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PROBABILISTIC ANALYSIS FOR LIQUEFACTION

by

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Chapter 1

INTRODUCTION

Incidents of ground failures by liquefaction have been observed on numerous occasions. The phenomena of liquefaction was dramatically illustrated in Niigata during the 1964 Japanese earthquake.

Throughout the past decade, liquefaction has been the subject of extensive studies (30). Considerable understanding of liquefaction has evolved from laboratory studies. In addition, various laboratory testing procedures have been developed to supply soil parameters needed for analytical techniques that predict liquefaction potential at a site. Data from field observations of liquefaction have been utilized to study the phenomenon as it occurs in the field and to develop preliminary analyses for liquefaction (2, 18, 29, 38).

Among these empirical methods of liquefaction analysis is the one presented by Seed and Idriss (1971). With their presentation, Seed and Idriss included a table entitled "Site Conditions and Earthquake Data for Known Cases of Liquefaction and Non-Liquefaction." This table, with thirty-five case histories from earthquakes ranging between the years 1802 and 1968, has been widely used by engineers to develop procedures for analysis of liquefaction potential.

Earlier studies of case histories were based on acceleration and in the context of the analysis techniques developed at the time of the study. Recent work has focused more on empirical procedures. Different parameters are now being used in the interpretation of field data. Interest in field observations where liquefaction was not observed is increasing. All these factors prompted the investigators of this report to document the original case histories together with newer ones in a detailed format suitable for use by various people who are presently re-examining field performance and developing empirical procedures for liquefaction.

Current analyses for liquefaction are typically limited to the assessment of ground failure potential in terms of a design level earthquake shaking without due regard to the range of possible future earthquake intensities. Furthermore, most analytical procedures currently developed are deterministic and do not provide for the uncertainties involved in the data and in the method of analysis.

Yegian and Whitman (1978) suggested that liquefaction analysis for a site be integrated into an overall risk analysis that begins with a study of the seismicity of the surrounding region and concludes with an estimated probability of foundation failure due to liquefaction of underlying soils. Such a risk analysis will require a probabilistic model describing the strength of soils against liquefaction, given a certain level of earthquake shaking. To develop such a model, the authors of this report evaluated the field data compiled in this report and established a new criterion for liquefaction analysis which employs earthquake magnitude and hypocentral distance, describing the intensity of the seismic event. Uncertainties present in this criterion were quantified and a simple probabilistic model was developed which provides estimates of the probability of liquefaction, conditional to the occurrence of a seismic event.

This report presents the results of the various investigations made in this research. Specifically, the report includes:

a. A new expanded list of case histories of liquefaction;
b. A new criterion for liquefaction analysis;
c. A probability model for the evaluation of the conditional probability of liquefaciton; and
d. Applications of the results presented.

Chapter 2

LIQUEFACTION CASE HISTORIES

Over the past decade, various researchers have developed empirical procedures for liquefaction analysis based on field observations of liquefaction or non-liquefaction during past earthquakes. The data has been drawn primarily from the survey of case histories presented by Seed and Idriss (1971). In the past few years, the need for re-evaluation and expansion of the currently used list to include data obtained from more recent earthquakes has been apparent. The authors of this report completed such an overall review which has provided additional information for many of the case histories as well as new field observations reported in the recent literature. This new survey of case histories includes a total of about 322 data points corresponding to 80 locations and 22 different earthquakes. Each data point corresponds to a specific depth within a soil profile of interest.

It should be noted that, for all cases studied, relevant data recorded in the literature have been taken as originally presented and are documented herein. There has been no attempt to alter or adjust numbers or ranges of data found, but rather, all case histories have been documented in this chapter exactly as they were presented in the given reference, and related soil boring logs, if found, have been made available in Appendix B for the scrutiny of the reader.

2.1 Earthquake Data

Kuribayashi and Tatsuoka (1975) listed over 30 seismic events in Japan during which incidents of liquefaction were observed. Unfortunately, for most of these cases no geotechnical information is available and hence have not been included in this investigation. The earthquakes considered and documented in this report are those for which descriptions of site conditions for associated cases of liquefaction or non-liquefaction could be found. In all, 21 earthquakes have been documented herein. In Appendix A detailed reviews of 8 of the more important seismic events are presented. Table 2-1 presents the pertinent data for the 21 events. Note the difference in the reported magnitude and in the magnitude scale for each seismic event listed in Table 2-1.

2.2 Site Information

Table 2-2 presents the relevant information for sites where liquefaction did or did not occur during the seismic events listed in Table 2-1. The earthquake magnitudes listed for each event are in Richter scale and were selected by the authors based on the reported values listed in Table 2-1. The distances of the sites from the earthquake sources have been reported in the literature in different terms. In Table 2-2 distinctions have been made between distance to energy release (DER), epicentral distance (EP) and hypocentral distancy (HY). The column labelled '% Fines' refers to percent by weight of the soil passing sieve #200. Note that for each site or location, each data point corresponds to a specific depth below the ground surface and that all units are in the SI system.

Table 2-3 summarizes the acceleration and duration for each case history and at each location. The ground acceleration documented in Table 2-3 may not be a good indication of the absolute maximum which occurred in the region. In fact, in some of the cases cited here, the values listed were not available from original references but rather from secondary sources which clearly state that many of these accelerations were not recorded values but were estimated from attenuation laws.

In summary, in this chapter the results of the review and re-evaluation of the case histories of liquefaction were presented. In all, pertinent information from 21 earthquakes, 80 locations and 322 points within the soil profiles investigated were presented. Appendix B includes the soil profiles used in obtaining the information documented in Table 2-2.

Chapter 3

ANALYSIS FOR LIQUEFACTION

3.1 Field Data Interpretation

Employing the case history data presented in Chapter 2, a criterion for liquefaction was developed expressed in terms of earthquake magnitude and hypocentral distance. The interpretation of case histories in terms of magnitude, M, and distance, R, has many advantages over the more commonly used methods which are based on ground acceleration and duration of motion. The use of M and R allowed for greater opportunities in gathering case histories. Sites where liquefaction had occurred but for which no measure of accelerations were available were included in these investigations, and numerous sites where liquefaction did not occur were also considered.

For the evaluation of the criterion for liquefaction, the parameter Liquefaction Potential Index, LPI, proposed by Yegian and Whitman (1976) was employed. LPI, which is inversely related to factor of safety against liquefaction, can be expressed as:

$$\mathrm{LPI} = \frac{\text{stress parameter, } S_c}{\text{strength parameter, } \bar{S}_c} \tag{3.1}$$

If

LPI > 1: liquefaction is likely to occur

LPI < 1: liquefaction is not likely to occur.

In the investigation reported herein, the parameter LPI was employed and an expression for it was developed as follows. The stress parameter was assumed to have the form

$$S_c = e^{c_1 M} (R + 25)^{c_2} \frac{\sigma_v}{\bar{\sigma}_v} \tag{3.2}$$

where M is the earthquake magnitude in Richter scale, R is the hypocentral distance in km, σ_v is the total vertical stress, $\bar{\sigma}_v$ is the effective vertical stress, and c_1 and c_2 are constants. The form for the strength parameter was assumed to be:

$$\bar{S}_c = c_3 N_c^{c_4} \tag{3.3}$$

where N_c is the standard penetration test (SPT) value, corrected for the overburden pressure as suggested by Seed (1976):

$$N_c = N \left(1 - 1.25 \log \bar{\sigma}_v \text{ (tsf)}\right) \tag{3.4}$$

where N is the SPT recorded in the field. Combining Eqs. 3.1, 3.2 and 3.3, LPI can be expressed as:

$$\text{LPI} = \frac{e^{c_1 M} (R + 25)^{c_2}}{c_3 N_c^{c_4}} \cdot \frac{\sigma_v}{\bar{\sigma}_v} \tag{3.5}$$

The values of the constants, c_1, c_2, c_3 and c_4 were evaluated using simulation techniques and all the data points presented in Table 2-2 which had % fines content of less than 10%. A non-linear multiregression analysis following an iterative approach was employed (5). The procedure involved the calculations of difference, DIF, using Eq. 3.6 for each case history and for an assumed set of values for the constants, c.

$$DIF = \ln S_c - \ln \overline{S}_c \qquad (3.6)$$

If this difference (DIF) for a case history was positive and the case was a "no" liquefaction or negative and the case was a "yes" liquefaction, then DIF for that case was squared and saved; otherwise it was discarded. This procedure was repeated for each case and the sum of the squared DIF's, S^2, was computed. The best estimate of the constants, c, were evaluated by minimizing the sum of these squared differences (S^2) for the entire case history list. The minimum value of the sum of the squares (S^2) is a measure of the uncertainty in both the interpretation technique and the data.

A computer program was coded to perform this iterative procedure for the evaluation of the constants, c. The results of these investigations show that the best estimates of the values of the constants are:

$c_1 = 0.2$;

$c_2 = -0.4$;

$c_3 = 0.464$; and

$c_4 \;=\; 0.4.$

Hence, the mean value of $\ln \overline{S}_c$ is

$$\ln \overline{S}_c \;=\; \ln 0.464 + 0.4 \ln N_c \tag{3.7}$$

and the mean value of the $\overline{S}_c$ is

$$\overline{S}_c \;=\; 0.464 \; N_c^{\,0.4} \; e^{0.5\sigma^2_{\ln \overline{S}_c}} \tag{3.8}$$

where $\sigma^2_{\ln \overline{S}_c}$ is the variance of $\ln \overline{S}_c$ as calculated from the regression analysis. The maximum value of the variance computed in this analysis was about 0.036. Thus, the maximum value of $e^{0.5\sigma^2_{\ln \overline{S}_c}}$ will then be about 1.02. For practical reasons, the mean value of the strength parameter can then be estimated from

$$\overline{S}_c \;=\; 0.464 N_c^{\,0.4} \tag{3.9}$$

Hence, the equation for the mean value of LPI can be written:

$$\text{LPI} \;=\; \frac{e^{0.2M} \, (R + 25)^{-0.4}}{0.464 \; N_c^{\,0.4}} \cdot \frac{\sigma_v}{\overline{\sigma}_v} \tag{3.10}$$

Figure 3-1 shows the case history data plotted using the values of the c constants obtained from the simulation procedure and shown in Eq. 3.10. The mean line corresponds to Eq. 3.7. The solid circles indicate liquefaction cases; conversely, the open circles indicate non-liquefaction cases.

The solid circles plotting below the mean line or the open circles plotting above the mean line are referred to as misclassified points and are the data points which contribute to the sum of the squares of the differences (S^2). The number of misclassified points will change from one iteration to the other depending upon the assumed values of the constants, c. For the converged values of the constants c shown in Eq. 3.10, the number of misclassified points, also shown in Fig. 3-1, was 77.

The results presented in Figure 3.1 and Eq. 3.10 are based on standard penetration test values corrected employing Seed's (1976) recommendation. The investigations described above were repeated using blow counts corrected as proposed by Gibbs and Holtz (8)

$$N_c = \frac{50N}{\bar{\sigma}_v(\text{PSI}) + 10} \tag{3.11}$$

The results from this second investigation yielded conclusions similar to what the previous results shown in Eq. 3.10 indicate. To substantiate this, a comparison is made in Fig. 3-2 between the two criteria obtained using Seed, and Gibbs and Holtz equations for N_c. From this figure it can be observed that the results from Gibbs and Holtz (normalized and plotted on the same axis as N_c obtained from Seed) yield approximately the same values of the strength parameter as those obtained from using corrected blow counts as proposed by Seed.

In conclusion, the equations proposed by Seed, and Gibbs and Holtz for correction of the blow count, result in identical criterion for liquefaction. For purposes of consistency, since Seed suggested Eq. 3.4 in his state-of-the-art paper, Eq. 3.10, which employs blow counts corrected according to Seed is recommended herein for the evaluation of liquefaction potential at a site.

3.2 Conditional Probability of Liquefaction

Eq. 3.10 for LPI can be used for a particular site to deterministically evaluate the liquefaction potential: when the computed LPI is greater than 1, liquefaction is "expected" to occur. However, an analysis of liquefaction potential involves many uncertainties. Quantification and incorporation of these uncertainties in the analysis are essential for a realistic assessment of the likelihood of liquefaction. Thus, Eq. 3.10 yields the mean value of LPI using mean or "expected" values of the parameters which define LPI. In addition, it is necessary to compute the coefficient of variation of LPI in order to make predictions of the probability of liquefaction.

The variance of LPI can be computed from (1)

$$\text{Var LPI} = \Sigma\left(\frac{\partial(\text{LPI})}{\partial x_i}\right)^2 \text{Var. } x_i \qquad (3.12)$$

in which x_i's are the variables defining LPI. The coefficient of variation of LPI (V_{LPI}) can then be evaluated from

$$V_{LPI} = \frac{(\text{Var LPI})^{\frac{1}{2}}}{\text{LPI}} \tag{3.13}$$

Assuming that the earthquake parameters, M and R, are specified, the coefficient of variation of LPI can be computed from Eqs. 3.12 and 3.13

$$V_{LPI}^2 = V_{\overline{S}_c|\ln N_c}^2 + 0.16\,\frac{\text{Var.N}}{N^2} + \{1 + (\frac{\sigma_v}{\overline{\sigma}_v})^2\}\frac{\text{Var }\gamma}{\gamma^2}$$

$$+ (\frac{\gamma_w}{\overline{\sigma}_v})^2 \text{ Var dw} \tag{3.14}$$

where $V_{\overline{S}_c|N_c}$ is the coefficient of variation of the strength paramter $\overline{S}_c$ given the corrected blow count N_c. Var.N, Var.γ and Var.dw are the variances of the blow counts, total unit weight and the depth of the water table, respectively, and γ_w is the unit weight of water. The determination of $V_{\overline{S}_c|N_c}$ is based on the results of the multiregression analysis performed for the evaluation of the strength parameter $\overline{S}_c$.

The variance of $\ln\overline{S}_c(\sigma^2_{\ln\overline{S}_c})$ can be determined from (5)

$$\sigma^2_{\ln\overline{S}_c} = \frac{S^2}{n_p-2}\{1 + \frac{1}{n_p} + \frac{(\ln N_{c_i} - \overline{\ln N_c})^2}{\Sigma(\ln N_{c_i} - \overline{\ln N_c})^2}\} \tag{3.15}$$

where: n_p is the number of misclassified points (77 in this investigation); N_c is the corrected blow count and $\overline{\ln N_c}$ is the average value of the $\ln N_i$ where i varies from 1 to 77 and S^2 is the sum of the squares of DIF (Eq. 3.16) obtained from the regression analysis. The coefficient of variation of $\overline{S}_c$ ($V_{\overline{S}_c|N_c}$) can be evaluated from

$$V^2_{\overline{S}_c|N_c} = e^{\sigma^2_{\ln \overline{S}_c}} - 1 \tag{3.16}$$

Table 3-1 represents values of $V_{\overline{S}_c|N_c}$ as a function of N_c determined from the regression analysis. This table illustrates that the coefficient of variation of $\overline{S}_i$ does not significantly change with N_c. In this investigation, a value of 0.035 was assumed for $V^2_{\overline{S}_c|N_c}$. Thus,

$$V^2_{LPI} = 0.035 + 0.16 \frac{Var.N}{N^2} + \{1 + (\frac{\sigma_v}{\overline{\sigma}_v})^2\} \frac{Var.\gamma}{\gamma^2} + (\frac{\gamma_w}{\overline{\sigma}_v})^2 \; Var.dw \tag{3.17}$$

The constant term in Eq. 3.17 is due to the uncertainty in the 'c' parameters which define the equation for LPI (Eq. 3.10). The rest of the terms in Eq. 3.17 describe the uncertainties in the soil parameters used in the liquefaction analysis procedure which is proposed herein. The use of V_{LPI} together with the mean value of LPI computed from Eq. 3.10 can provide an estimate of the conditional probability of liquefaction, defined as

$$P[LIQ.|M \text{ and } R] = P[LPI>1|M \text{ and } R] \tag{3.18}$$

The evaluation of P[LIQ] requires an assumption regarding the probability density function. Yegian and Whitman (1978) suggested the lognormal distribution for LPI. The form of LPI as given in Eq. 3.10 is very similar to that of peak ground acceleration. Donovan (1973) has shown that measured ground acceleration is lognormally distributed. Thus, LPI is also assumed to be lognormally distributed. The conditional probability of liquefaction is then determined by computing the standardized variable U:

$$U = \frac{m_{\ln LPI}}{\sigma_{\ln LPI}} \tag{3.19}$$

where

$$m_{\ln LPI} = \ln LPI - 0.5\,\sigma^2_{\ln LPI} \tag{3.20}$$

and

$$\sigma^2_{\ln LPI} = \ln(V^2_{LPI} + 1) \tag{3.21}$$

Using the computed U and the normal tables, the conditional probability P[LIQ.|M and R] can be determined.

3.3 Evaluation of Seed's (1976) Criteria for Liquefaction

In his state-of-the-art paper, Seed (1976) proposed a criterion for liquefaction which employs ground acceleration to describe the earthquake-induced shear stress. An evaluation of this criterion was made in light of the expanded data set presented in Table 2-2. Figure 3-3 shows the data plotted in the same manner as was done by Seed.

The solid line in Fig. 3-3 defines the criterion as was originally proposed by Seed (1976). Considering the new data set Seed's criterion is still valid except for two "liquefaction" data points which plot only slightly below the solid line. These two points correspond to the Niigata and Alaska case histories, which have all the rest of their liquefaction data points plotting above the solid line.

3.4 Comparison of Proposed Criterion with the Criterion Presented by Kuribayashi and Tatsuoka

Kuribayashi and Tatsuoka (1975), in their review of liquefaction during Japanese earthquakes, chose to study case histories (where liquefaction was observed) in terms of earthquake magnitude and epicentral distance. Based on 32 observations, they plotted earthquake magnitude versus the maximum epicentral distance at which liquefaction was observed. Figure 3-4 presents the criterion they proposed to relate earthquake magnitude to maximum epicentral distance beyond which liquefaction is unlikely to occur. In the following section, the validity of this criterion established without considering the site conditions for the case histories used will be investigated employing the results of the research reported herein.

It is important to recognize that, at a particular site, the looser the sand is, the larger will be the maximum distance at which the sand might liquefy during a particular seismic event. In theory, if one assumes that the looser the sand, the smaller its resistance against liquefaction and that there is no minimum strength which medium to fine sands have, there should be no limit to the distance at which liquefaction can occur in extremely loose sands.

The observation that the 32 data points from Japanese earthquakes define a limiting distance for liquefaction lead to the conclusion that either

a. There is a minimum strength against liquefaction which sands have (regardless of density) which limits the distance at which liquefaction can occur during earthquakes; and/or
b. During the seismic events considered there were no sites beyond the maximum distances reported which had very loose sands or looser than within the limiting bound observed.

To evaluate these two possibilities, the results of the present research were utilized. Referring to Fig. 3-1, it is observed that all liquefaction data points (solid circles) plot at a strength value greater than 0.8 regardless of the value of N_c. This may suggest that all sands have a minimum strength against liquefaction, or, possible the data set used is insufficient and is deficient in case histories of liquefaction with very small blow counts. Assuming that the minimum strength against liquefaction is 0.8, and using the LPI equation, a plot of earthquake magnitude and maximum hypocentral distance was generated and is shown in Fig. 3-4, together with the criterion proposed by Kuribayashi and Tatsuoka. From this figure it is concluded that the criterion based on the 32 case histories from Japan does not necessarily correspond to the minimum strength of sands against liquefaction.

The alternate explanation to the existence of a limiting distance, as suggested by the Japanese data, is that the maximum distance to liquefaction reported during each seismic event corresponded to a site condition which may not necessarily be the loosest possible. For example, during a small (M = 5) seismic event, the zone of influence around the epicenter will be smaller than during a much larger event. Thus, it is more likely that during large events, looser soils will be encountered then during smaller events. Hence, the segment of Kuribayashi and Tatsuoka's criterion at large magnitudes may correspond to looser soils than those corresponding to the small magnitude portion of the plot.

Figure 3-5 shows a comparison between the magnitude-distance plots obtained from this research with the Japanese criterion. From this figure it is concluded that, based on world-wide data and depending upon the SPT, liquefaction is likely to occur beyond the maximum distance given by the Japanese criterion. It appears that during the seismic events causing magnitudes 6 to 7, loose (N_c < 15 blows/ft) sand deposits were not present in the region of the earthquake beyond 30-50 km from the epicenters. In fact, Niigata sands have corrected blow counts of about 20 blows/ft. During the larger events having larger zones of influence, looser soils (N_c = 10-15) were encountered as indicated from the comparison made in Fig. 3-5.

