quad \frac{\tau_{max}}{\sigma_o'} \tag{4}$$

According to SEED's simplified procedure, the maximum stress ratio $\tau_{max}$ / $\sigma_o$' caused by an earthquake can be expressed by:

$$\frac{\tau_{max}}{\sigma_o'} = \frac{a_{max}}{g} \gamma_d \frac{\sigma_o}{\sigma_o'} \qquad (5)$$

then

$$\frac{\tau}{\sigma_o'} = 0.55 \left(\frac{3}{1 + 2K_o}\right) \frac{a_{max}}{g} \gamma_d \frac{\sigma_o}{\sigma_o'} \qquad (6)$$

where 0.55 is a correction factor for the external stress condition and the factor $3 / (1 + 2K_o)$ is correction for the $K_o$ condition. From the above, the factor of safety against liquefaction can be expressed as:

$$F.S. = \frac{\sigma_d / 2\sigma_o'}{\tau / \sigma_o'} \qquad (7)$$

For the condition of number of uniform stress cycles $N_c = 20$, $a_{max} = 0.13$ g, and $K_o = 0.5$, the variation of factor of safety against liquefaction with depth is plotted in Fig. 3. It can be seen that under these conditions, soils in Boreholes H-1 and H-2 are susceptible to liquefaction. However, under the proposed 4 m fill, there appears to be no danger of liquefaction at the location of any of the boreholes.

Analysis by Effective Stress Model

The method of effective stress model analysis assesses the liquefaction potential of a soil deposit by response analysis of simplified materials model. The basic principle of this method is to establish the response motion of horizontally layered sand deposits depending upon soil properties. By solving the differential equations involved by means of a computer (GHABOUSSI and DIKMEN, 1977), the stress-strain behavior and pore pressure development during an earthquake excitation in soil deposit can be obtained. For a soil element changing from certain stress condition to initial liquefaction, the stress history of the soil element is represented by a soil behavior model. For the present study, a simplified soil profile was used in the analysis, since the site condition is relatively uniform. Due to lack of typical strong motion earthquake records in the Taiwan area, the surface motion of El Centro, California earthquake of 18th May, 1940 was used in the analysis.

The results of the effective stress analysis are presented in Fig. 4 and Fig. 5. The results indicate that under the existing natural ground condition, there is high liquefaction potential. It is clear from Fig. 4 that under a maximum ground acceleration of 0.13 g, initial liquefaction would occur 2.7 sec after an earthquake and the sand layer would completely liquefy after 3.8 sec. It is interesting to note that during the earthquake shocking, pore pressure would develop rapidly within a depth of 1.5 m. With a 4 m hydraulic fill, the excess pore pressure remains constant after reaching certain level, indicating that initial liquefaction would not occur. In this case, the maximum excess pore pressure developed in the soil will be only 0.6 times of its initial effective stress. Improvement of the soil condition against liquefaction with a 4 m hydraulic fill as shown by the results of the semi-empirical analysis is confirmed by the results obtained in the more sophisticated effective stress analysis.

## REMEDIAL MEASURE FOR THE SITE

According to the results of evaluation, the factor of safety against liquefaction in the upper sand layer under a 4 m of fill at the site concerned is generally lower than 2. In view of the importance of the power plant and in order to avoid any possible damage due to unforeseen nonuniformity of the soil deposit, it was decided that some imporvement work should be carried out to reduce the liquefaction potential of the soil. In addition, the hydraulic fill would also be in a loose condition which will need further densification. There are many methods, such as vibroflotation, compaction sand piles, dynamic consolidation and preloading, which can be used for soil improvement. Each method has its advantages and limitations. After careful consideration of the various factors involved, including the thickness of the sand deposit, construction time and local construction capability and potential cost, compaction sand piles appear to be the most suitable method of treatment for the present site.

The principal criterion of soil improvement at the site is to increase the unit weight of the natural sand deposit and the hydraulic fill layer to a minimum relative density of 65%, which is equivalent to a factor of safety against liquefaction of about 3.5 (Fig. 6). Prior to the full scale imporvement work was commenced, compaction sand piles of different distribution spacings were tried at the site. After evaluating the relative effectiveness of each pattern, a triangular arrangement of 70 cm diameter, 7.5 m long piles with a pile spacing of 1.8 m was selected. In each pile, a volume of 5 cu m of sand was placed and compacted. The effective area of improvement due to the placement of each compaction pile is about 2.8 sq m. In order to evaluate the effectiveness of soil improvement, in situ standard penetration tests, relative density tests, and lateral load tests were carried out. Instrumentations including piezometers and inclinometers were also installed. At the time of preparation of this paper, soil improvement work has been carried out in the area for the first stage construction of the power plant where a 2 m thick layer of fill was placed. Figure 7 compares the N values and relative densities of the soils before and after improvement. Over 90% of the check test data show that the relative densities of the dydraulic fill and the upper layer of the original soil deposit after improvement are higher than 75%. This indicates that satisfactory result of densifying loose sand layers for the purpose of reducing their liquefaction potential can be achieved by means of compaction sand piles.

