ESTIMATION PROCEDURE OF LIQUEFACTION POTENTIAL AND ITS APPLICATION TO EARTHQUAKE RESISTANT DESIGN

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SUMMARY

This paper presents two simplified methods, which authors, Iwasaki et al proposed in 1978, for assessing seismic liquefaction potential of sands. To prove the effectiveness of these methods many case studies are conducted at liquefied sites and non liquefied sites related to past earthquakes. Shaking table tests are also conducted to clarify the properties of soil liquefaction. Furthermore, a case study on earthquake resistant properties of river dykes carried out according to the proposed methods on soil liquefaction propertes.

INTRODUCTION

It is well recognized that numerous engineering structures have been severely damaged due to liquefaction of the supporting soils during earthquakes. The authors, Iwasaki et al proposed two simplified methods with use of a liquefaction resistance factor F_L and a liquefaction potential index I_L to evaluate the liquefaction potential of saturated sandy soils. In the methods, the liquefaction potential of sandy soils can be estimated from N-values of standard penetration tests, unit weights, mean particle diameters and the maximum acceleration at the ground surface.

In this paper, the two simplified methods are firstly introduced, and to prove the effectiveness of the proposed methods the values of both F_L and I_L at 64 liquefied sites and 23 non-liquefied sites during past six earthquakes are calculated according to the simplified methods. Also shaking table tests on soil liquefaction are carried out for the saturated sandy model ground. Furthermore, a relation between excessive pore water pressure induced in the saturated sandy soils and the facter F_L is illustrated based upon dynamic soil tests and shaking table tests. Lastly a case study on earthquake resistant proporties of river dykes considering soil liquefaction are carried out according to the proposed methods on soil liquefaction properties.

SIMPRIFIED METHODS

Liquefaction Resistance Factor (F_L)

An ability to resist the liquefaction of a soil element at an arbitary depth may be expressed by the liquefaction resistance factor (F_L) identified by Eq.(1).

$$F_L = R / L \quad \text{.....} \quad (1)$$

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When the factor F_L at a certain soil is less than 1.0, the soil is judged to liquefy during earthquakes.

R in Eq.(1) is the in-situ resistance or undrain cyclic strength of a soil element to dynamic loads during earthquakes, and can be simply evaluated according to numerous undrained cyclic shear test results using undisturbed specimens, as follows,

for 0.04 mm $\leq D_{50} \leq$ 0.6 mm

$$R = 0.0882\sqrt{\frac{N}{\sigma_v' + 0.7}} + 0.225 \log_{10}\frac{0.35}{D_{50}} \quad \text{..... (2a)}$$

for 0.6 mm $< D_{50} \leq$ 1.5 mm

$$R = 0.0882\sqrt{\frac{N}{\sigma_v' + 0.7}} - 0.05 \quad \text{..... (2b)}$$

where N is the number of blows of the standard penetration test, σ_v is the effective overburden pressure (in kgf/cm^2), and D_{50} is the mean particle diameter (in mm).

L in Eq.(1) is the dynamic load induced in the soil element by a seismic motion, and can be simply estimated by

$$L = \frac{\tau_{max}}{\sigma_v'} = \frac{\alpha_{s\ max}}{g} \cdot \frac{\sigma_v}{\sigma_v'} \cdot r_d \quad \text{..... (3)}$$

where τ_{max} is the maximum shear stress (in kgf/cm^2), $\alpha_{s\ max}$ is the maximum acceleration at the ground surface (in gals), g is the acceleration of gravity (=980 gals), σ_v is the total overburden pressure in (kgf/cm^2), and r_d is the reduction factor of dynamic shear stress to account for the deformation of the ground. From a number of seismic response analyses for grounds, Iwasaki et at (1978) proposed the following relation for the factor r_d.

$$r_d = 1.0 - 0.015Z \quad \text{..... (4)}$$

where z is the depth in meters.

Liquefaction Potential Index (I_L)

An ability to resist liquefaction at a given depth of grounds can be evaluated by the factor F_L. However it must be noticed that the damage to structures due to soil liquefaction is considerably affected by the severity of liquefaction degree. In view of this fact, Iwasaki et al (1978) also proposed the liquefaction potential index (I_L) defined by Eq.(5) to estimate the severity of liquefaction degree at a given site.

$$I_L = \int_0^{20} F \cdot W(Z)dZ \quad \text{..... (5)}$$

where $F=1-F_L$ for $F_L \leq 1.0$ and $F=0$ $F_L \geq 1.0$, and $W(Z) = 10-0.5Z$ for $Z \leq 20$ and $W(Z)=0$ for $Z\ 20<$(Z:depth in the ground in meters). W(Z) accounts for the degree of soil liquefaction according to the depth and the depth of 20 meters are decided considering the liquefaction phenomena during the past earthquakes. For the case of $F_L = 0$ for the entire depth, I_L becomes 100 being the highest, and for the case of $F_L \geq 1.0$ for the entire depth, I_L becomes 0 being the lowest.