In conclusion, the results of this research indicate that whereas the criterion proposed by Kuribayashi and Tatsuoka may be valid for certain regions in Japan, it should not be indiscriminately used in engineering practice. This criterion is not valid for sands with the N_c less than 15. Kubo et al. (1977) presented a case history from the 1977 Rumanian earthquake which plotted to the right of the Japanese criterion, confirming the conclusion made herein.

Chapter 4

APPLICATIONS

In this chapter, a number of applications of the proposed method of liquefaction analysis will be presented.

4.1 Preliminary Liquefaction Analysis

Eq. 3.10 for LPI can be used to deterministically evaluate the liquefaction potential at a particular site. To estimate the conditional probability of liquefaction P[LIQ. |M and R], the coefficient of variation of LPI (Eq. 3.14) is used together with the Normal tables for probability density function. Fig. 4-1 shows results of probability calculations made assuming the typical range 0.2 to 0.5 for the coefficient of variation of LPI, V_{LPI}. This plot can be used in engineering practice in preliminary studies of liquefaction for selected design seismic events. Note that the curve corresponding to $V_{LPI} = 0.0$ describes a deterministic analysis of liquefaction.

The conditional probabilities shown in Fig. 4-1 can be used to compute the overall probability of ground failure by liquefaction, P[LIQ.], which is given by:

$$P[LIQ.] = \sum_{M,R} P[LIQ.|M \text{ and } R] \cdot P[M,R] \tag{4.1}$$

where P[LIQ.|M and R] is the probability of liquefaction conditioned upon the occurrence of a seismic event of magnitude M and distance R and P[M,R] is the probability of that event occurring. The overall probability of liquefaction is obtained by summing the contributions of all possible earthquakes (M and R) to this probability.

4.2 Pore Pressure Prediction

Procedures currently used for the estimation of pore pressures in sands are based on the earthquake-induced shear stresses obtained from the application of the one-dimensional shear wave propagation theory, and on the laboratory pore pressure data on cyclically-loaded specimens of sands. Such procedures for pore pressure prediction involve many uncertainties, and are complicated and expensive to apply. Yegian (1980) proposed an empirically developed model for the prediction of pore pressures in loose, saturated sands. The model employs LPI to define a threshold event causing 100% pore pressure response with normalized laboratory cyclic behavior curves, in order to predict the excess pore pressure generated during events smaller than the event causing liquefaction. The pore pressure response, r_u, is defined as:

$$r_u = \frac{\Delta u}{\bar{\sigma}_v} \tag{4.2}$$

in which Δu is the excess pore water pressure for level ground conditions. Thus, a pore pressure response of 100% (r_u = 1) indicates that the sand under study has liquefied.

In engineering practice, common analysis of liquefaction involves the determination of whether or not the pore pressure response is greater than 1 for a given seismic event. Thus, a computed LPI less than 1 may indicate that the sand is not likely to liquefy, but does not indicate the level of excess pore pressure below levels that might still be generated during that particular event. Such increases in excess pore pressure below levels causing liquefaction may be of such magnitude as to reduce the effective stresses in the soil to levels consequential to the dynamic response of the deposit, and to the settlement of a structure founded on the deposit.

The liquefaction analysis procedure described in this report can be used to develop such a model for excess pore pressure prediction as a function of earthquake magnitude and distance. Yegian (1980) showed that the pore pressure response parameter, r_u, can be related to LPI as:

$$r_u = \frac{2}{\pi} \text{ arc sin } (\text{LPI})^{\frac{1}{2\alpha\beta}}, \text{ LPI} < 1.0 \qquad (4.3)$$

where α is a curve-fitting parameter used to relate laboratory pore pressure data for a particular soil to the ratio of the number of equivalent cycles of stress application to the number of cycles causing the parameter, r_u, to be equal to 1. The parameter β is the slope of the laboratory strength data when plotted on log-log paper. Definition of possible ranges for these parameters was attempted on the basis of published laboratory strength and pore pressure data. Values of β were estimated from laboratory strength curves suggested by various investigators for different types of tests and sands. Based on this review, values of β ranged between 0.10 and 0.25, with an average value of 0.19. A similar study of published data on excess pore pressure plotted against the normalized number of cycles yielded a range of values for α between 0.5 and 1.0. Seed and Booker (1977) recommended a typical value of 0.7.

Using the ranges for α and β given above, together with the equation for LPI (Eq. 3.10), plots of pore pressure response versus earthquake magnitude, distance and soil strength are generated as shown in Fig. 4-2. This plot can be used in preliminary studies to determine expected buildup of excess pore pressure in a particular sand deposit during a given seismic event. Fig. 4-2 demonstrates that while an LPI of 0.8 (factor of safety of 1.25) may imply safety against liquefaction, there may be a pore pressure response of up to 50%.

Thus, the model described enables quick evaluation of pore pressures for preliminary studies and provides an opportunity to combine future field data and laboratory data on pore pressures in a simple, logical and consistent manner.

4.3 Liquefaction Risk Analysis

Probabilistic seismic hazard analysis has received increased attention in the past decade. Computer programs are now available to compute annual probability of a certain seismic parameter exceeding a specified value. The input information to such an analysis includes the seismic source data and an attenuation law relating the seismic parameter of interest to earthquake magnitude and distance. The details of such an analysis are beyond the scope of this report and are reviewed by Yegian (1979). These computer programs can also be used to evaluate an overall annual probability of liquefaction at a site. The attenuation law specified in seismic hazard analysis is usually of the form:

$$A = k_1 e^{k_2 M} (R + 25)^{k_3} \tag{4.4}$$

where k_1, k_2 and k_3 are constants.

The output of the analysis is the annual probability of 'A' exceeding a certain value 'a'. In a similar way, liquefaction risk analysis can be performed since the equation for LPI has the same form as the attentuation law used in seismic hazard analysis. For a given sand deposit, LPI will be given by

$$LPI = k_1 e^{k_2 M} (R + 25)^{k_3} \tag{4.5}$$

where

$$k_1 = \frac{1}{0.464 \quad N_c^{0.4}} \cdot \frac{\sigma_v}{\bar{\sigma}_v}$$

$k_2 = 0.2$ and

$k_3 = -0.4.$

Using these parameters to define the attenuation law in the computer program, one can compute the annual probability of LPI exceeding a certain value. If the value to be exceeded is assigned as 1.0 in the analysis, the output is the annual probability of liquefaction.

Liquefaction risk analysis can provide information enabling comparisons between the various factors contributing to the overall seismic risk to a constructed facility. The analysis can also be used to study the degrees of influence of the various parameters involved to identify the major factors contributing to the likelihood of liquefaction at a site.

Chapter 5

SUMMARY

A comprehensive reivew of all liquefaction case histories was made and an expanded data set was prepared. This report presented the newly compiled field data together with a criterion for liquefaction which is based on the field observations of liquefaction and non-liquefaction. A simple probability model was presented to calculate the probability of liquefaction conditional to the occurrence of a particular seismic event.

Applications of the proposed criterion and the probability model were discussed and comparisons were made between the results of this research and those previously published by other investigators.

It is concluded that liquefaction analysis procedures, which are to some extent invariably based on field observations, include significant uncertainties because of the uncertain and erratic nature of the data used. Hence, caution is made herein when such procedures are used in a deterministic manner using average values of the parameters involved. To make reliable predictions of liquefaction potential, proper evaluations of these uncertainties should be made. The probability model presented in this report enables such an evaluation to be made easily. The importance and significance of these uncertainties to the overall safety considerations for a constructed facility can be determined by incorporating the liquefaction study into an overall risk study for the facility.

Table 2-1 Earthquake Data

EARTHQUAKE	DATE	LOCATION	FOCAL DEPTH (KM)	LAT. & LONG. OF EPICENTER	MAGNITUDE	REFERENCE
E-1	1802 12/9	Sado Island Japan		37.8N, 138.4E	6.6 K	12, 27
E-2	1887 7/22	Koshigun Japan		37.7N, 139.0E	6.1K	12, 18, 27
E-3	1891 10/28	Mino-Owari Japan		35.6N, 136.6E	8.4 K	14, 18, 27 37
E-4	1906 4/18	San Francisco California	25	38.1N, 122.8W 38.0N, 123.0W	8.3 R 8.2 U 8.2 R 8.3 U 8.25 R	19, 24, 28, 48, 49
E-5	1925 6/29	Santa Barbara California		34.3N, 119.8W	6.3 R 6.2 ML 6.25 R	19, 24, 29
E-6	1933 3/11 3/10	Los Angeles Long Beach California	16	33.6N, 118.0W	6.3 U 6.3 R 6.25 R	19, 24, 41
E-7	1944 12/7	Tohnankai Japan	25	33.7N, 136.2E 33.8N, 136.0E	8.3 R 8.3 U 8.0 JMA 8.0 R	14, 18, 19, 24, 37
E-8	1948 6/28	Fukui Japan	20	36.1N, 136.2E 36.5N, 136.0E	7.3 U 7.3 JMA 7.2 R 7.2 U 7.3 R	14, 18, 19, 24, 28, 29
E-9	1955	San Francisco Concord Bay Area California			5.4 R	41.
E-10	1957 3/22	San Francisco Daly City California		37.7N, 122.5W	5.3 R 5.3 ML 5.5 R	19, 29, 37, 41, 49
E-11	1960 5/22	Chile		39.5S, 74.5W	8.4 U 8.4 R 8.5 U 8.75 U	19, 29, 37
E-12	1964 6/16	Niigata Japan	30 40	38.4N, 139.2E	7.5 U 7.5 R 7.5 JMA 7.3 U 7.7 U	10, 11, 18, 28, 29, 37, 45

Table 2-1 (cont'd)

E-13	1964 3/27 3/28	Alaska	20 33	61.0N, 147.7W	8.3 U 8.3 MS 8.4 R 8.5 U 8.4 - 8.6 U 8.5 U	19, 29, 36, 37, 49.
E-14	1965 9/10	San Francisco Concord Bay Area California		38.0N, 121.8W	4.9 MB 4.9 ML	19
E-15	1963 2/21 2/22 3/25	Ebino Japan	 0 0 10	38.0N, 130.7E	5.7, 6.1 K 5.6 U 5.7, 5.4 U 6.3 R	18, 39.
E-16	1968 5/16	Tokachi-Oki Japan	20	40.7N, 143.6E	7.9 U 7.8 R	18, 20, 29.
E-17	1968 7/1	Saitama Japan	50	36.0N, 139.4E	6.1 JMA	33
E-18	1970 3/28	Gediz Turkey		37.1N, 30.5E	7.1 R	41
E-19	1971 2/9	San Fernando California	13 8	34.4N, 118.4W	6.6 ML 6.6 R 6.4 U 6.2 MB 6.5 U 6.5 MS 6.4 R 6.4 ML	19, 32, 36, 41, 49.
E-20	1972	Yokohama			7.3 JMA	33
E-21	1978 6/12	Miyagiken-Oki Japan	40 25	38.2N, 142.2E 38.2N, 142.1E	7.4 JMA 7.4 U	16

JMA = Japan Meteorological Agency magnitude scale
K = Kawasumi magnitude scale
MB = Body Wave magnitude
ML = Local magnitude
MS = Surface Wave magnitude
R = Richter magnitude
U = unspecified

Table 2-2 List of Case Histories

EARTH-QUAKE (E)	SITE LOCATION	RICHTER MAGNI-TUDE	DISTANCE (KM)	WATER TABLE (Meter)	DEPTH (Meter)	STANDARD PENETRA-TION TEST VALUE,N	% FINES	SOIL LIQUE-FACTION	REFER-ENCES
E-1	Niigata Japan	6.6	39/DER		(Same as in E-12)			No	12,29
E-2	Niigata Japan	6.1	47/DER		(Same as in E-12)			No	12,29 29,37
E-3	Ogaki City Japan	8.4	30/EP	0.8	5.0	4	8	Yes	14,29 29,37
					10.0	20	5	Yes	
					11.0	16	3	Yes	
					12.0	17	4	Yes	
					13.0	17	5	Yes	
					14.0	17	10	Yes	
					15.0	16	7	Yes	
					16.0	14	7	Yes	
					17.0	12	7	Yes	
	Ogase Pond Japan		30/EP	2.4	4.0	11	10	Yes	
					5.0	12	5	Yes	
					6.0	15	4	Yes	
					7.0	19	3	Yes	
					10.0	20		Yes	
					11.0	17	4	Yes	
					12.0	45	1	Yes	
	Unuma Town Japan		30/EP	1.9	3.0	15	4	No	
					4.0	16	3	No	
					5.0	19	2	No	
					6.0	17	3	No	
					7.0	19	6	No	
					8.0	20	3	No	
					9.0	16	5	No	
	Ginan W. P.S. Japan		30/EP	2.0	2.0	5	6	Yes	
					3.0	3	8	Yes	
					4.0	10	5	Yes	
					5.0	8	3	Yes	
					6.0	8	5	Yes	
					7.0	11	5	Yes	
					8.0	8	7	Yes	
					9.0	10	5	Yes	
					10.0	11	2	Yes	
					11.0	12	1	Yes	
					12.0	16	1	Yes	
					13.0	23	1	Yes	
					14.0	23	1	Yes	
					15.0	19	1	Yes	
					16.0	19	2	Yes	
E-4	Foot of Market Zone, San Francisco California	8.3	15/HY	2.4	4.6	16		Yes	41
					7.6	16		Yes	
	South of Market California		13/HY	1.5	4.6	24		Yes	
E-5	Sheffield Dam, Santa Barbara California	6.3	11/DER	4.6	7.6	14[1]		Yes	29
E-6	Western LNG. Terminal Los Angeles California	6.3	16/HY	3.1	6.1	13		No	41
					7.9	10		No	
					11.0	20		No	
				5.5	7.3	20		No	
					11.0	8		No	
					13.4	13		No	
					16.5	21		No	

Table 2-2 (cont'd)

				3.1	6.1	14		No	
					7.3	7		No	
					9.5	15		No	
					6.4	13		No	
E-7	Komei Town Japan	8.3	165/EP	2.0	3.5	3	10	Yes	14
					4.0	7	6	Yes	
					5.0	7	9	Yes	
					6.0	10	11	Yes	
					8.0	26	7	No	
					9.0	37	8	No	
					10.0	38	6	No	
					11.0	43	8	No	
					12.0	40	6	No	
					13.0	39	9	No	
					14.0	45	10	No	
	Meiko Street Japan		165/EP	0.5	3.5	2	12	Yes	14
					10.0	32	10	No	
					11.0	49	5	No	
					13.0	44	5	No	
					14.0	28	8	No	
					15.0	37	3	No	
	Ienaga Shinden Japan		165/EP	2.5	14.0	24	10	No	14
					15.0	25	10	No	
					16.0	36	11	No	
E-8	Takaya Town Japan	7.3	5/EP	0.8	6.0	26	5	No	14
					7.0	27	4	No	
					8.0	29	3	No	
					9.0	29	2	No	
					10.0	31	5	No	
	Takaya Town Japan		5/EP	3.7	4.2	12	4	Yes	14
					5.2	14	3	Yes	
					6.2	19	5	Yes	
					7.2	19	2	Yes	
					8.2	17	2	Yes	
					9.0	22	6	Yes	
					10.1	15	6	Yes	
					11.1	17	4	Yes	
					19.0	31	7	No	
	Shonai Temple Japan		5/EP	1.2	1.5	5		Yes	14
					2.5	3		Yes	
					3.1	3		Yes	
					4.0	7		Yes	
					4.5	18		No	
					5.5	20	0	No	
					10.0	5		Yes	
					11.0	7		Yes	
					12.0	35		No	
					14.0	38		No	
					18.0	32		No	
	Agr. Union Japan		5/EP	0.9	6.1	6	0	Yes	14
					7.0	9	0	Yes	
					7.5	19	0	No	
					8.5	28	0	No	
					9.0	24	15	No	
					10.5	10	5	Yes	
					16.0	25	0	Yes	
					16.5	40	0	No	
					17.0	44	0	No	
E-9	Joaquin Aq. San Francisco California	5.4	40/HY	2.4	17.3	22		No	41

Table 2-2 (cont'd)

E-10	San Francisco California	5.3							
	St. Francis Cir.		11/HY	4.6	6.1	4		No	41
	Lake Merced		7/DER	2.4	3.1	7		Yes	29
	Duboce Ave. & Sanchez St.		10/HY	3.7	4.0	14		No	41
	Foot of Market Zone		16/HY	2.4	4.6	16		No	41
					7.6	16		No	
				4.9	6.1	52		No	
	South of Market Zone		13/HY	1.5	4.6	24		No	41
	Mission Creek		11/HY	1.5	6.1	6		No	41
	Polk & Golden Gate Ave.		16/HY	4.6	6.1	20		No	41
	Polk & Market St.		16/HY	2.4	4.6	20		No	41
	Welden near Barneveloe St.		11/HY	0.9	1.2	4		No	41
					1.2	6		No	
				1.2	3.7	6		No	
					4.3	6		No	
	Mission St. & Spear St.		16/HY	3.1	3.7	11		No	41
					4.0	10			
	Alameda California								
	Park St. & Otis Dr.		24/HY	1.8	5.8	12		No	41
				1.2	5.8	16		No	
	Singleton Ave.		24/HY	1.8	3.7	10		No	41
	Treasure Island		20/HY	2.4	7.6	3		No	41
					9.1	5		No	
				1.8	5.8	7		No	
					5.8	5		No	
					3.1	9		No	
					4.6	5		No	
					4.6	8		No	
					4.6	5		No	
					4.6	15		No	
	W. 5th St. & Ave.D		22/HY	1.8	3.1	3		No	41
				2.1	2.7	7		No	
	Westline Ave.		22/HY	0.6	1.5	13		No	41
	Emeryville		26/HY	1.2	4.3	7		No	41
	Westline M.C.		20/HY	1.2	4.6	5		No	41
					3.7	12		No	
E-11	Puerto Montt Chile	8.4	112/DER	3.7	4.6	6		Yes	29
					4.6	8		Yes	
					6.1	18		No	
E-12	Nippon Fire & Marine Ins. Niigata Japan	7.5	51/EP	0.9	4.0	4		Yes	11
					5.0	5		Yes	
					6.0	6		Yes	
					7.0	8		Yes	
					8.0	15		No	
					9.0	10	0-10	Yes	
					10.0	7		Yes	
					11.0	12		No	
					12.0	21		No	
					13.0	32		No	
	Tribune P.S. Japan		51/EP	0.9	5.0	2		Yes	11
					6.0	2		Yes	
					7.0	3		Yes	
					8.0	1		Yes	
					9.0	2	0-10	Yes	
					10.0	3		Yes	
					11.0	30		No	
					14.0	17		No	
					15.0	2		Yes	
					16.0	20		No	

Table 2-2 (cont'd)

	Benten-cho Japan		51/EP	0.9	4.8	10		Yes	47
					4.8	10		No	
					5.4	8		Yes	
					6.4	10		Yes	
					7.4	6	0-10	Yes	
					8.4	12		Yes	
					9.4	11		Yes	
					10.4	6		Yes	
					11.0	15		Yes	
					11.0	15		No	
	Benten-cho Japan		51/EP	0.9	3.6	6		Yes	47
					3.6	6		No	
					4.5	8		Yes	
					5.5	10		Yes	
					6.5	7		Yes	
					7.5	8		No	
					8.5	10	0-10	Yes	
					9.5	12		Yes	
					11.5	6		Yes	
					12.0	20		Yes	
					12.0	20		No	
					14.4	32		No	
					15.4	30		No	
					17.0	34		No	
	From Ohsaki (1966) Fig. 34		51/EP	0.9	1.4	3		Yes	22
					2.4	2		Yes	
					4.4	5		Yes	
					5.4	2		Yes	
					6.0	4		Yes	
					6.0	4		No	
					6.4	7		No	
					7.4	9	0-10	No	
					10.4	15		No	
					11.4	25		No	
					12.4	32		No	
					13.4	17		No	
					14.4	19		No	
					15.0	19		No	
					15.4	18		Yes	
					16.4	26		Yes	
E-13	Snow River-B605A Alaska	8.4	142/EP	0.0	3.1	5	10	Yes	25
					7.6	5	10	Yes	
	Snow River-B605 Alaska		142/EP	2.4	8.5	8	10	Yes	25
				0.0	3.1	5	10	Yes	
					7.6	5	10	Yes	
					12.2	5	10	Yes	
	Quartz Creek B676 Alaska		145/EP	3.1	13.7	42		No	25
					16.8	86		No	
	Scott G.-B348 Alaska		126/EP	0.0	3.7	9	10	Yes	25
					15.2	11	10	Yes	
	Valdez Dock Alaska		72/HY	2.1	3.5	9		Yes	3
					6.4	11		Yes	
				1.5	5.0	15		Yes	
					6.6	10		Yes	
				1.4	1.7	7		Yes	
					4.7	15		Yes	
					6.4	10		Yes	
E-14	Joaquin Aq. Concord California	4.9	24/HY	2.4	17.1	22		No	41
E-15	Ebino Town Japan	6.3	8/EP	1.8	3.0	7	10-20	Yes	39
					4.0	7	10-20	Yes	
	Yoshimatsu Town Japan		8/EP	1.4	3.0	7	20	Yes	39
					4.0	7	20	Yes	
					5.0	7	20	Yes	
					6.0	7	20	Yes	