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TABLE 1 Seismic Information Relating to the Hsin-Ta-Kong Site from 1897 to 1977

| Information | Seismic Zone | Hsin-Ta-Kong Site |
|---|---|---|
| (1) Occurrence frequency of earthquake with M > 7.0 | WSZ | 27 yrs |
| | ESZ | 7 yrs |
| (2) Distance of the nearest earthquake epicenter M > 7.0 to site | WSZ | 60 km |
| | ESZ | 75 km |
| (3) Magnitude M of the nearest earthquake in (2) | WSZ | 7.1 |
| | ESZ | 7.3 |
| (4) Number of minor earthquake occurred within 20 km to site since 1973 | - | over 40 |

TABLE 2 Maximum Ground Acceleration, $a_{max}$, at the site for a Return Period of 50 years

| $a_{max}$ | Reference |
|---|---|
| 0.09 g | HSU (1975) |
| 0.13 g | MAU et al (1978) |
| 0.17 g | TSAI et al (1977) |

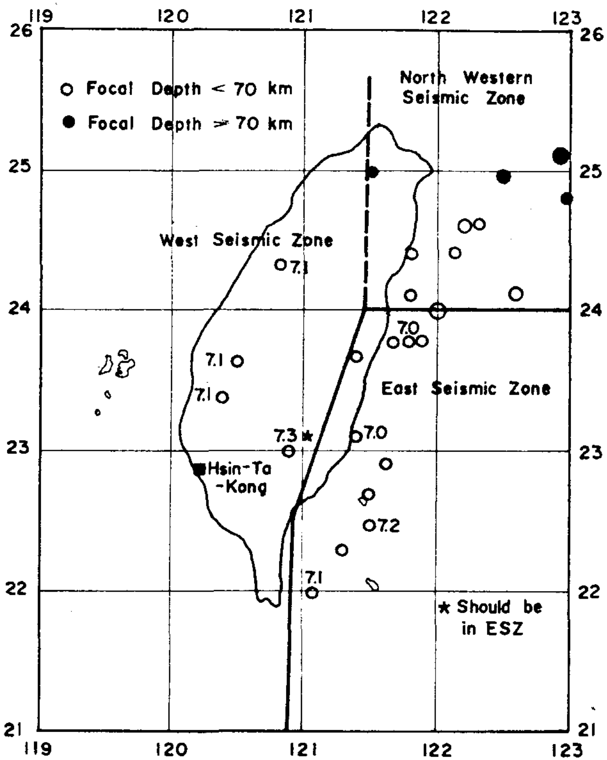


Fig. 1 Epicenter Distribution of Strong Ground Motions Since 1897 and Seismic Zones in Taiwan

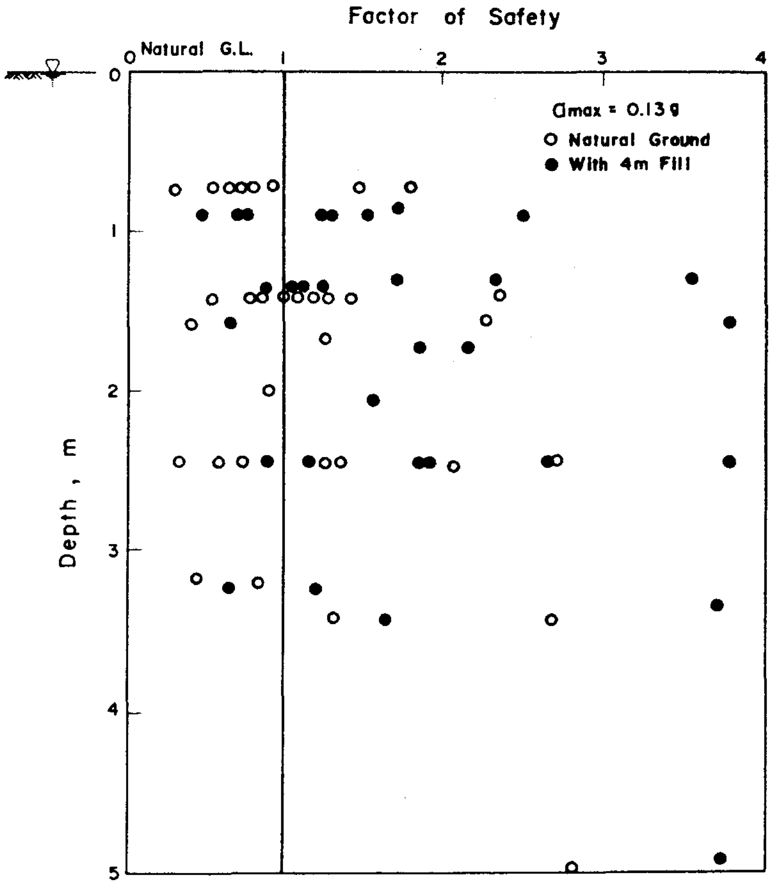


Fig. 2 Computed Factor of Safety against Liquefaction by Using Seed's Empirical Method