CASE STUDIES ON F_L AND I_L FOR PAST EARTHQUAKES

Both the liquefaction resistance factor F_L and the liquefaction potential index I_L were calculated using the proposed methods for 64 liquefied sites and 23 non-liquefied sites where geotechnical informations are available during the following six earthquakes: the Nobi Earthquake of 1891 (Magnitude =8.0), the Tonankai Earthquake of 1944 (M=8.0), the Fukui Earthquake of 1948

(M=7.8), the Niigata Earthquake of 1964 (M=7.5), the Tokachi-oki Earthquake of 1968 (M=7.9) and the Miyagi-ken-oki Earthquake of 1978 (M=7.4) and according to these results the properties of Both F_L and I_L are investigated.

Characteristics of Factor F_L

Figs.1(A) and 1(B) show typical calculation results of F_L with depth at a liquefied site and a non-liquefied site, respectively. It can be seen that F_L is mostly less than 1.0 for the liquefied layers, and greater than 1.0 for the non-liquefied layers.

Fog.2 shows the frequency and the accumulative incidences of F_L-values calculated for both liquefied and non-liquefied layers at all sites. According to Fig.2, it is found that the distribution of F at the liquefied layers is very different from that at the non-liquefied layers. At the liquefied layers most (about 87%) of F_L-values distribute in the range less than 1.0, while at the non-liquefied layers most (about 85%) of F_L-values distribute in the range more than 1.0. However, it must be also noticed that about 13 % of $F_L$values exceed 1.0 at the liquefied layers and about 15% of F_L-values are less than 1.0 at the non-liquefied layers.

Characteristics of Index I_L

Fig.3 summarizes both the relation between the number of cases and I_L, and the relation between the accumulative percentages and I_L, at all liquefied and non-liquefied sites. It is found from this figure that I_L for liquefied sites seems to be higher than that at non-liquefied sites, i.e., for non-liquefied sites I_L is mostly less than 15 and the percentage that I_L is less than 5 is aboyt 70%, and on the other hand for liquefied sites the percentage that I_L is less than 5 is only about 20% and at about 50% of the sites I_L is more than 15. From these results, the following simplified procedure for assessing soil liquefaction based on the index I_L may be proposed as a preliminary guideline.

$I_L = 0$: Liquefaction risk is very low,
$0 < I_L \leq 5$: Liquefaction risk is low,
$5 < I_L \leq 15$: Liquefaction risk is high,
$15 < I_L$: Liquefaction risk is very high.

As mentioned in the above, it is shown that the index I_L may be resonably used to assess the liquefaction potential at a certain site.

SHAKING TABLE TESTS ON SOIL LIQUEFACTION

Shaking table tests were carried out to clarify the properties of soil liquefaction and the effectiveness of the proposed liquefaction resistance factor F_L. A loose saturated sand ground model with 0.95m deep, 6m long and 3m wide was prespared in a steel container on a shaiking table. In the tests, the table was shaked in the sinusoidal motion with a constant frequency of 7Hz, and the input table accelerations ranged from 30 gals to 250 gals.

Figs.4(A) and 4(B) show the typical relationships between ground accelerations, pore water pressures and F_L-values for non-liquefied cases and liquefied cases, respectively. In these figures, F_L-value are estimated by Eqs.(2) and (3). It is found that F_L-values decrease according to the increase of pore water pressures, and that F_L-values are less than 1.0 for the liquefied layers and are higher than 1.0 for the non-liquefied layers.

Hereupon the degree of soil liquefaction is defined by the factor L_u,

$$L_u = \Delta u/\sigma_v' \quad \text{.....} \quad (6)$$

where Δu is the excessive pore water pressure. Sands with L_u of 1.0 are assumed to completely liquefy. Fig.5 summarizes the relation between F_L and L_u for the liquefied layers. From this figure, it is seen that F_L decrease as L_u increases and that F_L is less than 1.0 for L_u of 0.5 or higher and is more than 1.0 for L_u of 0.5 or lower. Furthermore, it is clarified that the sand layers are likely to completely liquefy when F_L decreases to less than 0.6.