Table 2-2 (cont'd)

E-16	Hachinoe	7.8	160/EP	1.0	1.5	12		No	23
	Japan				2.5	12		No	
	(P1)				3.5	16		No	
					4.5	30		No	
					5.5	40	5-10	No	
					6.5	57		No	
					7.5	50		No	
					8.5	40		No	
	(P2)		160/EP	1.5	2.5	15		No	23
					3.5	33		No	
					4.5	27		No	
					5.5	27		No	
					6.5	24	5-10	No	
					7.5	29		No	
					8.5	26		No	
					10.0	32		No	
	(P4)		160/EP	1.3	1.5	10		No	23
					2.5	12		No	
					3.5	16		No	
					4.5	25		No	
					5.5	35	5-10	No	
					6.5	35		No	
					7.5	15		No	
					8.5	24		No	
					9.5	17		No	
	(P5)		160/EP	1.5	2.5	12		No	23
					3.5	37		No	
					4.5	35		No	
					5.5	35	5-10	No	
					6.5	32		No	
					7.5	37		No	
	(P6)		160/EP	1.5	2.0	1		Yes	23
					3.0	2		Yes	
					4.0	2		Yes	
					5.0	3	5-10	Yes	
					6.0	4		Yes	
					7.0	22		No	
					8.0	32		No	
					9.0	27		No	
	Hachinoe Plant		160/EP	1.0	2.0	2	10	Yes	20
	Japan				3.0	2	10	Yes	
					4.0	2	10	Yes	
	Hachinoe Acc.		160/EP	1.3	2.5	5		No	20
	Japan				3.5	27		No	
					4.5	26		No	
					5.5	24	5-10	No	
					6.5	23		No	
					7.5	26		No	
					8.5	22		No	
	Nanaehama Beach		160/DER	1.0	2.0	6	10-20	Yes	15
	Hakodate				3.0	4	10-20	Yes	
	Japan				5.0	6	10-20	Yes	
					6.0	7	20	Yes	
					7.0	10	20	Yes	
E-17	Saitama	6.1	69/HY	8.0	10.0	14		No	33
	Japan				16.0	20		No	
	101-2				23.0	32		No	
					28.0	42		No	
					32.0	24		No	

Table 2-2 (cont'd)

	105-2		69/HY	8.0	10.0	47		No	33
	119		69/HY	2.0	6.3	10		No	33
	121		69/HY	2.0	4.5 6.0 33.0	6 4 42		No No No	33
	130		69/HY	3.5	6.5 7.5 11.5	5 20 25		No No No	33
	602		79/HY	3.0	3.8 5.0	5 6		No No	33
E-18	Bursa, Turkey	7.1	130/HY	3.7	7.0	12		No	41
E-19	Jensen F. Plant San Fernando California	6.4	24/HY	18.3 14.3 1.5 16.8	19.8 25.9 18.3 4.6 19.8	15 32 42 8 21		No No No Yes Yes	41
E-20	Yokohama Japan	7.3	280/HY	3.1	6.1 12.2	4 6		No No	26
E-21	Arahama Japan	7.4	119/EP	2.0	4.0 5.0 6.0 7.0 8.0 9.0 10.0	9 10 10 9 10 9 8	0-5	Yes Yes Yes Yes Yes Yes Yes	47
	Yuriage Japan		115/EP	2.0	4.0 6.0 8.0 10.0 12.0 14.0	22 25 26 23 28 36	0-5	No No No No No No	47

[1]calculated from relative density of 40%
DER = Distance to Energy Release
EP = Epicentral Distance
HY = Hypocentral Distance

Table 2-3 Acceleration and Duration Data

EARTHQUAKE (E)	DATE	SITE LOCATION	RICHTER MAGNITUDE	ACCEL-ERATION (%g)	DURATION (sec)	REFERENCES
E-1	1802 12/9	Niigata Japan	6.6	0.12	20	29
E-2	1887	Niigata Japan	6.1	0.08 0.12	12 12	29, 37
E-3	1891 10/28	Ogaki City Ogase Pond Unuma Town Ginan W. P.S. Japan	8.4	0.35 0.35 0.35 0.35	75 75 75 75	28, 37
E-5	1925	Sheffield Dam Santa Barbara California	6.3	0.20	15	29
E-7	1944 12/7	Komei Town Meiko Street Japan	8.3	0.08 0.08	70 70	29, 27
E-8	1948 6/28	Takaya Town Shonai Temple Agr. Union Japan	7.3	0.30 0.30 0.30	30 30 30	29, 37
E-10	1957 3/22	Lake Merced California	5.3	0.18	18	29, 37
E-11	1960 5/22	Puerto Montt Chile	8.4	0.15	75	29, 37
E-12	1964 6/16	Niigata Japan	7.5	0.16	40	29, 37
E-13	1964 3/27 3/28	Snow River Quartz Creek Scott Glacier Valdez Alaska	8.4	0.15 0.12 0.16 0.25	180 180 180 180	29, 37
E-16	1968 5/16	Hachinohe Hokadate Japan	7.8	0.21 0.18	45 45	29
E-19	1971 2/9	Jensen Plant San Fernando California	6.4	0.40 0.35	NR 15	35, 37
E-21	1978 6/2	Arahama Yuriage Japan	7.4	0.24 0.29	NR NR	47

Table 3.1 Coefficient of Variation of the Strength Parameter $\overline{S}_c$, $V_{\overline{S}_c | N_c}$

N_c Corrected Blow Count	$V^2_{\overline{S}_c \| N_c}$
5	0.0346
10	0.0333
20	0.0335
30	0.0342
40	0.0350
50	0.0358
60	0.0365

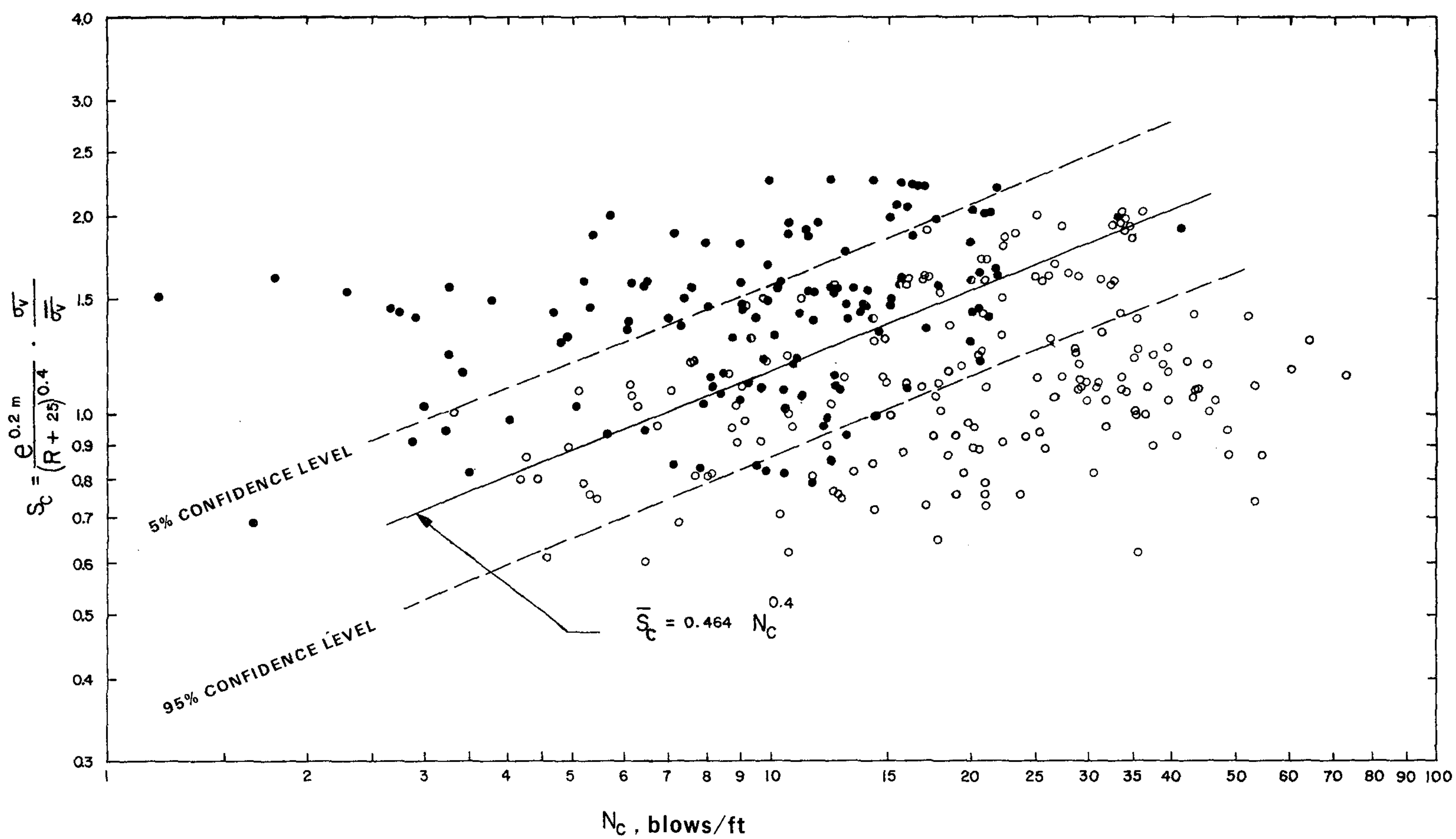


Fig. 3-1 Strength Parameter S_c, versus Corrected Blow Counts.

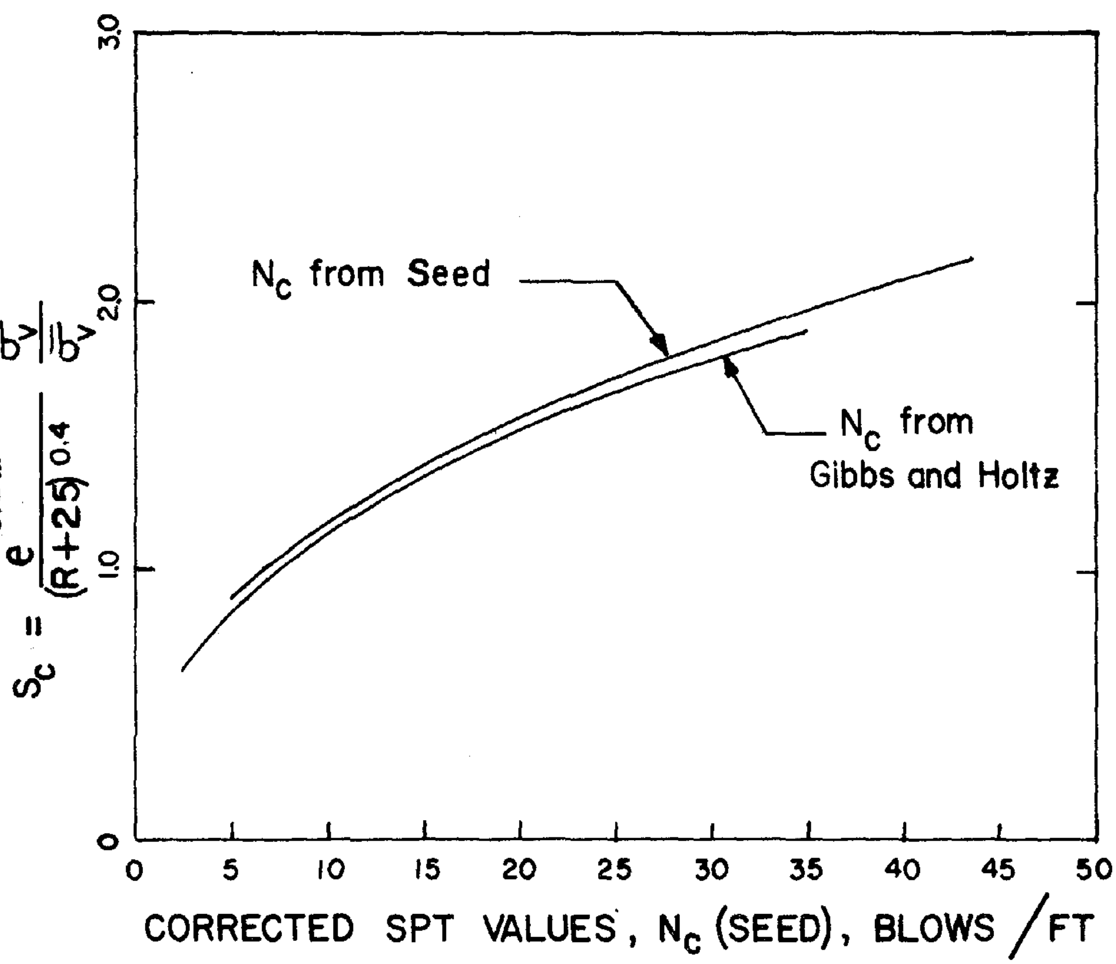


Fig. 3-2 Comparison Between Liquefaction Criteria Obtained Using Corrected Blow Counts from Seed (1976), and Gibbs and Holtz.

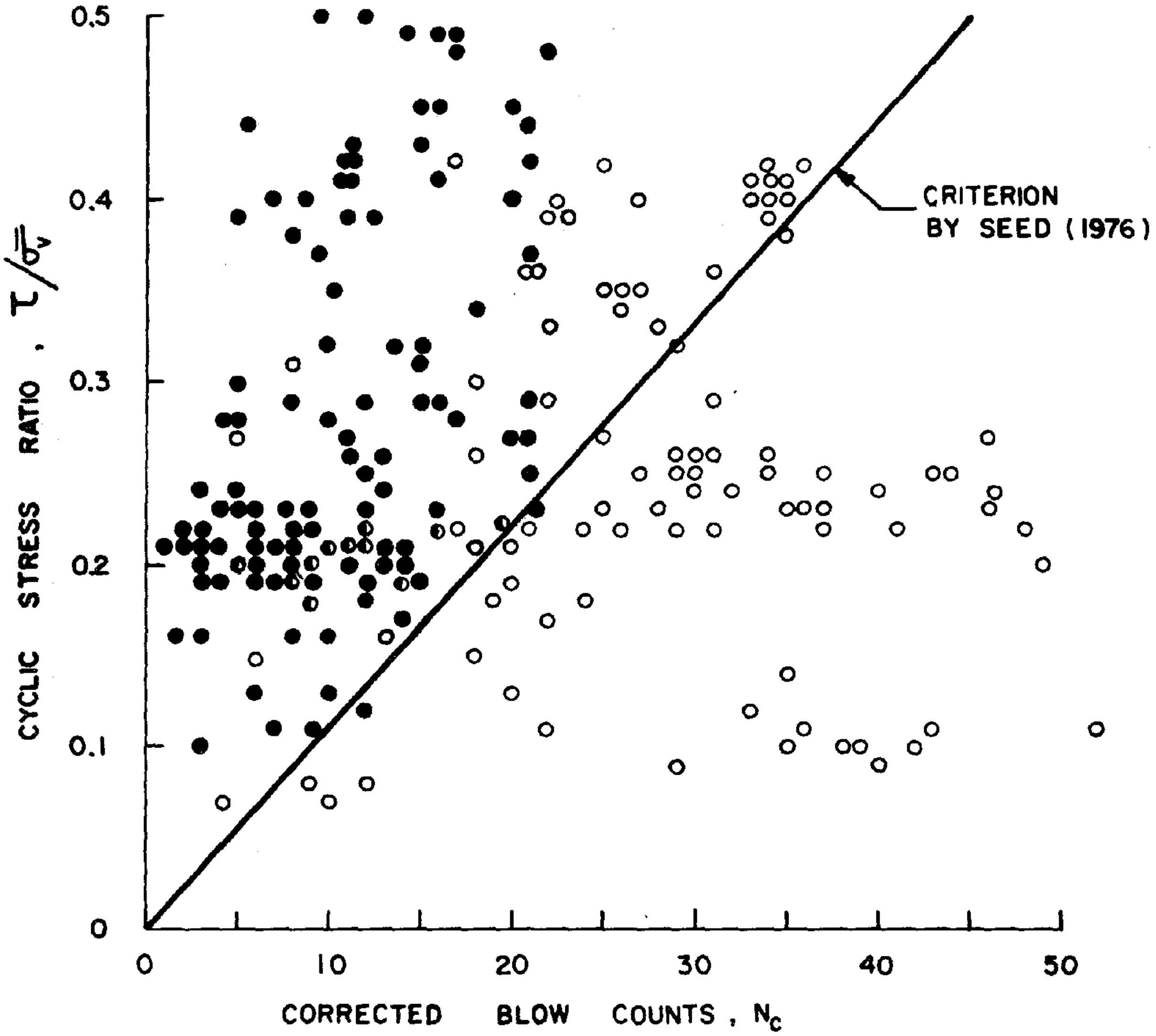


Fig. 3-3 Evaluation of Seed's (1976) liquefaction criterion.

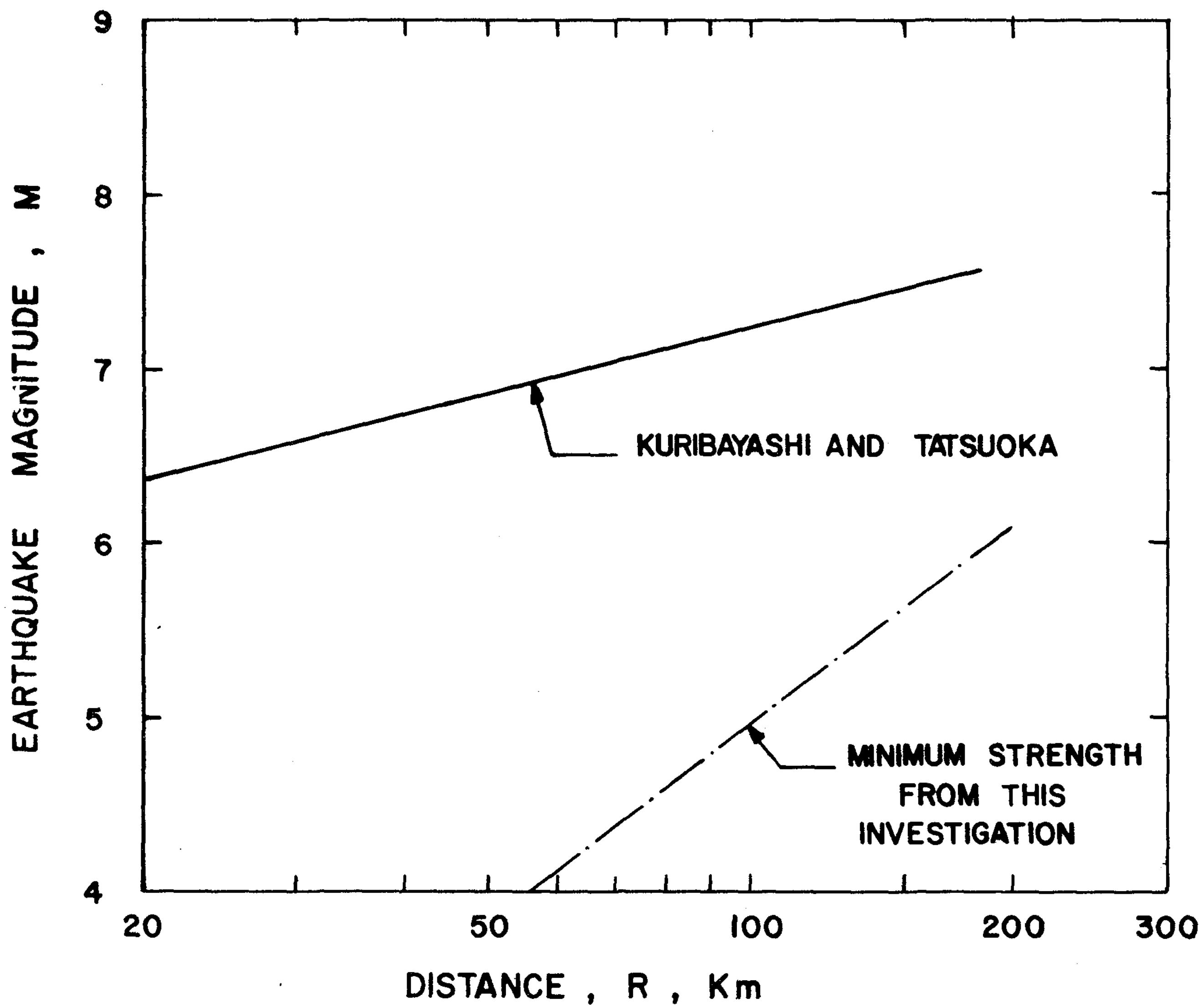


Fig. 3-4 Comparison Between Liquefaction Criterion of Kuribayashi and Tatsuoka, and Criterion from this Investigation Using a Minimum Value for Soil Strength.