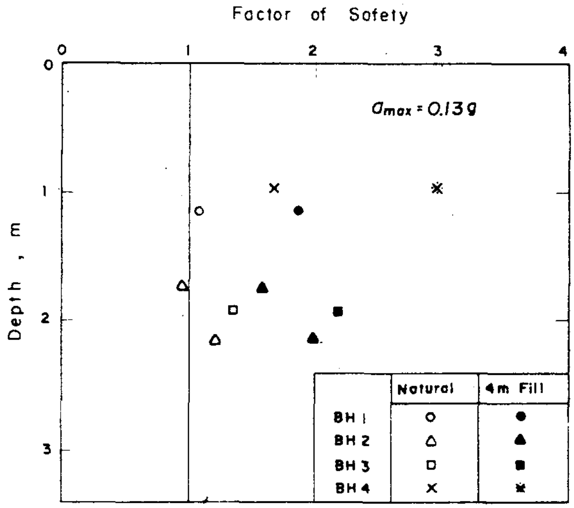


Fig. 3 Factor of Safety against Liquefaction Computed by Analytical Method

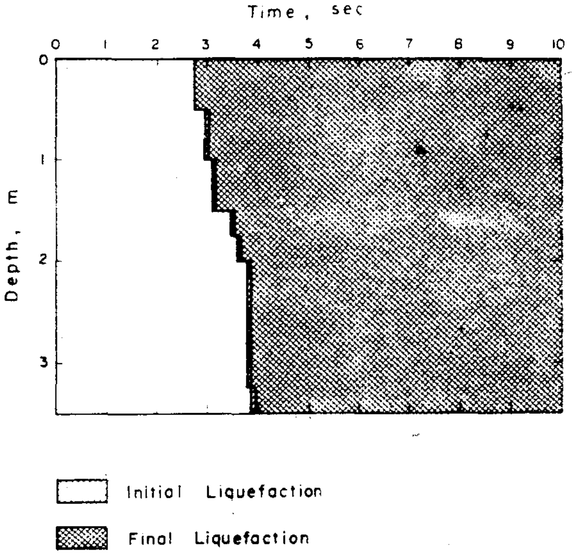


Fig. 4 Development of Liquefaction with Time for Existing Natural Ground

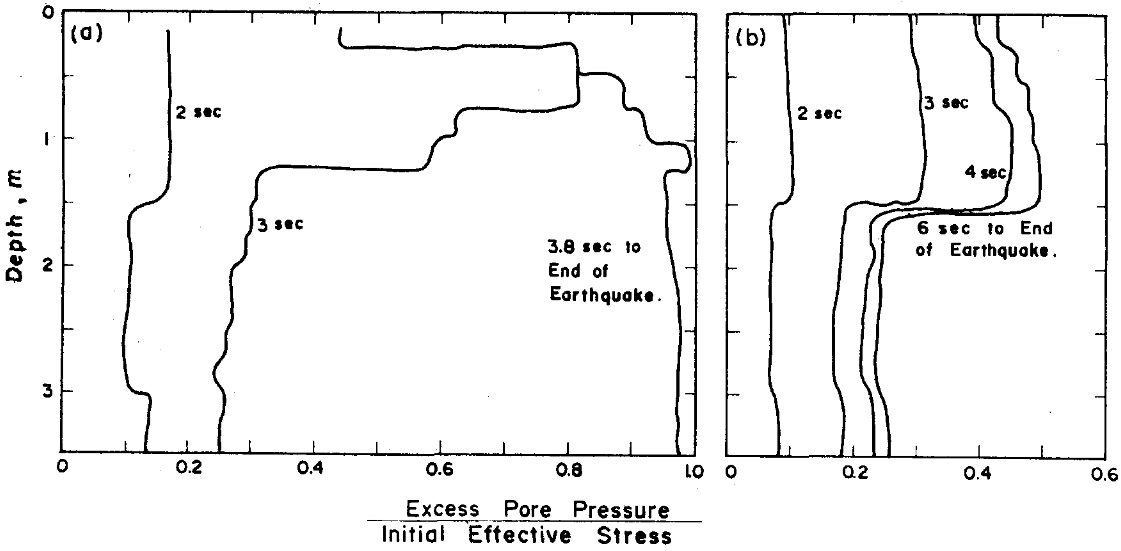


Fig. 5 Pore Pressure Distribution at Various Time Period
(a) for Existing Natural Ground Condition
(b) for Ground with 4 m Fill