From these shaking table tests, it is clarified that the proposed factoer F_L may be adequately used to estimate soil liquefaction potential of saturated sand layers.

RELATION BEWEEN PORE WATER PRESSURE AND F_L

Excessive pore water pressures generated in sand layers is very important for soil liquefaction studies. In this paragraph the simplified procedures for evaluating excessive pore water pressure using a F_L-value are introduced.

From dynamic triaxial tests on cyclic strength for soil liquefaction, a typical relation between the shear stress ratio τ/σ_v' (τ:shear stress) and the number of cycles N_l to generate liquefaction is shown in Fig. 6, and the relation is approximately given by Eq.(7),

$$(\tau/\sigma_v') = aN_l^b \quad \text{.....} \quad (7)$$

where constant values, a and b are decided basing on the dynamic triaxial tests as shown in Fig.6. If liquefaction assumes to occur for the cyclic strength R with the number of cycles N_R and for the dynamic load L with the number of cycles N_L, the relations on both R and L may be concluded in Eq.(8).

$$R = (\tau_R/\sigma_v') = aN_R^b,\ L = (\tau_L/\sigma_v') = aN_L^b \quad \text{.....} \quad (8)$$

where τ_L and τ_L are the shear strength and the shear load, respectively. From Eqs.(1) and (8), the following relation is obtained.

$$F_L = (N_R/N_L)^b \quad \text{.....} \quad (9)$$

On the other hand, the relations between $\Delta u/\sigma_v'$ and N/N_l(N, N_l : number of cycles before liquefaction and that at complete liquefaction, respectively) are obtained, for example, as Fig.7. Hereupon because N_R and N_L are regarded as N and N_l, respectively, the following relation can be assumed.

$$(N_R/N_L) = (N/N_l) \quad \text{.....} \quad (10)$$

From Eqs.(9) and (10), Eq.(11) can be concluded.

$$(N/N_l) = (F_L)^{1/b} \quad \text{.....} \quad (11)$$

Therefore, the pore water pressure can be estimated by the factor F_L as follows, according to the test results shown in Fig.7.

$$(\Delta u/\sigma_v') \sim (N/N_l) = (F_L)^{1/b} \quad \text{.....} \quad (12)$$

Pore water pressures can be estimated by the factor F_L basing on shaking table tests, i.e., by using the relation shown in Fig.5. Fig.8 summarizes a relation between pore water pressure and F_L according to the proposed methods, i.e., dynamic soil tests and shaking table tests. From this figure, pore water pressure can be simply evaluated by F_L-values.

CASE STUDY ON EARTHQUAKE RESISTANT PROPERTIES FOR RIVER DYKES

The Yuriage-kami River Dyke was damaged during the Miyagi-ken-oki Earthquake in 1978. The river dyke near the one point Y-1 was damaged severely and on the other hand at the other points Y-3 and Y-4, the river dyke was not damaged. In this paragraph the analytical results on the stability for both the damaged river dyke and the non-damaged river dyke considering the excessive pore water pressure induced by the earthquake are introduced.

The excessive pore water pressure is estimated simply from F_L-values by using the equation (12). In this example, the F_L-values are calculated in detail, i.e., the in-situ resistance R and the dynamic load L in the equation (1) are estimated from the dynamic triaxial tests and the finite element analyses, respectively.

Fig.9 shows the distribution on F_L-values in the sandy soils calculated at both the damaged river dyke and the non-danaged one. Comparing the results for both sites, it can be seen that the area whose F_L-values are less than 1.0 at the damage river dyke is larger than that at the non-dameged one.

Fig.10 shows the distribution of the excessive pore water pressure (Δu) calculated based on the results in Fig.9 at both the damaged river dyke and the non-damaged one. The magnitude of the excessive pore water pressure at the damaged river dyke seems to be larger than that at the non-damaged one.

The analyses on the stability of the river dykes are conducted by using Friction Circle Method to get the minimum safety factor F_S. The inputted horizontal seismic coefficients (k_h) are 0.0 (i.e., before earthquakes), 0.15 and 0.2. Furthermore the effects of soil liquefaction, i.e., the excessive pore water pressure calculated in Fig.10, to the stability of river dykes are also investigated. Fig.11 summarizes the relationship between the minimum safety factor F_S and the horizontal seismic coefficient considering the excessive pore water pressure Δu. It is found that the factor F_S considering the occurrence of excessive pore water pressure decreases and the factors F_S at the damaged river dyke are less than the ones at the non-damaged river dyke under considering the excessive pore water pressure.