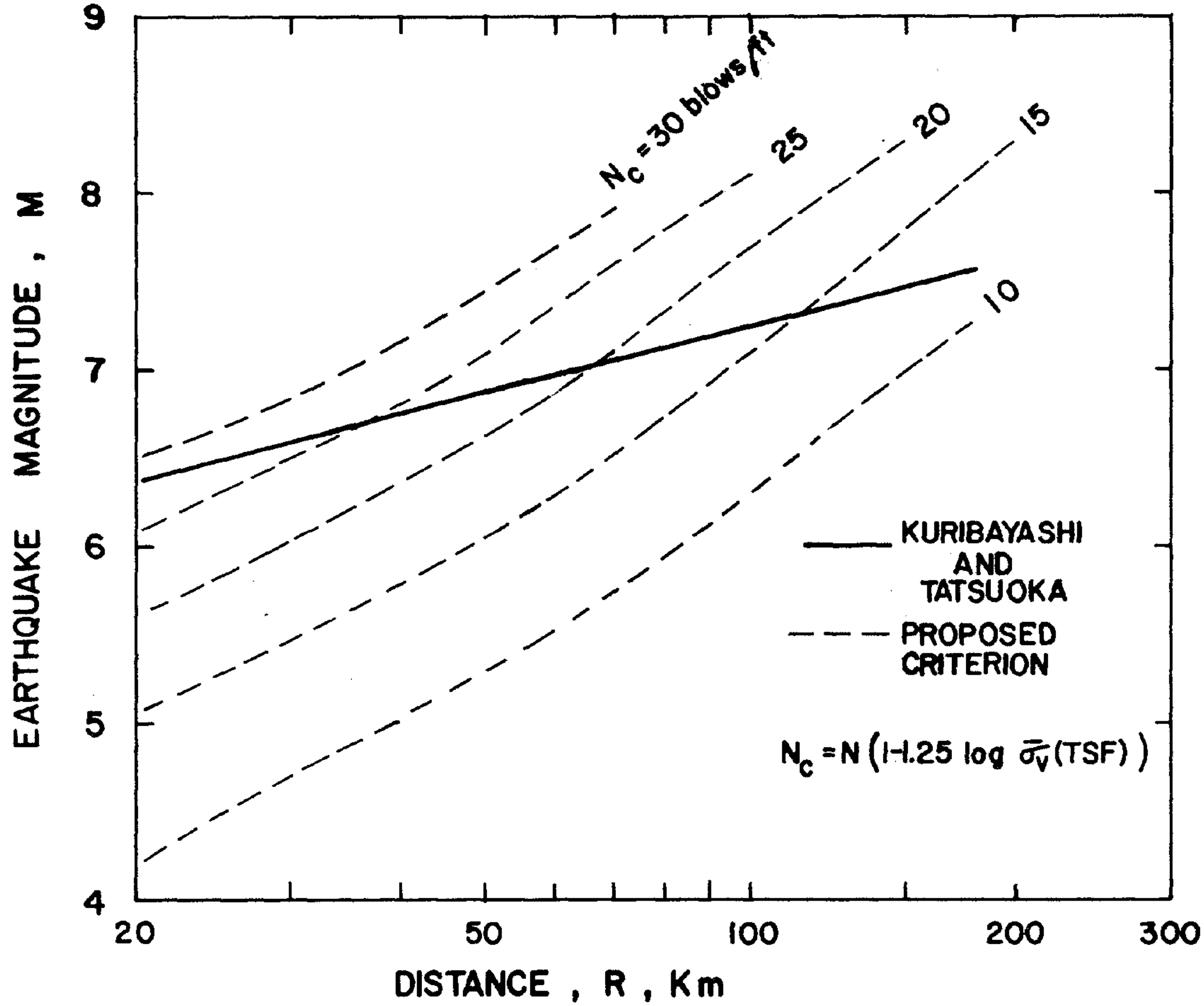
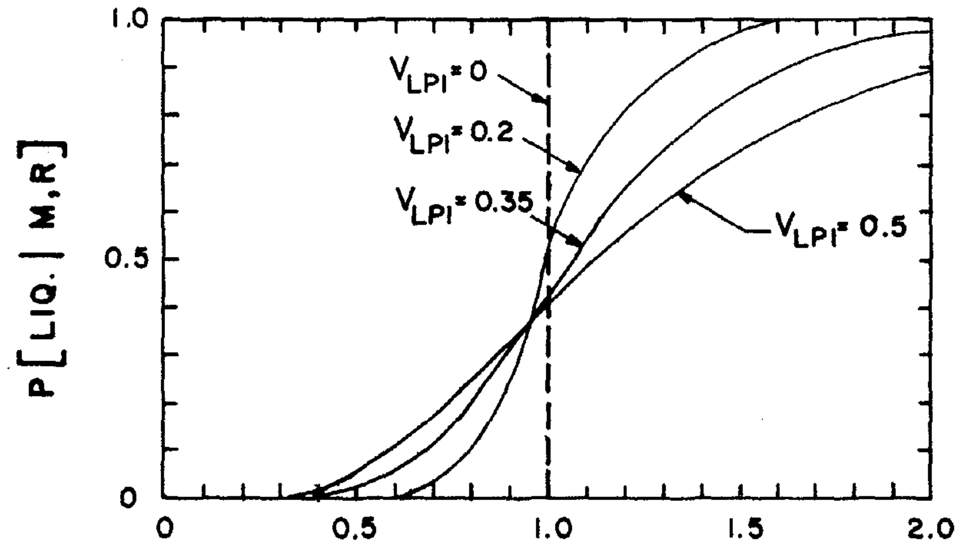
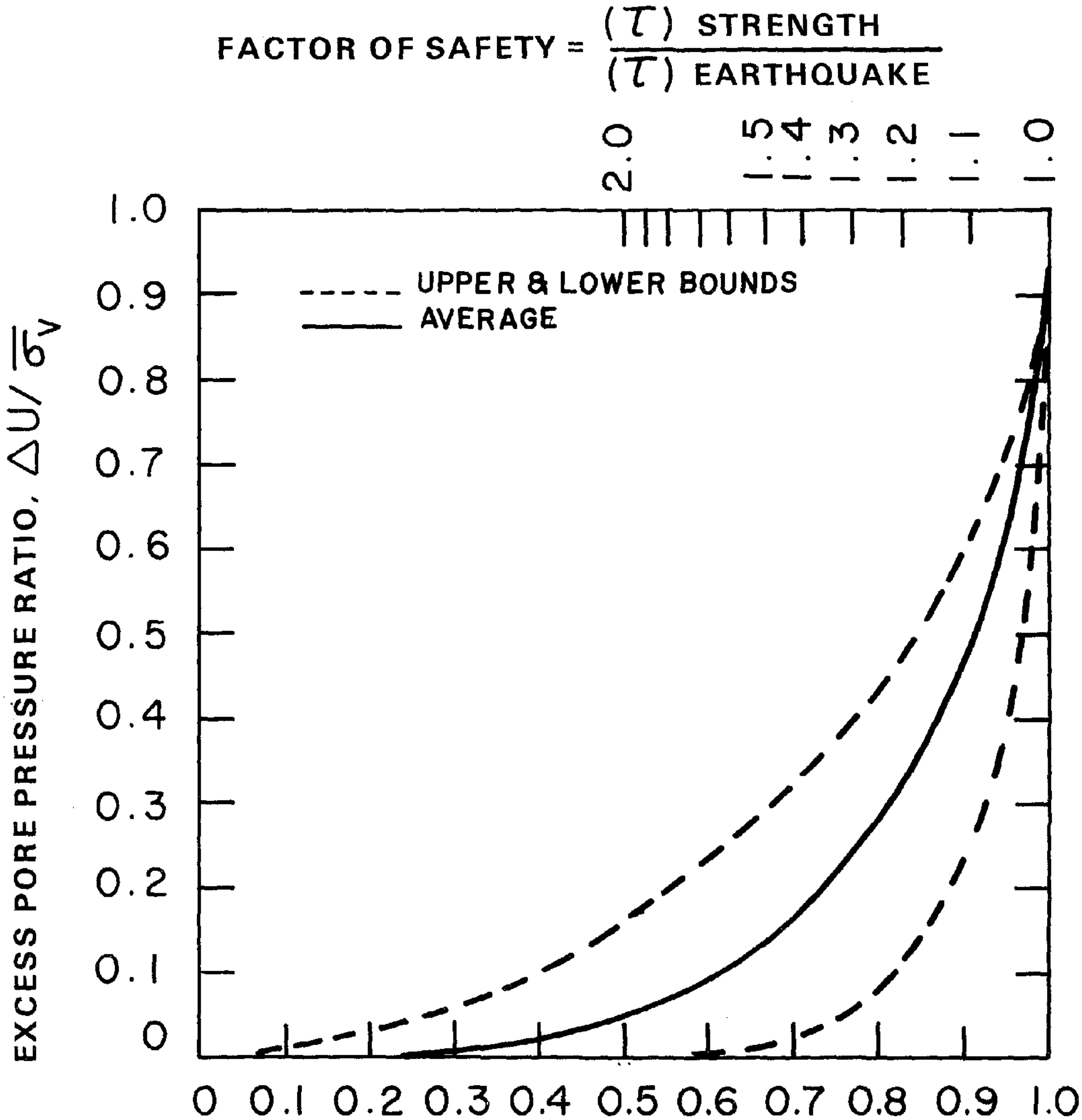


Fig. 3-5 Comparison Between Liquefaction Criterion of Kuribayashi and Tatsuoka, and the Proposed Criterion



$$LPI = \frac{e^{0.2M}(R+25)^{-0.4}}{0.464 N_c^{0.4}} \cdot \frac{\sigma_v}{\bar{\bar{\sigma}}_v}$$

Fig. 4-1 Conditional Probability of Liquefaction versus LPI.



$$LPI = \frac{e^{0.2M}\,(R+25)^{-0.4}}{0.464\,N_c^{0.4}} \cdot \frac{\sigma_v}{\bar{\sigma}_v}$$

Fig. 4-2 Excess Pore Pressure Ratio versus Liquefaction Potential Index, LPI.

REFERENCES

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APPENDIX A

DESCRIPTIONS OF SELECTED EARTHQUAKES

Mino-Owari (1891)(E3);
Tohnankai, (1944)(E7);
Fukui (1948) (E8)

The earthquakes occurring at Mino-Owari, Tohnankai and Fukui, with magnitudes of 8.4, 8.3 and 7.2, respectively, were the subject of Kishida's investigation into the occurrence of liquefaction. Based on the field behavior of sands taken from case histories, presented by Yokoo and the Geographical Survey Institute (1949, printed in Japanese), Kishida (1970) proposed criterion for assessing liquefaction potential of a level deposit of soil during an earthquake. In his paper, Kishida presents the results of field investigations at the places where sand volcanoes were formed and eruption of water and soil was observed during these earthquakes. The soil strata which were presumed to have liquefied were identified on the basis of previous studies and compared with the observed phenomena during the earthquakes. The results indicate that the eruption of water and soil during the earthquakes was not necessarily associated with a complete liquefaction of sand of the type observed during the Niigata earthquake of 1964. Included within the paper are microscopic photographs of sands which were presumed to have liquefied. Seed and Idriss (1971), in their documentation of results, have also presumed these deposits to have liquefied.

Borings were taken at the places where sand volcanoes and the eruption of water and soil occurred. Borings were also made for comparison at the places where such phenomena was not observed. Kishida (1970) has, with the aid of Standard Penetration Tests carried out in the field and the results of lab tests, attempted to identify the liquefied zone of the soil on the basis of his own previous research and has checked his results with the case histories presented by Yokoo and the Institute.

In all, 9 sites are taken from this study of these earthquakes for documentation by Seed and Idriss (1971) in their presentation. These nine cases, plus two additional ones, are documented in this report along with the corresponding soil profile and Standard Penetration Test results, for the scrutiny of the reader.

Chile (1960) (E11)

The 1960 Chilean earthqake, with a main shock of 8.4 on the Richter scale, caused extensive damage to structures. The extreme importance of foundation soils in resistance to failure was clearly evidenced. Substantial settlements, rotations and displacements were the results of soil failure. One such incident of foundation failure was reported by Steinbrugge and Clough (1960):

> "A building which experienced liquefaction of its foundation materials was the reinforced concrete hotel Perez Rosales on the Puerto Montt waterfront. This hotel, almost completed at the time of the earthquake, had a 5 story center section with one

> and two story wings on either side. Footings were the reinforced concrete spread type. During the earthquake the building sank, with the waterfront side settling approximately 15 inches more than did the opposite side. The smaller wings did not settle as much as did the main building. The result was differential settlement between the main structure and its wings which caused spectacular structural damage."

As in the case just cited, many cases of liquefaction of loose sandy soil occurred and were the apparent cause of a substantial number of failures. Considerable field evidence suggests that certain soils had been temporarily rendered "semi-liquid" by the earthquake, indicating liquefaction.

The soils of interest have been formed by the usual processes in the region of heavy rainfall, influenced by the deposition of volcanic ash and by glaciation. The soil that formed the foundation of structures in many cases consisted of morraines or glacial outwash.

Puerto Montt proved to be one of the most interesting locations for soil investigation. The topography of the city drops rapidly by 107 meters, from glacial terraces at the northerly edge of the city, to low flat land along the waterfront. The entire length of the waterfront consists of a surface soil of artificial fill, primarily gravel, mud and sand (6). The disastrous damage to structures supported by or retaining this material is considered to be due largely to liquefaction, as a result of the earthquake motion. The hydraulic fill of the harbor proper and the dumped fill in the naval base area behaved in a manner so as to produce almost total damage. These loose sandy soils were so affected by the earthquake motion that they lost essentially all of their shear strength. In many cases along the waterfront, total devastation was evident.

Boring data for a site of a gravity quay wall on the Puerto Montt waterfront revealed the following about the backfill and lower strata:

Elevation, m	Description	Standard Penetration blows/foot
+9.5 to +4.5	Hydraulic fill of fine sand with silt and clay.	5
+4.5 to -6.0	Fine sand and coarse particles	30
-6.0 to -10.5	Gravel	Too hard to measure
-10.5 to -15.5	Consolidated soft material with coarse sand	

The upper backfill was very loose, almost in a quick condition when inspected one month later.

A very heavy rain had fallen prior to the earthquake even though the rain season still lay in the future. This helped account for the high degree of saturation of the soils.

Qualitatively, this earthquake has proven to be quite instructive as to foundation failures; however, quantitatively, it has proven less so. Seed and Idriss (1971) included three cases in their paper, all of which are taken from the Puerto Montt area. These are apparently the results of private communication with Lee as noted in their paper. Unfortunately, this earthquake, with such extensive occurrence of liquefaction, has not been better studied in a quantitative manner.

Niigata (1964) (E12)

The Niigata earthquake of 1964 occurred with a magnitude of 7.5 on the Richter scale. Its focus, considered to lie within an old structure, was at a depth of approximately 40 km. The epicenter of the earthquake, according to the Japan Meteorological Agency, fell at 38.4° N. Lat. and 139.2° E. Long. The Building Research Institute had installed an SMAC type strong motion seismograph in the basement and a D.C. type strong motion seismograph on the 4th floor of an apartment building at Kawagishi-cho, Niigata City (the next apartment building overturned during the earthquake). These instruments recorded a maximum acceleration of 0.19 g in the basement location (10).

The site condition prior to the earthquake has proven to be of great significance to the occurrence of such widespread liquefaction of saturated sands during the earthquake.

In the years previous, the Niigata area had suffered intense ground subsidence. Through the years 1957-1959, the largest rate of settlement was observed at as much as 56 cm during one year. Investigation confirmed that the subsidence resulted from an excess pumping up of underground water mainly for the natural gas industries. The rate of settlement had been controlled by the year 1963 to be as low as 6 cm per year (9).

Emptying into the sea at Niigata are two major rivers, the Shinano River and the Agano River. These rivers have been the cause of repetitions of flooding with enormous quantities of flooding soils being deposited downstream. The attacks of flooding were so frequent that the history of Niigata might be deemed as the history of flood control (45). Loose and thick deposits of sand down to depths of 40 meters are characteristic of subsoil condition along the river banks in and around the city of Niigata.

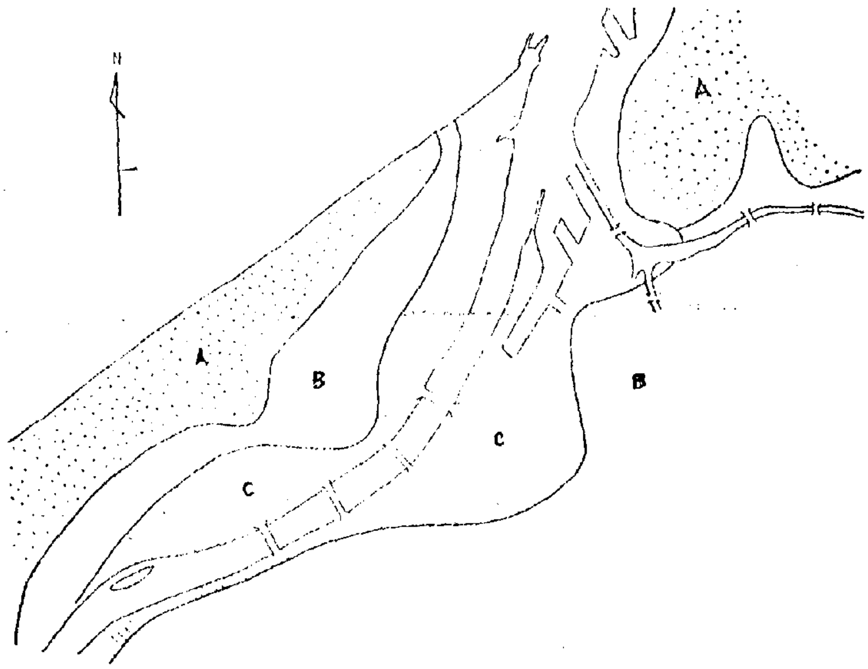
It has been a unanimous opinion that the earthquake damage in the Niigata area was characterized by the subsoil condition, that is, the damage was aggravated by the occurrence of liquefaction (9).

For the purpose of investigation of subsoil-related damage and corresponding intensities, Niigata was divided into three areas:

Area A - where no damage or only slight damage to buildings occurred.

Area B - where buildings suffered intermediate damage.

Area C - where buildings were damaged very heavily.



After the earthquake, much effort was expended to explore the subsoil condition in these three zones. Fortunately, due to the history of subsidence in the city and plain area, there existed many records of boring data along with test results of soils samples. One striking feature of Niigata sand being found from this data is that its grain-size distribution is extremely uniform. Coefficients of uniformity are generally smaller than 5 (21). Extremely small damage in "A" Zone as compared to "B" Zone may be attributed to the fact that "A" Zone consists mainly of sand dune with the ground water located at a deep level, while in "B" Zone the ground surface is flat and low with ground water at a depth less than 0.5 m. Loose sand, with extremely low blow count and almost ground surface ground water in Zone "C" probably is the cause of such heavy damage.