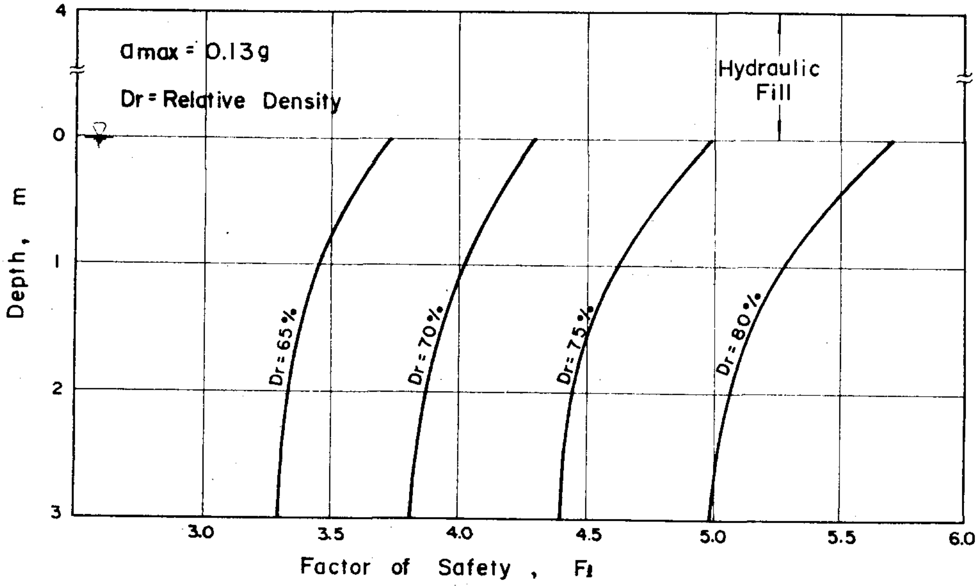


Fig. 6 Factor of Safety Versus Relative Density of the Top Sand Layer

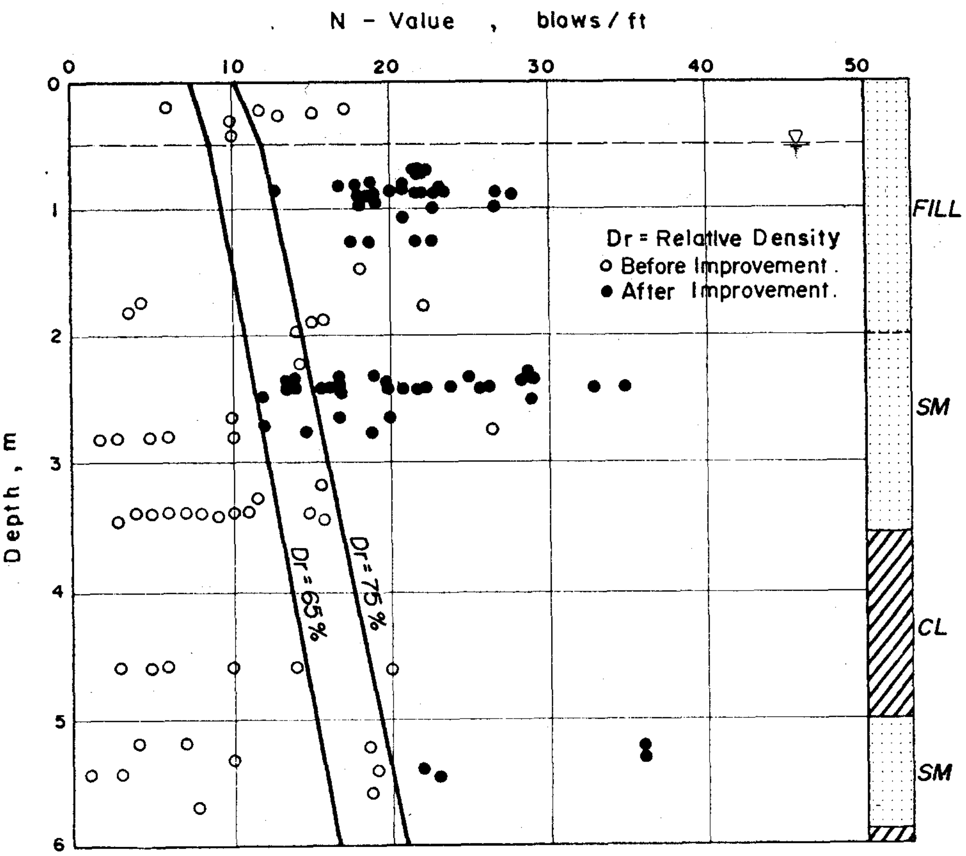


Fig. 7 Variation of N-Values with Depth Before and After Soil Improvement