As the above-mentioned, It is also confirmed that the excessive pore water pressure induced during earthquakes is one of very important factors for the stability of soil structures.

CONCLUSIONS

Two simplified methods based on the liquefaction resistance factor F_L and the liquefaction potential index I_L are proposed to assess the liquefaction potential. From the studies it is found that F_L-value is mostly less than 1.0 for liquefied layers and greater than 1.0 for non-liquefied layers, and a very reasonable factor to estimate the soil liquefaction for a certain layer. It is also found that I_L-value at liquefied sites differs noticeably from those at non-liquefied sites and seems to be a reasonable index to assess the liquefaction potential at a certain site.

From the experimental tests, it is also shown that the effects of liquefaction can be reasonably assessed by F_L-values. The importance of the effects of soil liquefaction during earthquakes to the earthquke response properties of river dykes is also clarified.

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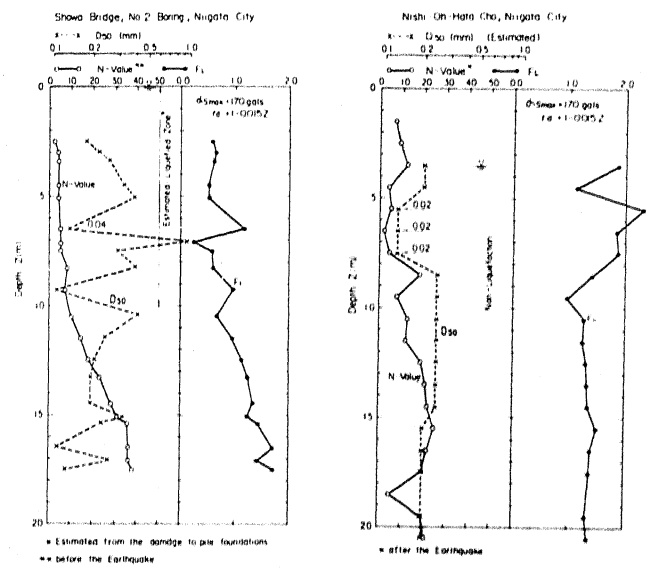


Fig.1 Relationships between F_L and Depth

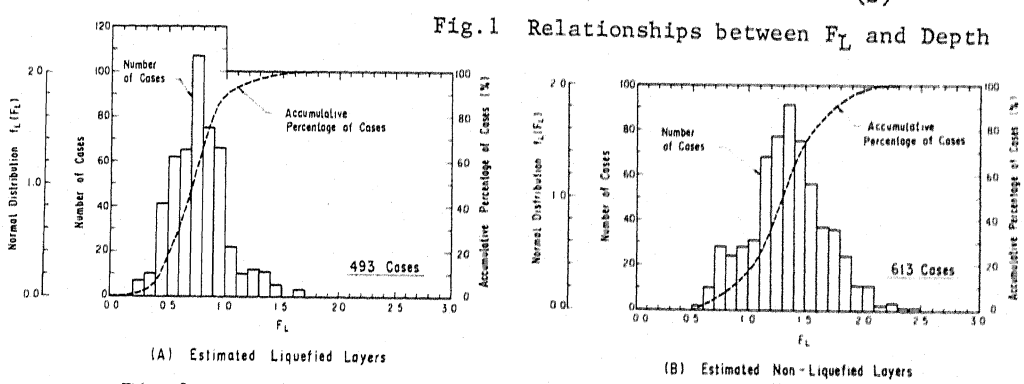


Fig.2 Distribution of F_L Values Comparing Liquefied Sites with Non-Liquefied Sites

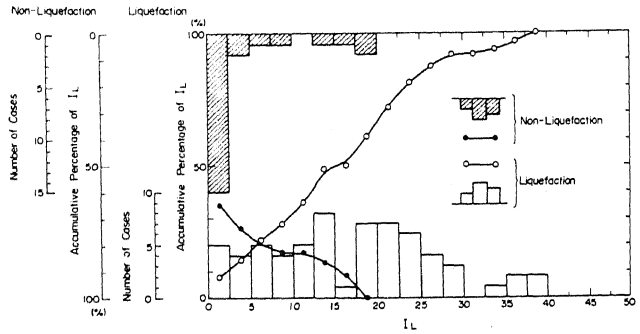


Fig.3 Distribution of I_L Values Comparing Liquefied Sites with Non-Liquefied Sites