In those areas of Niigata City where structural damage was appreciable, there were numerous sand craters indicating the presence of excess pore water pressure within the sand, which is a prime characteristic of liquefaction in sands. Comprehensive analysis indicates that the ejected sand originated from depths not exceeding 10 m (10).

Again, it should be emphasized that, in the usual case, extent of damage is expressed by means of damage to the superstructure; however, as far as the Niigata earthquake concerns, there was little damage to the superstructure. Rather, the majority of damage was due to soil liquefaction. In all, 5 sites were documented for this case study. The soil information for each site are summarized in Table 2-2 and the soil profiles are included in Appendix B.

Alaska (1964) (E13)

The Alaskan earthquake of 1964 has been recorded at Richter scale readings varying from 8.3 to 8.6 (34). The focal depth was recorded to be between 20 to 30 km. It occurred with such intensity that a large amount of territory was affected, including Valdez and Anchorage with surrounding areas. Bridge structures, land slopes, railroad embankments and other buildings were damaged by the quake.

Soil condition was found to be an integral part of the cause of damage (25). Many of the bridge structures traversed rivers with banks consisting mainly of sands, with some silts, in a saturated condition. Leaning of piers and slumping of banks indicate liquefaction in many places (25).

In the Valdez area, slumping of the waterfront soils was determined to be caused by liquefaction (3). The land slopes to the sea from the north end of Valdez. Slides which occurred to the North of Valdez have not been labeled as having been triggered by liquefaction, but slides which had occurred at the dock area were. The soil at Valdez was found to contain horizontal continuity and vertical uniformity (3).

The geologic environment in Valdez provides optimum conditions for the phenomena of liquefaction and bears many similarities to other liquefaction flow sites (3).

Ebino (1968) (E15)

Beginning on February 21, 1968 a series of earthquakes occurred in the zone of Ebino Town, commonly referred to as "the 1968 Ebino earthquakes". The magnitude of the second and greatest of the earthquakes was registered at 6.3 on the

Richter scale. The epicenter occurred approximately 8 km to south of Ebino Town with a focal depth of not more than a few kilometers (39).

Much damage resulted, involving buildings, various slopes, embankments, bridges and highways. Most of the damage can be attributed to the sediment of a volcanic product referred to as "Shirasu".

Some question has arisen as to whether the Ebino earthquakes are volcanic or structural in origin. The Ebino district lies, when seen from the viewpoint of geological structure, right on the markedly volcanic structure line of the Kirishima volcano and the Kakuto formation group (39). At some parts, however, the geology is extremely folded with minor faults and at other parts the beds are disturbed. The axis of syncline and anticline presents an arc, with its hollow part directed toward Mt. Iimori, approximately 7.8 km to the south of the center of Ebino Town (39).

The so-called Shirasu (white sand) is a Pleistocene volcanic product in terms of geological age, and in origin it is classified with the unwelded part of pumice falls and their secondary deposits. In its natural state, it presents loose rock texture, but when disturbed is merely a sandy granular material. Shirasu mass mainly consists of pumiceous fragments or flakes of the size ranging from very fine grain to fist size. The effective unit weight when submerged is very small because the specific gravity of Shirasu particles is small and the void ratio is compartively large. The grading of the Shirasu of the Ebino district is fine and uniform.

The results of investigation made on the soil where sand boils occurred at the time of the earthquake show that Shirasu that lay accumulated under the barley fields at about 3.0 meters deep was ejected above the common sand that had sedimented over it. The depth of the ground water was approximately 1.0 meter. At another location, the Shirasu lying 3.0 meters below the surface with ground water at 1.75 m depths was ejected. The blow counts recorded indicate a maximum of approximately 7 (SPT results).

Shirasu is much different from common sand in that liquefaction does not occur with common sand, generally, if density is high, but with Shirasu liquefaction will occur even at 100% relative density (39). Shirasu is much more susceptible to liquefaction than other sands (40). However, according to research by Yamanouchi and Mori (1970), "undisturbed" samples of alluvial Shirasu including silt obtained in Kagoshima City, could not be liquefied. By this fact, they pointed out, liquefaction of Shirasu is a problem of primarily "clean" Shirasu.

The areas where sand boils occurred with Shirasu were studied and two borings are included in the tables with all available data.

Tokachioki (May 16, 1968) (E16)

The Tokachioki earthquake of May 16, 1968 occurred off the Pacific coast of northern Japan with an epicenter, as reported by the Japan Meteorological Agency, at 40.7° N. lat. and 143.7° E. long. and a focal depth of approximately 20 km. The magnitude as recorded on the Richter scale wad 7.8 (20).

The earthquake was responsible for extensive damage to buildings, natural slopes, embankments, earth dams, port facilities and for the liquefaction and subsidence of sandy ground at several locations. Complete liquefaction of level sandy ground (saturated) took place in recent hydraulic fills, loose backfills and in swampy lowlands. In some cases loose backfills of sand above ground water level, subsided considerably due to densification, indicating no liquefaction. Damage in many cases could be attributed to the heavy rains which preceeded the earthquake, producing a high ground water level. Also cited as an important cause of damage was the long duration of the strong ground motion registered at about 2-3 minutes.

Many field investigations were carried out following the earthquake and in the cases where it proved possible, comparison was made with available records from before the earthquake. A few of those cases are now cited in the following:

Field studies to clarify the characteristics of liquefaction of level sandy ground were carried out at Nanaehama Beach near Hakodate in northern Japan. The hydraulic fills underwent complete liquefaction during the earthquake. Reclamation was made by hydraulic fills approximately three years before the earthquake. Water and soils began to spout out from the ground surface soon after the beginning of the earthquake. Many sand volcanoes were found (15). The results of analysis of the liquefaction of saturated sands based on Kishida's criteria (14) and on the observed phenomena indicated that the soil stratum which lay between about 1 and 5 meters deep had a high potential for liquefaction. The grain composition and the roundness of soil particles in the soil stratum shows good agreement with the soil found in the sand volcanoes. This corresponds with the hydraulic fill (15).

Another site of sandy ground showed a variety of damage features due to liquefaction. A portion of the ground consisted of loose sand and the other part was of dense sand, the latter suffering little damage. The denser portion had been compacted by means of vibroflotation, which demonstrated in this case its effectiveness for preventing liquefaction. The site studied was a paper manufacturing plant located in the city of Hachinohe which lies in northern Japan, 560 km north of Tokyo. The ground consists of sands almost entirely down to a depth of more than 20 meters from the ground surface. The top 5 meters had been excavated and backfilled with waste sand, this occurring over the majority of the site. From the experience obtained from the earthquake at Niigata 1964, recognition of the liquefaction problem led to the adoption of a pile foundation and the use of the compaction technique of vibroflotation. This application proved successful and while excavated and backfilled ground liquefied almost all over the site cuasing subsidence and cracking with eruption of sand and water, compacted portions of the ground remained intact. Many structures and facilities were damaged severely. Comparison was made of before and after soil boring data at three locations and documented in this report (23).

Almost all of liquefaction occurrences took place in the districts where loose saturated sands had sedimented at about 40 meters thick. Soil profiles and associated blow count data for these cases just cited and others are included within the context of this report.

APPENDIX B

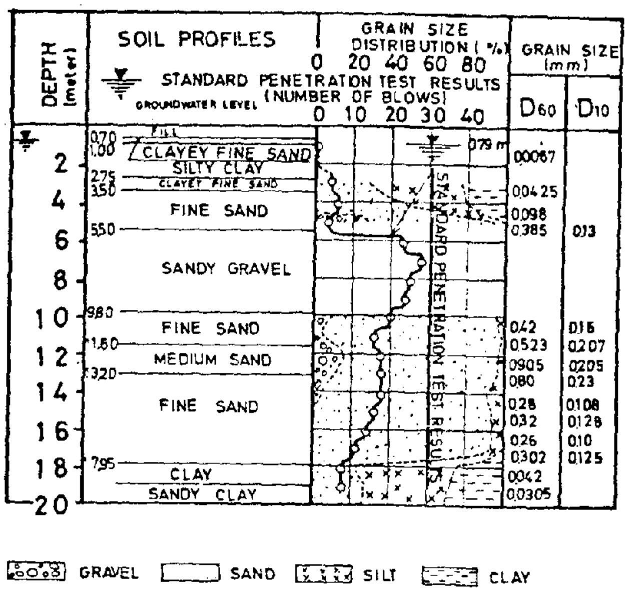
SOIL PROFILES

Earthquake: Mino-Owari, Japan (1891) (E3)
Location: Ogaki City

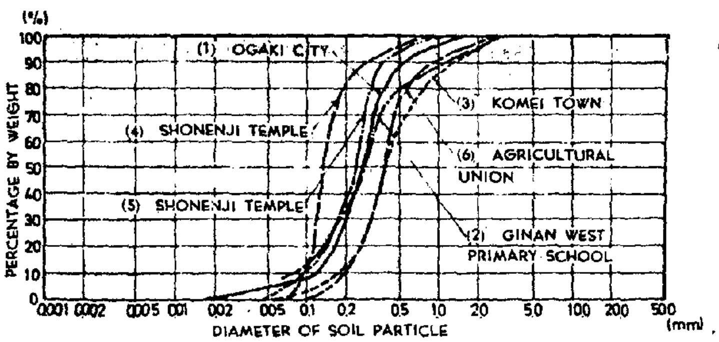
Site Condition (23):

Area was in an artesian condition and artesian wells were used.

"Eruption of water and soil was observed. It is not clear whether the eruption of water resulted from a liquefied condition or from an artesian condition."



(I) ŌGAKI CITY BANGOKU TOWN



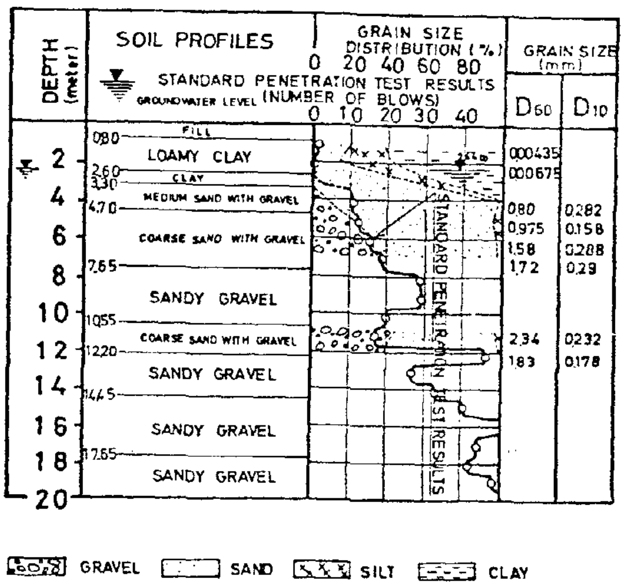
(7) **Grain Size Distribution Curves**

Earthquake: Mino-Owari, Japan (1891) (E3)
Location: Ogase Pond

Site Condition (14):

"Located near small hills and might be in an artesian condition."

"Cracks were found and white mixture of soil and water came out from ground."

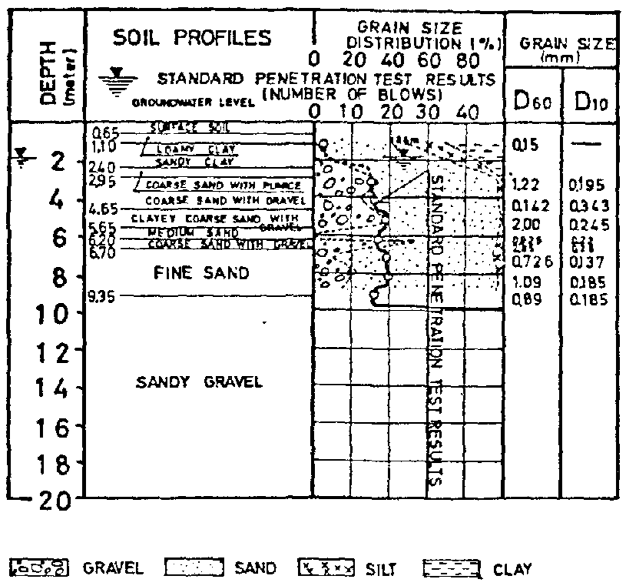


(IV) OGASE POND

Earthquake: Mino-Owari, Japan (1891) (E3)
Location: Unuma Town

Site Condition (14):

"Located near small hills and these areas might be in and artesian condition."

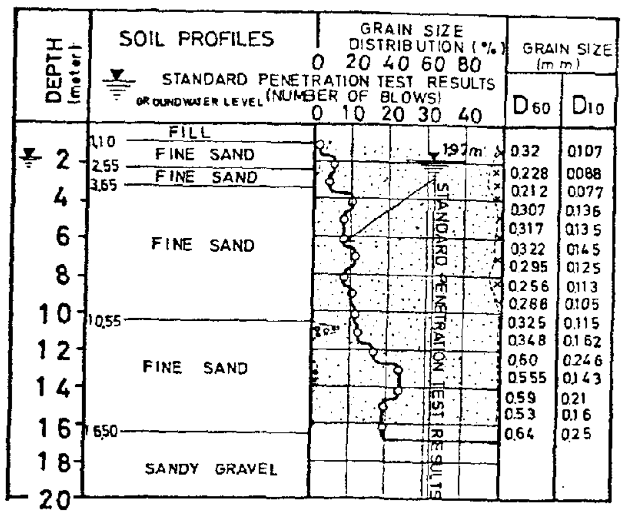


(III) UNUMA TOWN

Earthquake: Mino-Owari, Japan (1891) (E3)
Location: Ginan West Primary School

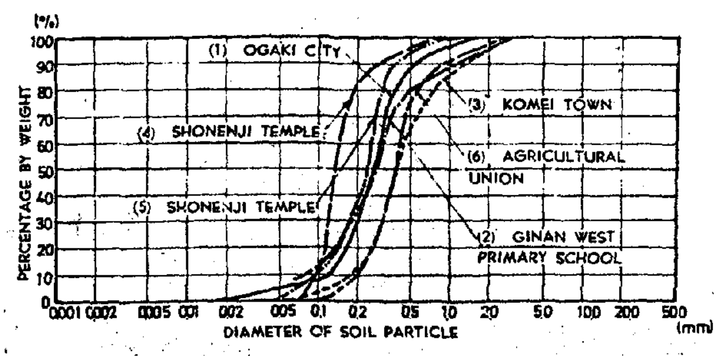
Site Condition (14):

"Many sand volcanoes occurred during earthquake."



GRAVEL SAND SILT CLAY

(II) GINAN WEST PRIMARY SCHOOL

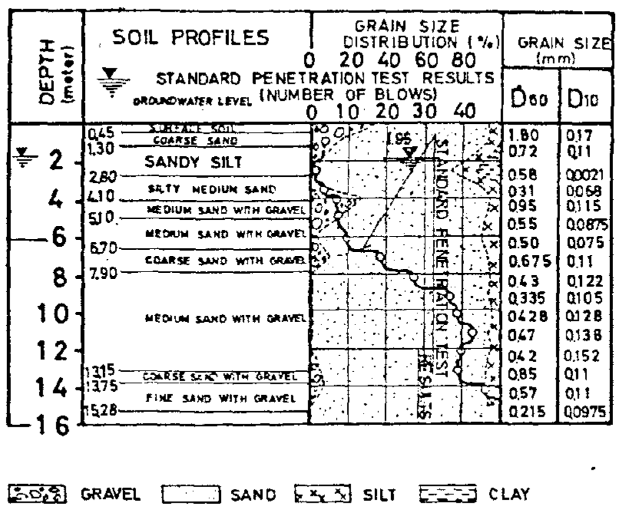


(7) Grain Size Distribution Curves

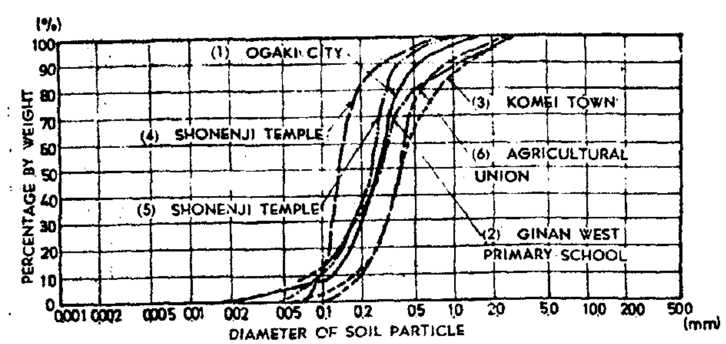
Earthquake: Tohnankai, Japan (1944) (E7)
Location: Komei Town

Site Condition (14):

Subsidence of the ground and extensive damage to houses occurred as a result of liquefaction of the sand. A temple which was supported on piles did not show any settlement but the ground around the temple subsided ~40 cm and water erupted during the earthquake.



1) KOMEI TOWN

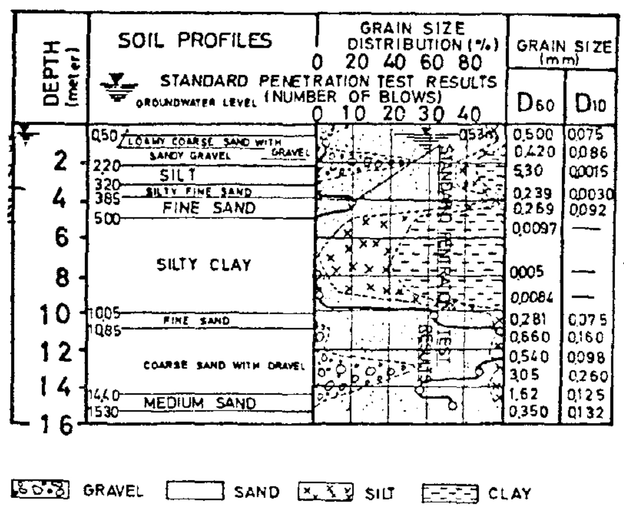


(7) **Grain Size Distribution Curves**

Earthquake: Tohnankai, Japan (1944)(E7)
Location: Meiko Street

Site Conditions (14):

Very fine soil came out from the ground and houses settled as much as about 1.00 meter.



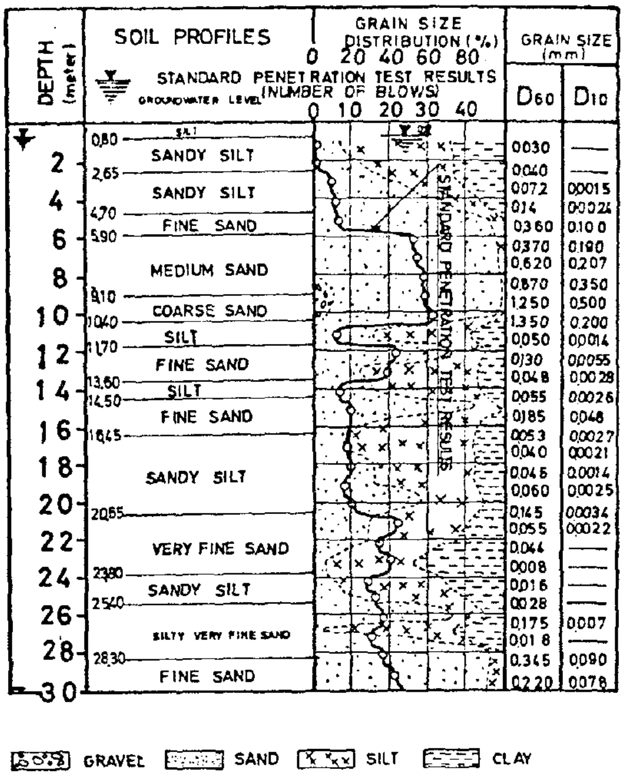
(II) MEIKO STREET

Earthquake: Fukui, Japan (1948) (E8)
Location: Takaya Town 2-168

Site Condition (14):

Old village where the ground level is about 1.00 m higher than the surrounding paddy field.

"No eruption of water and sand volcanoes were found."



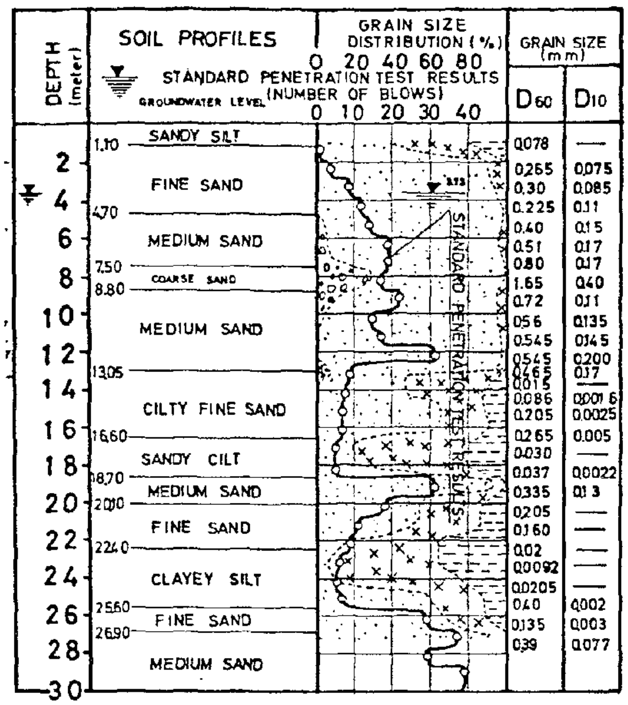
(II) TAKAYA TOWN 2-168

Earthquake: Fukui, Japan (1948) (E8)
Location: Takaya Town 45-35

Site Condition (14):

Paddy field which drains rapidly when the irrigation pump stops.

Sand volcanoes were recognized—sand volcanoes are approximately parallel with the Kuzuryu River. Much eruption of waters.



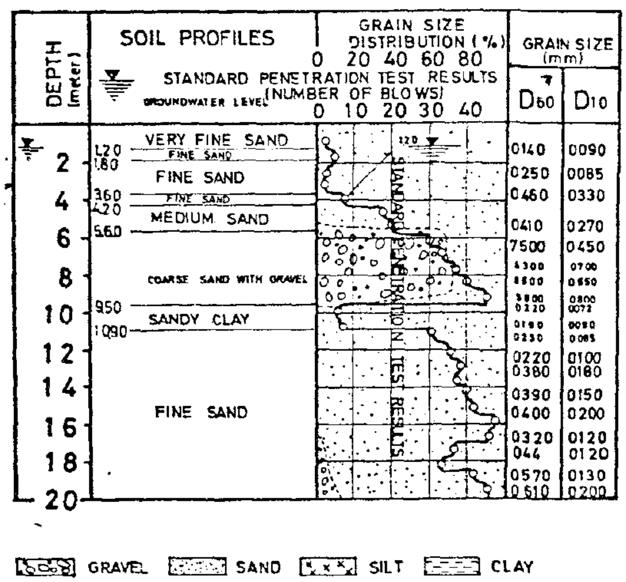
GRAVEL SAND SILT CLAY

(1) TAKAYA TOWN 45-35

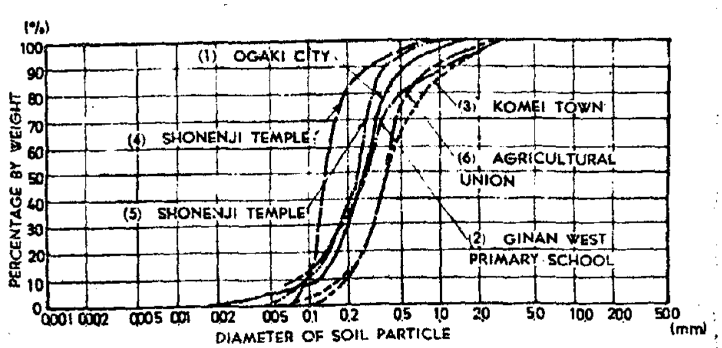
Earthquake: Fukui, Japan (1948) (E8)
Location: Shonenji Temple

Site Condition (14):

Eruption of water and sand volcanoes were quite prominent and the main building of the temple settled 0.30 m as a result of the liquefaction of the sands.



(III) SHONENJI TEMPLE



(7) **Grain Size Distribution Curves**

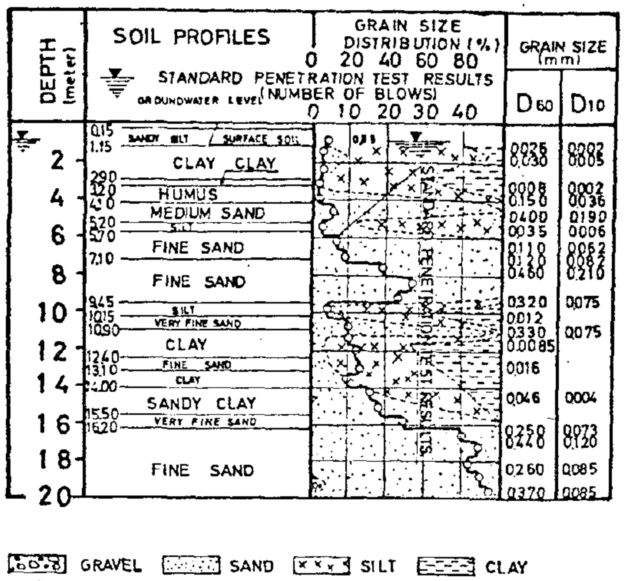
No Liquefaction

Earthquake: Fukui, Japan (1948) (E8)
Location: Agricultural Union

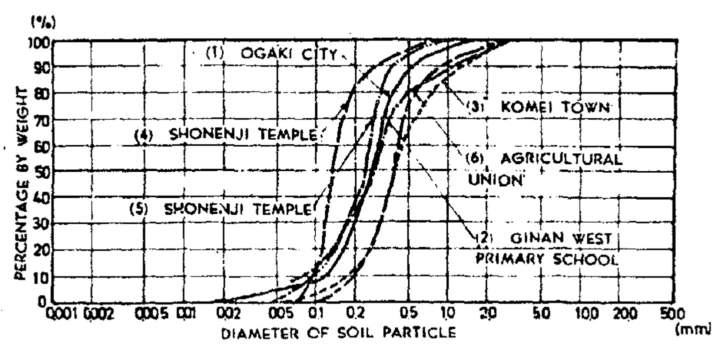
Site Condition (14):

Located in center of old village—the ground level of which is a little higher than the surrounding paddy field.

"No eruption of water and sand volcanoes."



(IV) AGRICULTURAL UNION



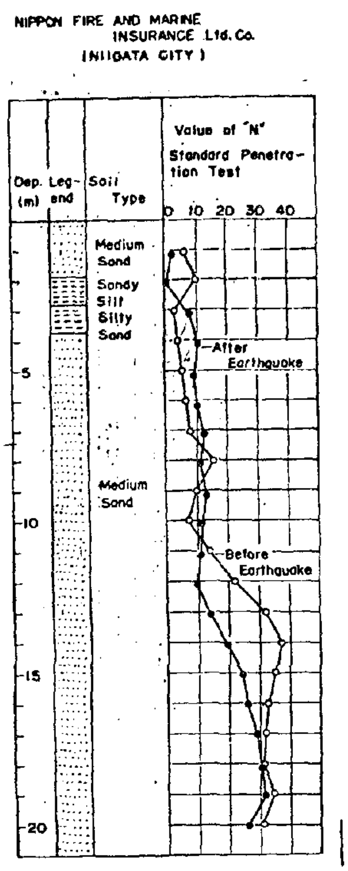
(7) **Grain Size Distribution Curves**

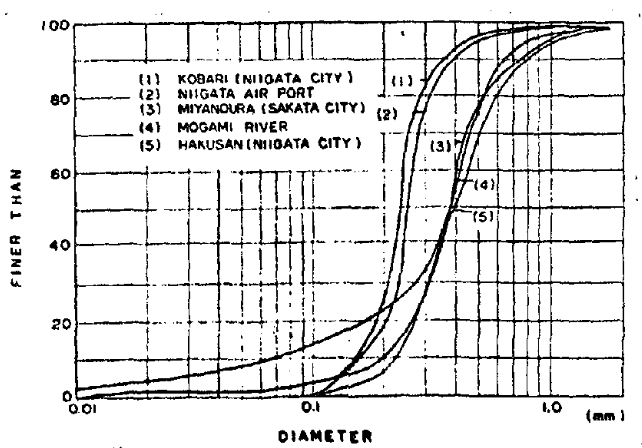
Earthquake: Niigata, Japan (1964) (E12)
Location: Nippon Fire and Marine Insurance, Niigata

Site Condition (11):

Concrete Building—"The grain size distribution curves of materials in the ground which liquefied are shown below. Liquefaction 'may have' occurred in coarser materials as well as in medium-fine."

It is supposed that the change of values of "N" is not caused by the quake of ground, but by the liquefaction and sedimentation of sand due to the earthquake. Further, the sand layer was estimated to be liquefied to the depth of ~16 m.

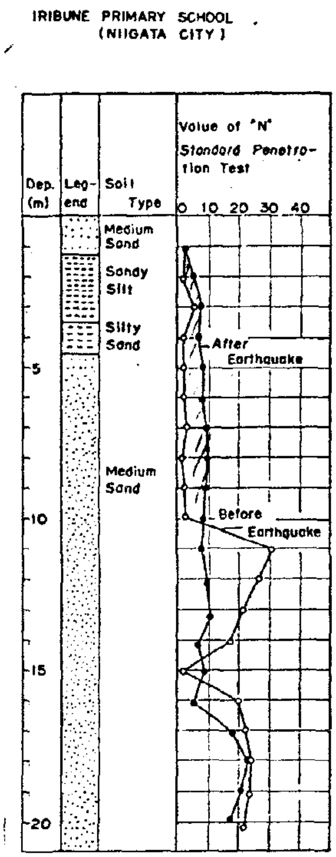




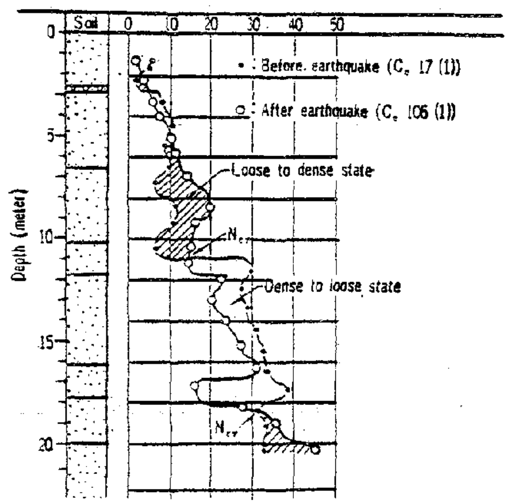
Earthquake: Niigata, Japan (1964) (E12)
Location: Iribune Primary School, Niigata City

Site Condition (11):

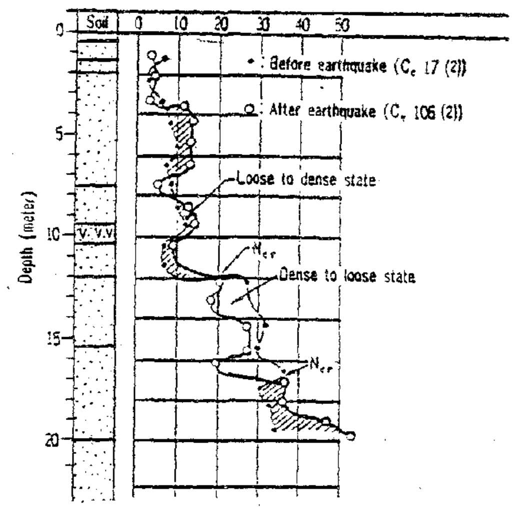
"The soil around the piles 6.0 m. long was in quick condition and the building settled about 1.00 m. all over the covered area. Building tilted about 2 degrees in short span direction."



Earthquake: Niigata, Japan (1964) (E12)
Location: Benten-cho (13)



Earthquake: Niigata, Japan (1964) (E12)
Location: Benten-cho (13)



Earthquake: Niigata, Japan (1964) (E12)
Location: Figure 34, Ohsaki (1966)

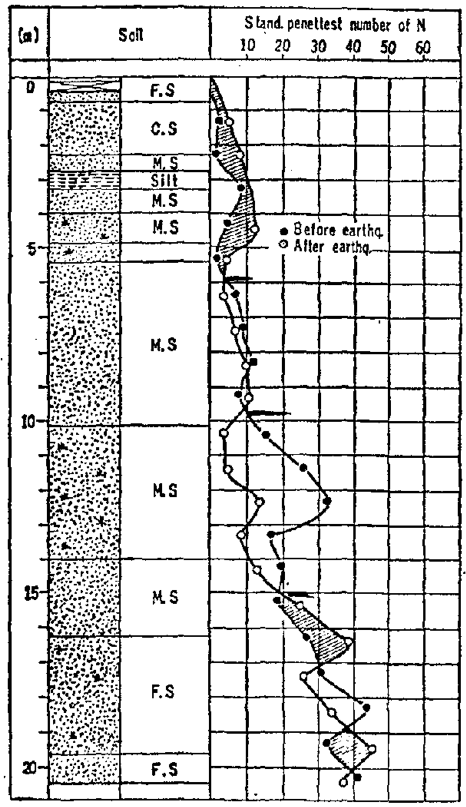


Fig. 34

Earthquake: Alaska (1964) (E13)
Location: Snow River Bridge 605A

Site Condition (25):

> "From the comments of first hand observers and from the behavior of the Bridge Foundation, it is clear that liquefaction of cohesionless soils did occur in this region. Reports mentioned 'mud' oozed up in cracks and that the 10' high road embankment was reduced to the level of the flood plain downstream movement of the footings and upstream tilts of the pier shafts indicate liquefaction at a depth below footing level."

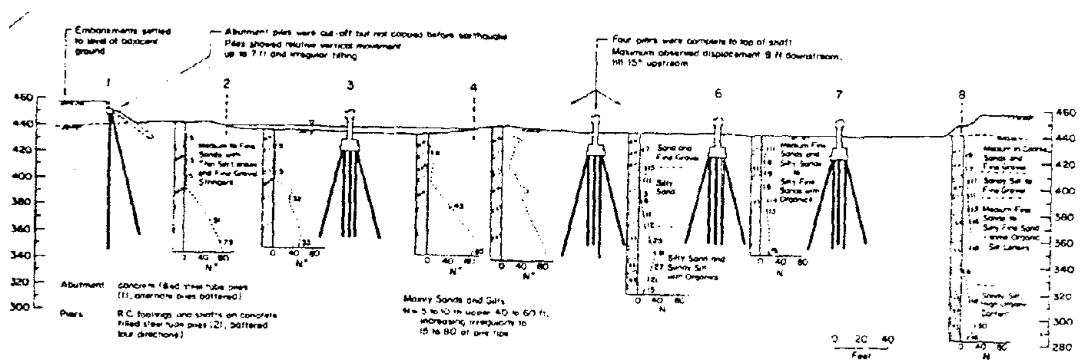


FIG. 6.—BRIDGE NO. 605A-SNOW RIVER 3
Under Construction, Centerline Section Looking Downstream, Natural Scale

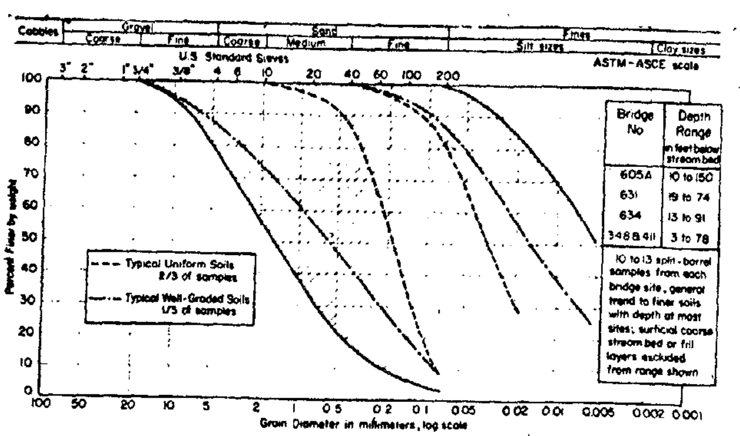


FIG. 22.—TYPICAL GRAIN-SIZE DISTRIBUTION RANGE FOR SANDS AND SILTS AT BRIDGE SITES WHERE SEVERE FOUNDATION DISPLACEMENTS OCCURRED

Earthquake: Alaska (1964) (E13)
Location: Snow River Bridge 605

Earthquake Information (3):

"Faulting: Thrusting of the continent over the ocean floor along a plane dipping 5°-15° North or NW or with Downwardslip of the continent along a near vertical plane—in either case the strike of the fault is NE in the vicinity of Kodak Island. Reverse faulting on a fault with strike N62°-72°E and a steep dip to the SE."

Site Condition (25):

Bridge structure—flat ground surface.

No specific comment on liquefaction for this bridge. "Abutments moved toward one another."

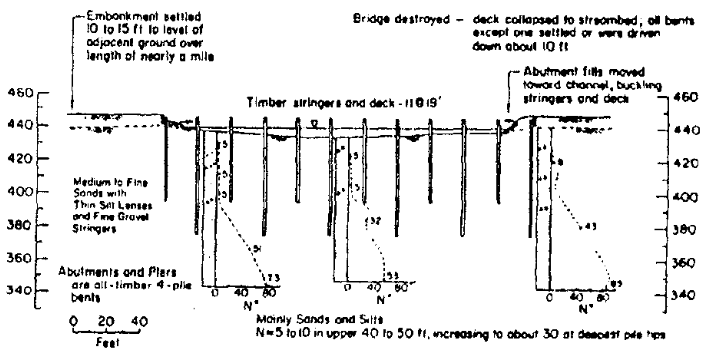


FIG. 7.—BRIDGE NO. 605-SNOW RIVER 3
Centerline Section Looking Downstream, Natural Scale

Earthquake: Alaska (1964) (E13)
Location: Quartz Creek Bridge 676

Site Condition (25):

Bridge structure—flat ground surface.

"Damage minor."

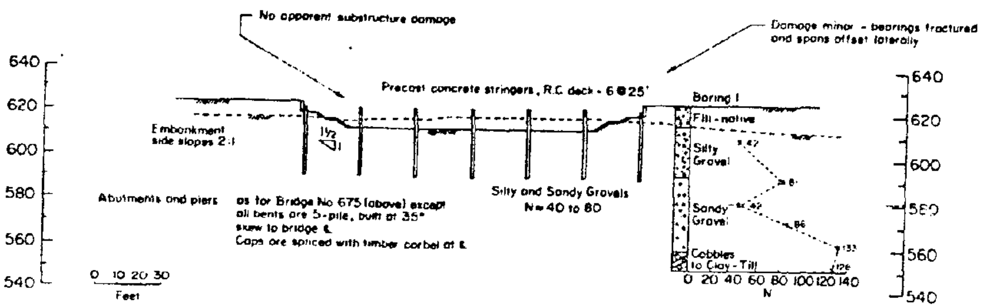


FIG. 11.—BRIDGE NO. 676-QUARTZ CREEK
Centerline Section Looking Downstream, Natural Scale

Earthquake: Alaska (1964) (E13)
Location: Scott Glacier Bridge 348

Site Condition (25):

Bridge structure over stream—flat ground surface founded on piles.

"In all these cases it seems likely that the major cause of damage was the liquefaction of the sandy and silty soils into which the pile foundations were driven. Evidence that liquefaction did occur: building settled ~2' into the ground and ground cracks up to several inches wide and several feet deep were observed near building site, small mounds of fine sand were noted alongside the fissures."

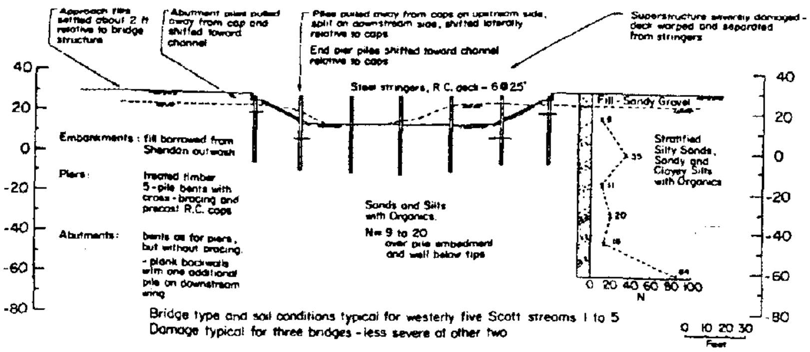


FIG. 17.—BRIDGE NO. 348-SCOTT GLACIER STREAM 1
Centerline Section Looking Upstream, Natural Scale

Earthquake: Ebino, Japan (1968) (E15)
Location: Ebino Town

Earthquake Information (39):

No definite opinion yet whether or not the Ebino earthquakes are volcanic or structural or if they have to do with 1968 Hyuga-Nada earthquake (Mag. 7.7)

Site Condition (39):

Shirasu (white sand)—sandy granular material lies 3.0 m below surface.

Sand boils occurred—Shirasu ejected. Since Shirasu is much different from common sand, such as found in Niigata in its granular properties, it was not known whether liquefaction could easily occur. Liquefaction does not occur with common sand if density is high but with Shirasu it occurs even at 100% relative density.

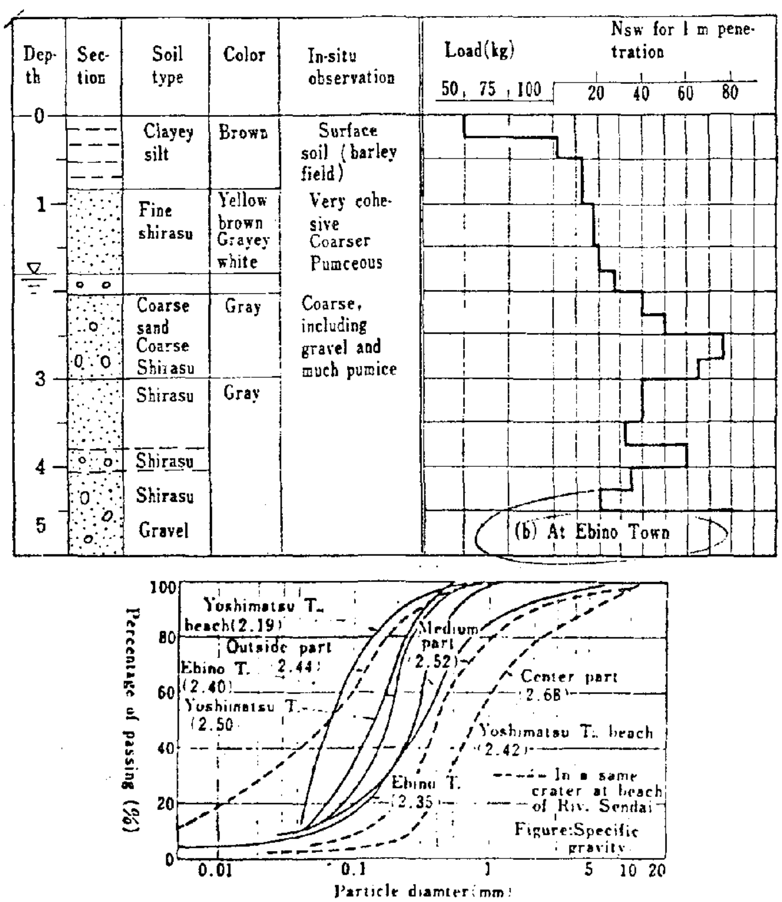


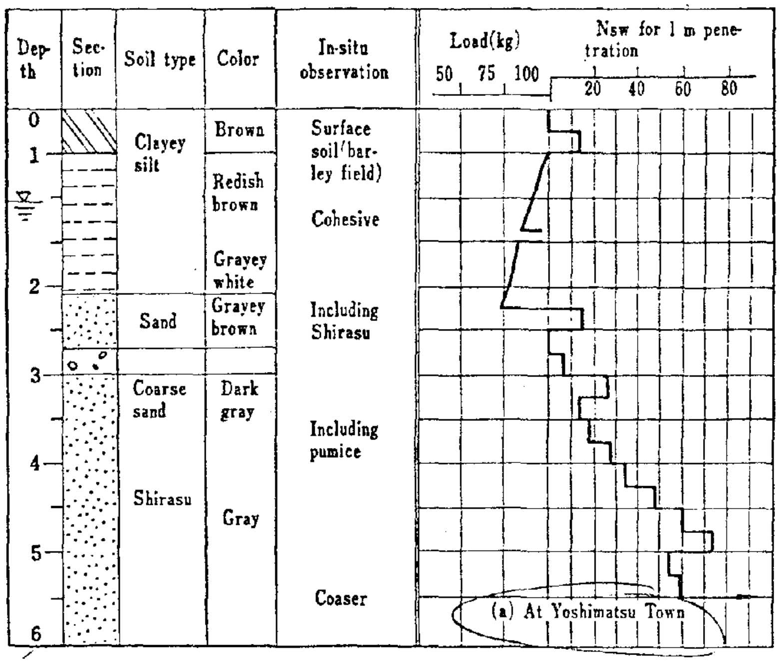
Fig. 7. Grading curves of ejected Shirasu in Ebino Earthquake (Yamanouchi et al.)

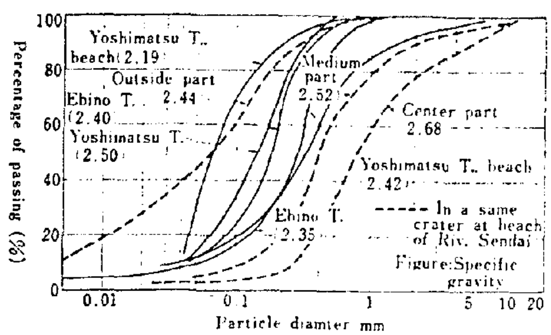
Earthquake: Ebino, Japan (1968)(E15)
Location: Yoshimatsu Town

Site Condition (39):

Barley fields—Shirasu lies accumulated 2.95 m deep.

Shirasu ejected above the common sand that had sedimented over it. What is noticeable about Shirasu is that it was more susceptible to liquefaction than other sands as evidenced in laboratory tests.





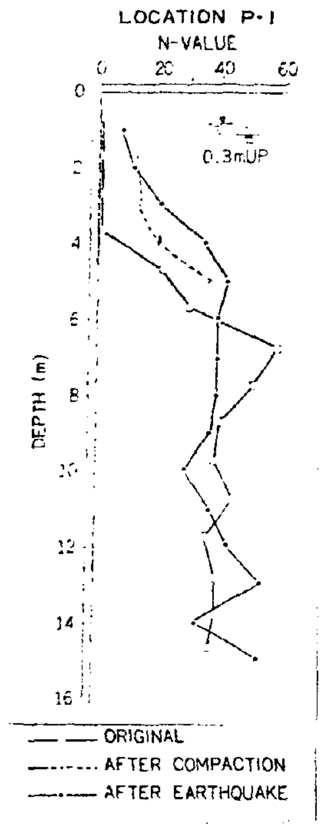
:. 7. *Grading curves of ejected Shirasu in Ebino Earthquake*
(Yamanouchi et al.)

No Liquefaction

Earthquake: Tokachi-oki, Japan (1968)(E16)
Location: Hachinohe P-1

Site Condition (23):

"The loose sands down to a depth of 5 to 6 m. had been compacted by means of vibroflotation."



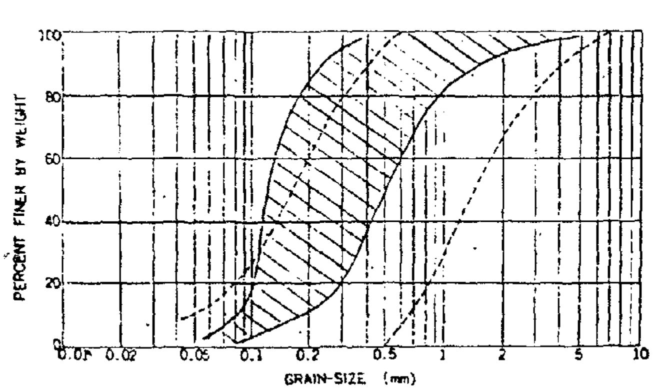


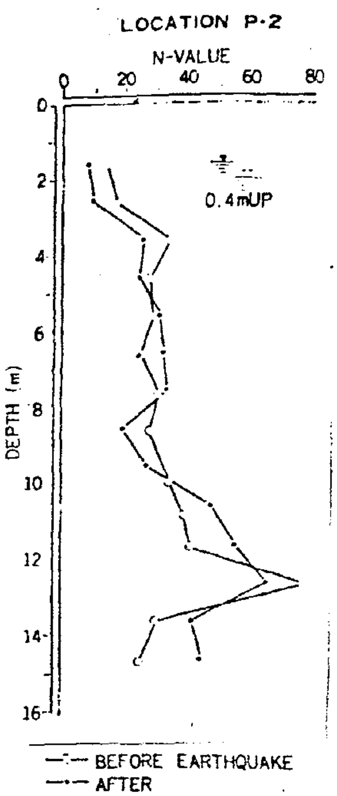
Fig. 6. Grain-Size Distribution of Loose Sands in the Site

Earthquake: Tokachi-Oki, Japan (1968)(E16)
Location: Hachinohe P-2

Site Condition (23):

"Site located in the wood at the central part of boring site where fairly dense sand had not been disturbed by excavation or backfilling."

"There occurred no liquefaction during the earthquake."



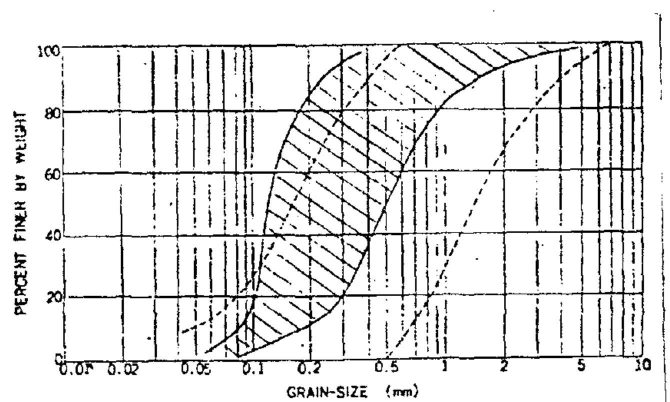


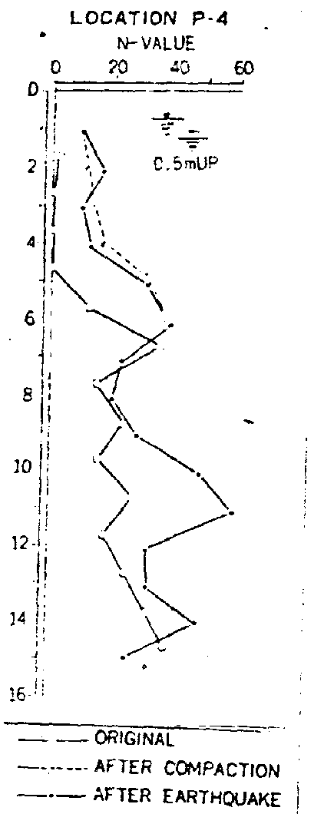
Fig. 6. Grain-Size Distribution of Loose Sands in the Site

No Liquefaction

Earthquake: Tokachi-Oki, Japan (1968)(E16)
Location: Hachinohe P-4

Site Condition (23):

"The loose sands down to a depth of 5 to 6 m, had been compacted by means of vibroflotation."



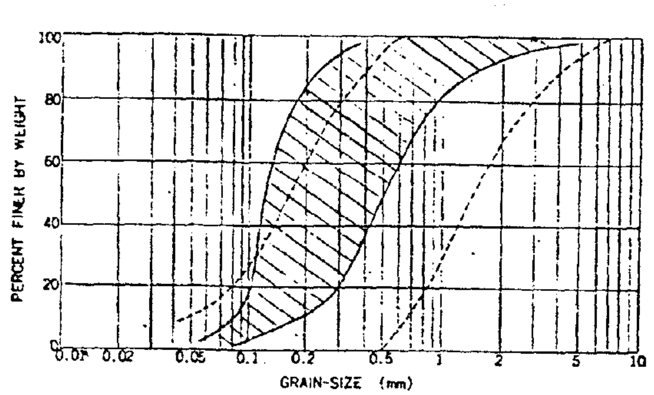


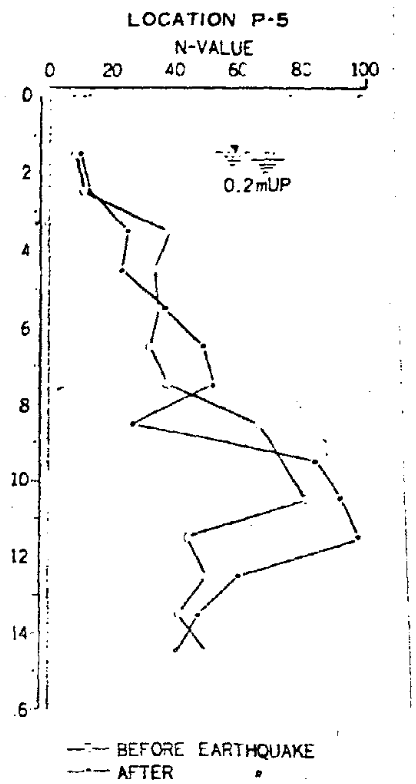
Fig. 6. Grain-Size Distribution of Loose Sands in the Site

Earthquake: Tokachi-Oki, Japan (1968)(E16)
Location: Hachinohe P-5

Site Condition (23):

"Located in the wood at the central part of the site where fairly dense sand had not been disturbed by excavation or backfilling."

"There occurred no liquefaction during the earthquake."



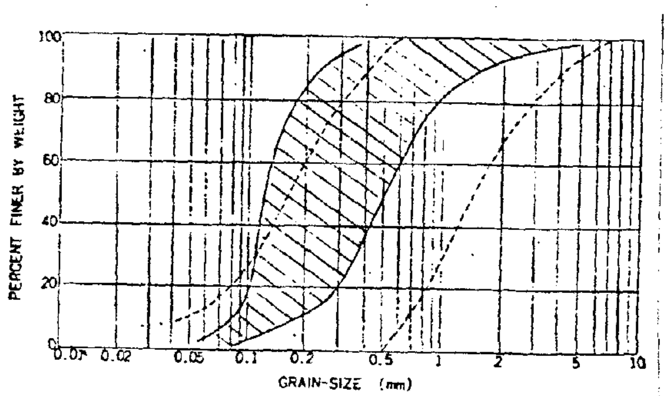


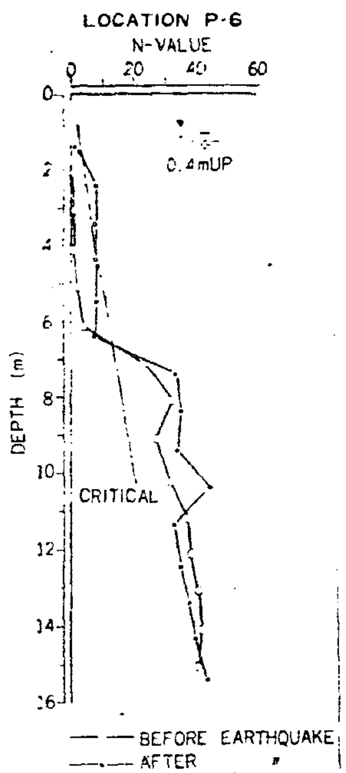
Fig. 6. Grain-Size Distribution of Loose Sands in the Site

Earthquake: Tokachi-Oki, Japan (1968)(E16)
Location: Hachinohe P-6

Site Condition (23):

"Flat sandy beach fronting on Pacific Ocean. Sands almost entirely down to a depth of more than 20 m. Top 5 m. had once been excavated and backfilled with waste sand—this was done all over the site."

"At this location the ground was apparently liquefied. N values of the SPT which had been extremely small before the quake increased to considerable values—indicating that the loose sands were liquefied, consolidated and as a result were compacted by the earthquake motion."



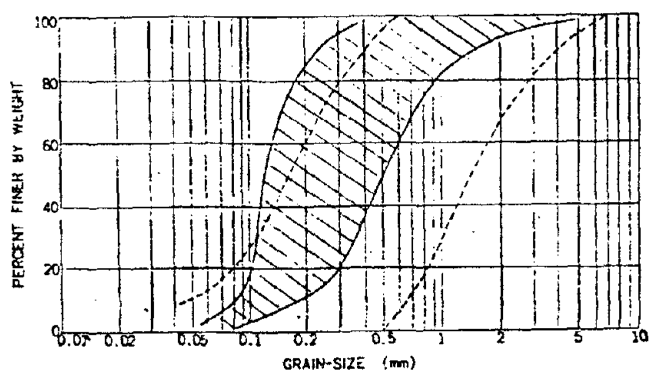


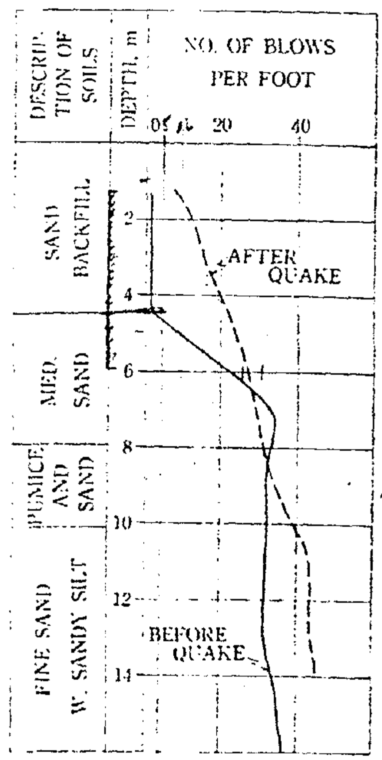
Fig. 6. Grain-Size Distribution of Loose Sands in the Site

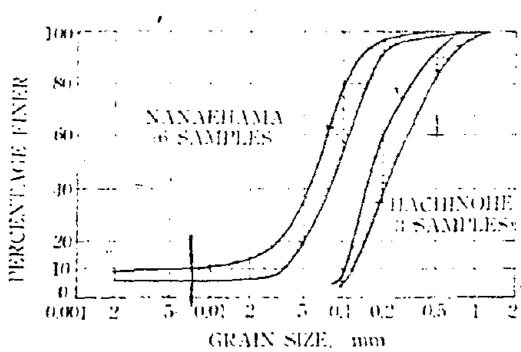
Earthquake: Tokachi-Oki, Japan (1968)(E16)
Location: Hachinohe Plant

Site Condition (46):

"Building site located on backfilled sand - prior to construction, loose sand was densified by vibroflotation-pile supported."

"Unimproved backfilled area underwent complete liquefaction causing many structures to tilt, settle or float up."





No Liquefaction

Earthquake: Tokachi-Oki, Japan (1968) (E16)
Location: Hachinohe Accelerometer

Site Condition (46):

"300 meters from shoreline and mounted on a hollow concrete block on 4 timber piles 4 m. long."

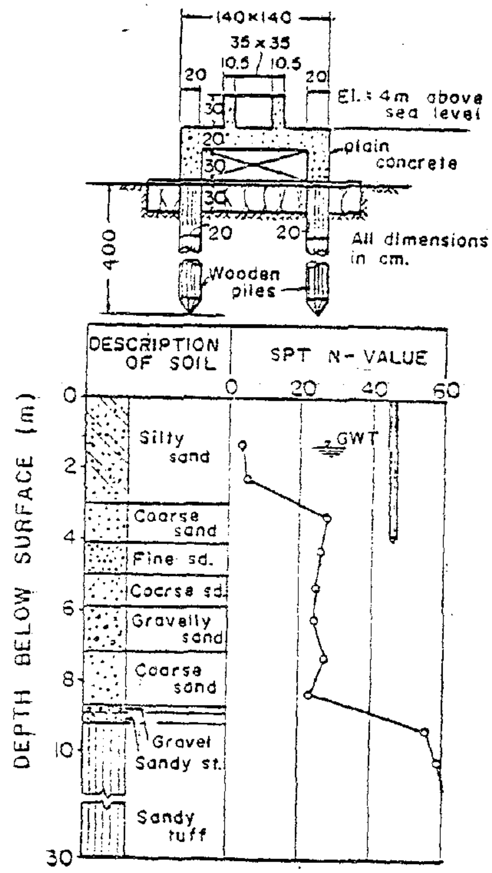


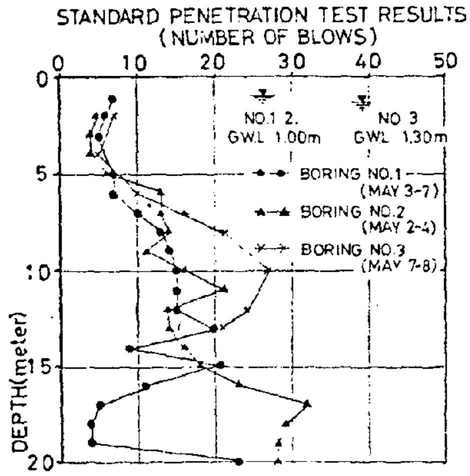
Fig. 5. Foundation and soil profile for the accelerometer at Hachinohe (Ref. 6)

Earthquake: Tokachi-Oki, Japan (1968)(E16)
Location: Nanaehama Beach Hakodate

Site Condition (46):

"Level sandy ground - beach faces Hakodate Bay and the liquefaction of the soil occurred at the place where reclamation was made by hydraulic fills ~3 years before quake."

"The water and the soils began to spout out from the ground surface soon after the beginning of the Earthquake and continued for about 1 hour. The reclaimed area was liquefied completely from the ground surface to some depth, soils were so soft that people had trouble walking up to week after quake. There were many sand volcanoes."



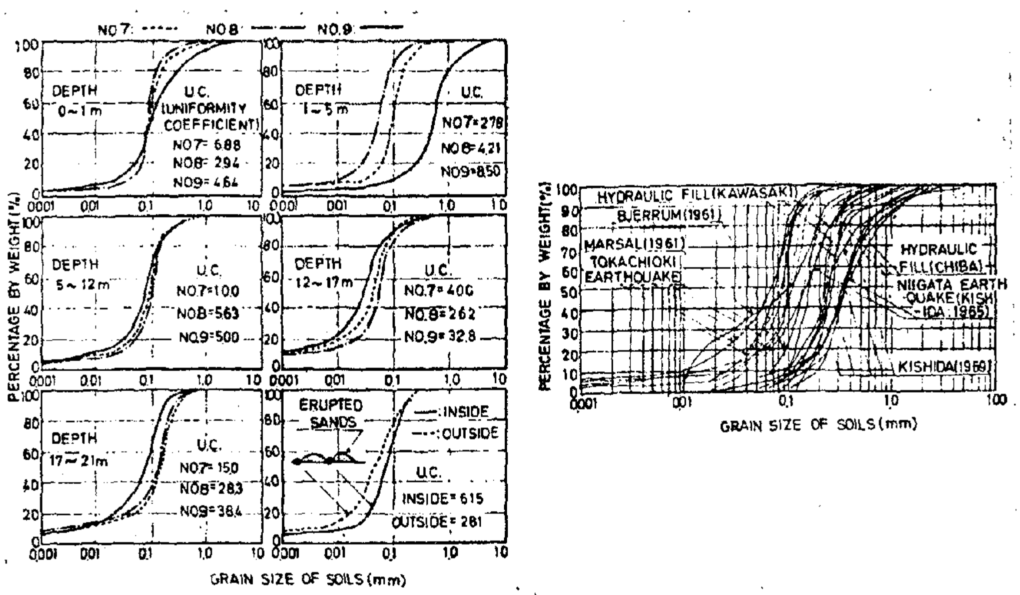
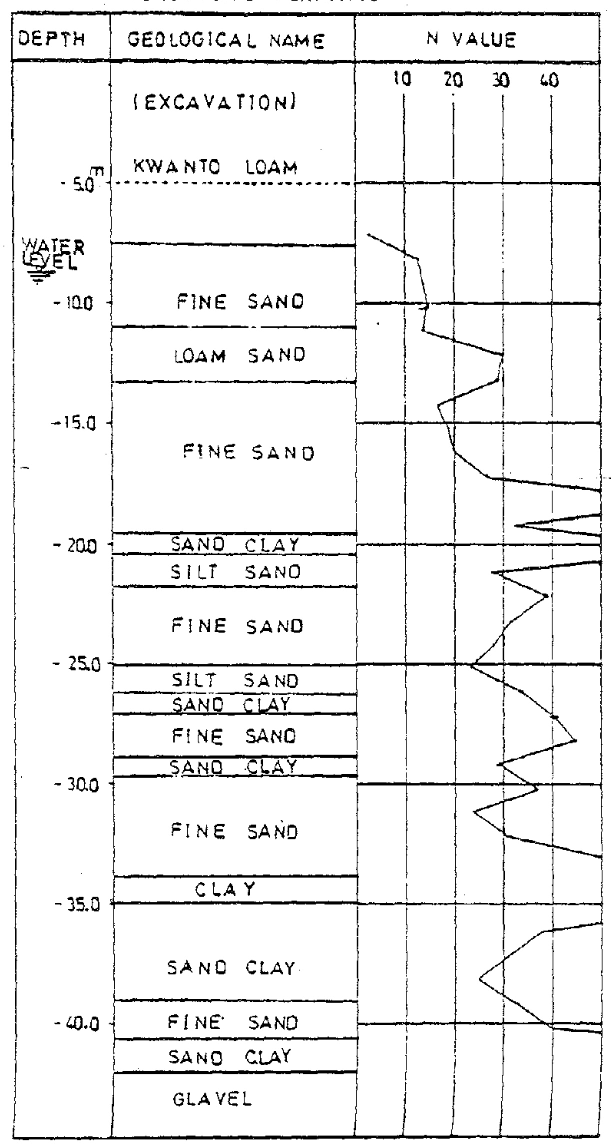


Fig. 5. Typical grain size distribution curves of soils

<u>No Liquefaction</u>

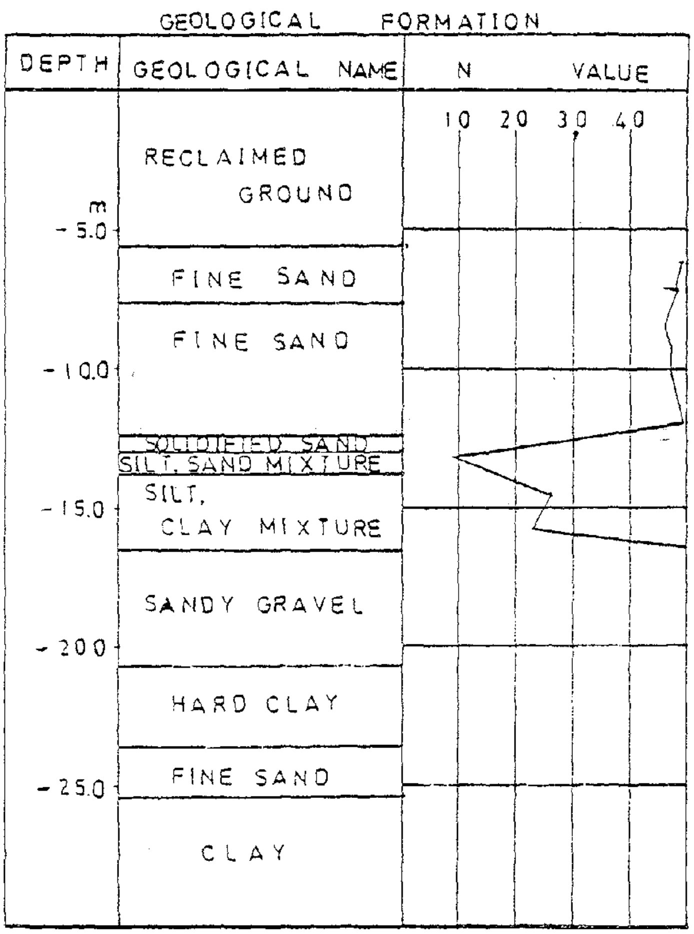
Earthquake: Saitama, Japan (1968)(E17)
Location: Site 101-2 (33)

GEOLOGICAL FORMATION



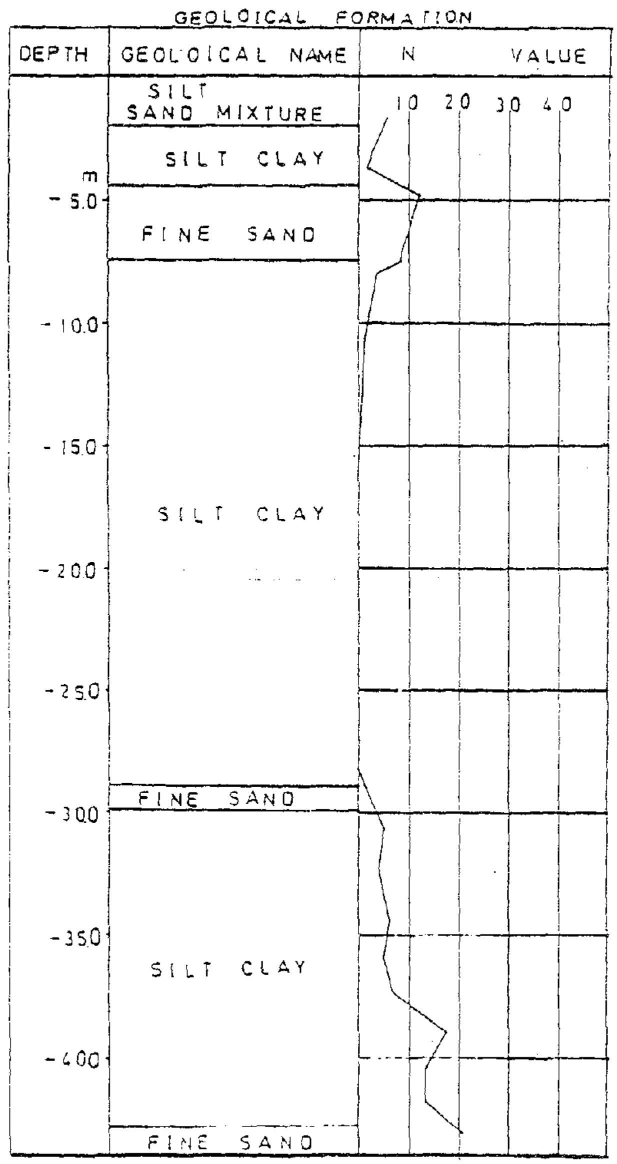
No Liquefaction

Earthquake: Saitama, Japan (1968)(E17)
Location: Site 105-2 (33)



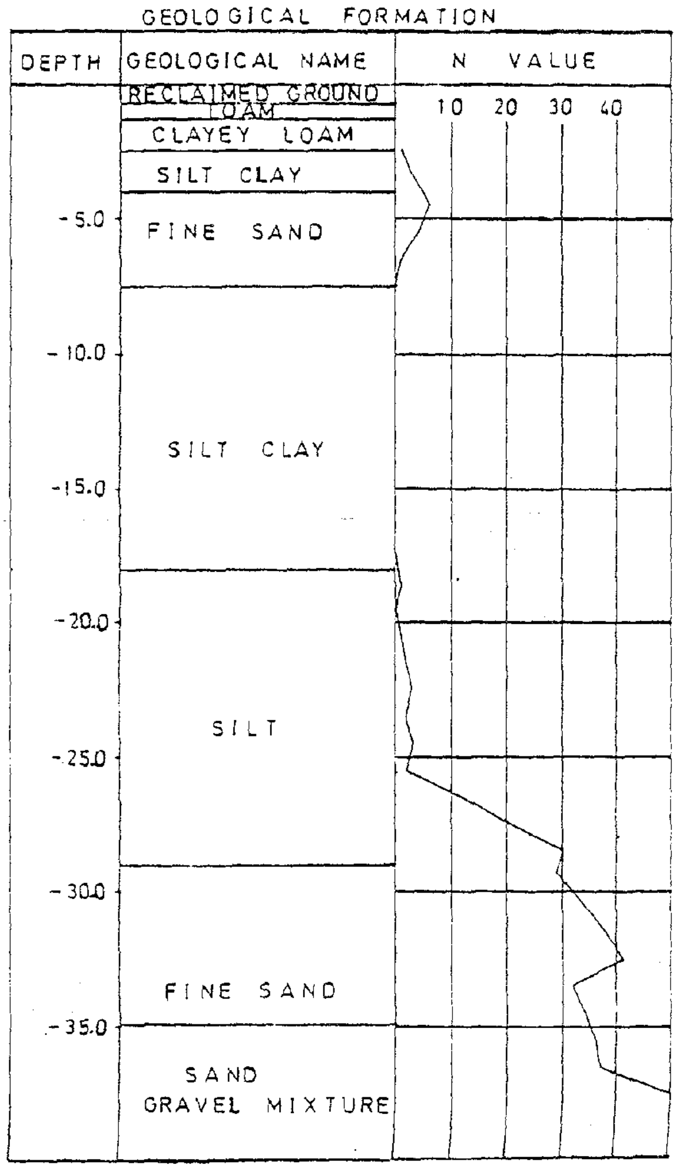
No Liquefaction

Earthquake: Saitama, Japan (1968)(E17)
Location: Site 119, (33)



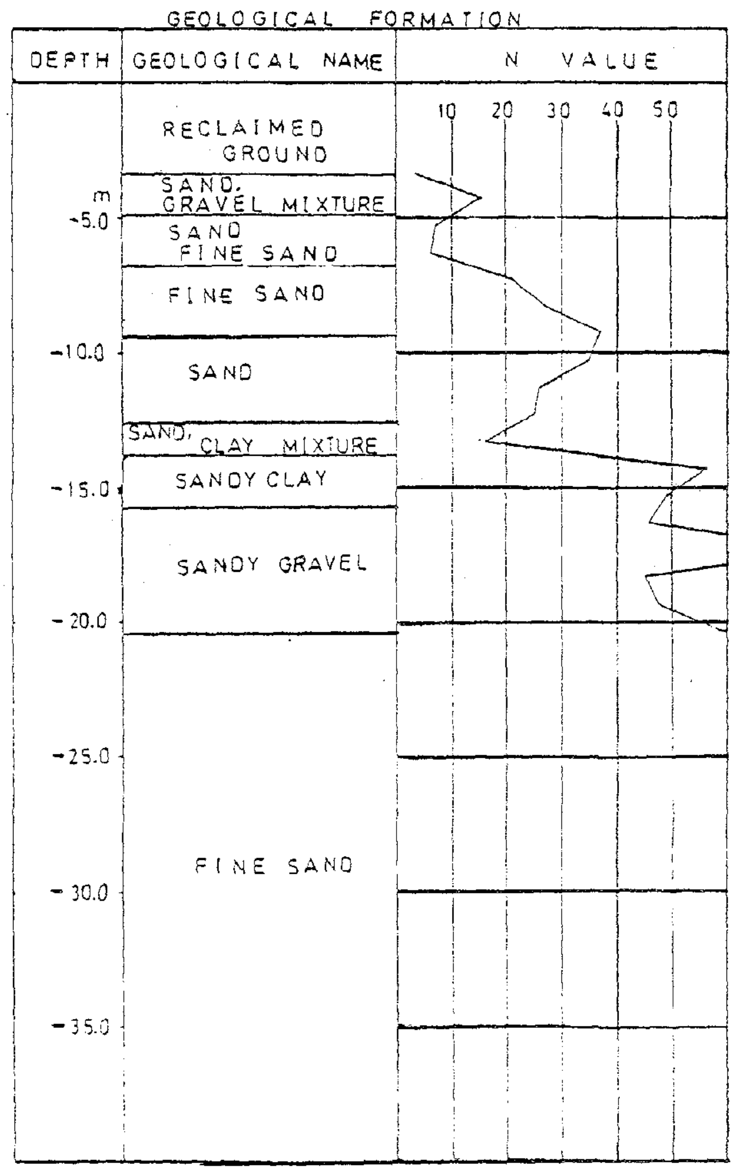
No Liquefaction

Earthquake: Saitama, Japan (1968)(E17)
Location: Site 121 (33)



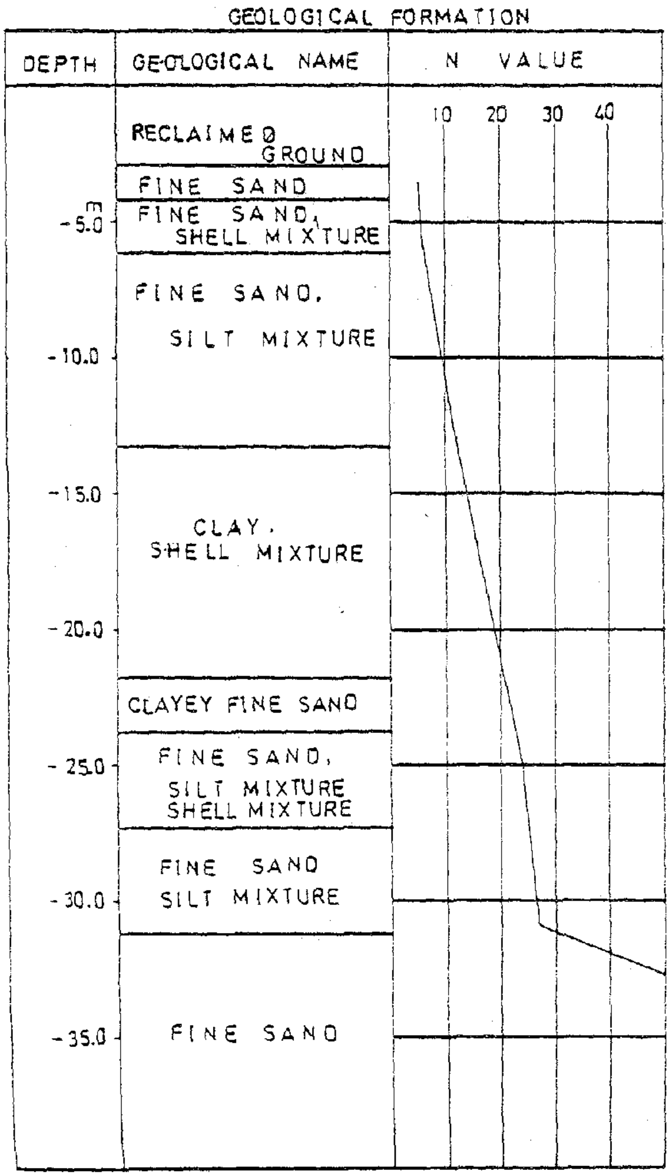
No Liquefaction

Earthquake: Saitama, Japan (1968)(E17)
Location: Site 130 (33)

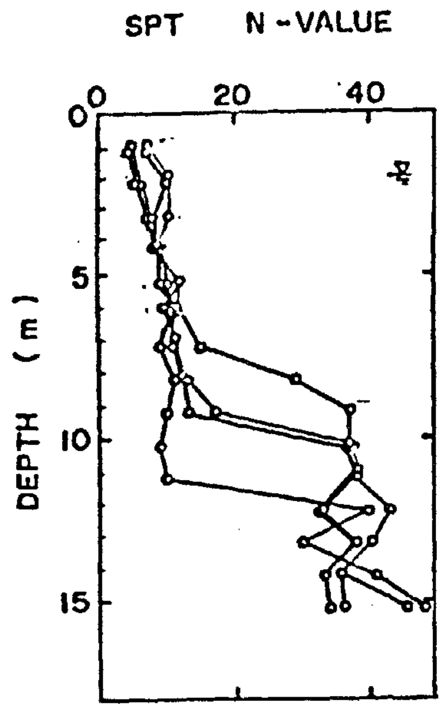


No Liquefaction

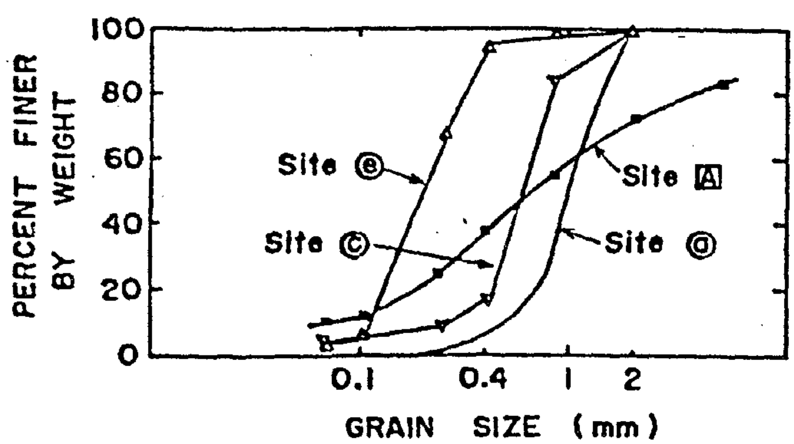
Earthquake: Saitama, Japan (1968)(E17)
Location: Site 602 (33)



Earthquake: Miyagiken-Oki, Japan (1978)(E21)
Location: Arahama, (Site a)(47)



Site ⓐ



Earthquake: Miyagiken-Oki, Japan (1978)(E21)
Location: Yuriage, (Site c)(47)