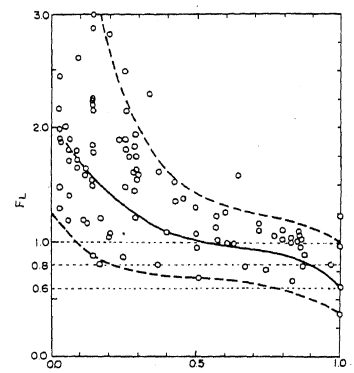
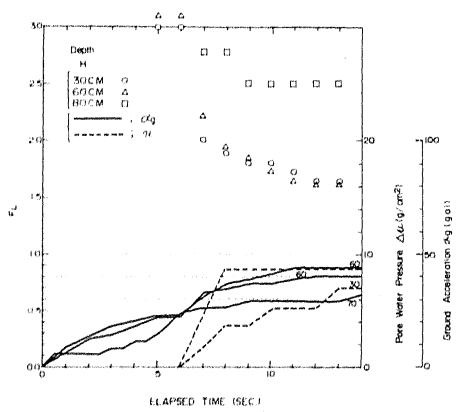
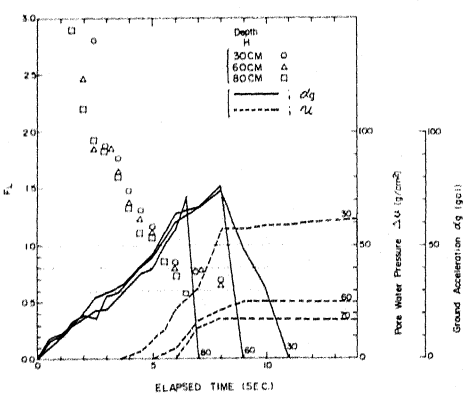


Fig.5 Relationships between F_L and $\Delta u/\sigma'_v$ in Shaking Table Tests



(A) Non-Liquefied Case



(B) Liquefied Case

Fig.4
Relationships between Pore Water Pressure and F_L Values in Shaking Table Tests

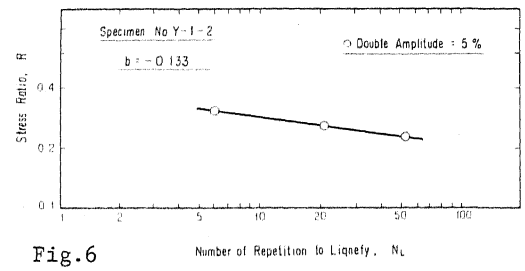


Fig.6
An Example of Relationship between R and N_L (Dynamic Triaxial Test)

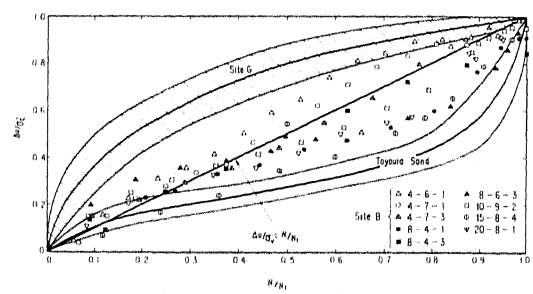


Fig.7
Distribution of I_L Values and Liquefied Sites with Non-Liquefied Sites

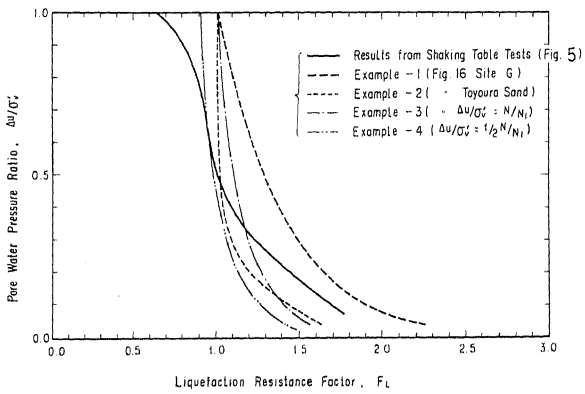


Fig.8 Relationships between $\Delta u/\sigma_v'$ and F_L

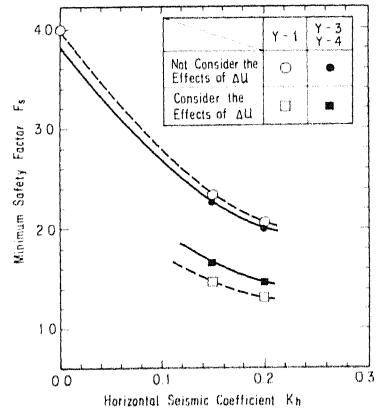


Fig.11 Stability of River Dykes Considering Excessive Pore Water Pressure Occurred During Earthquakes

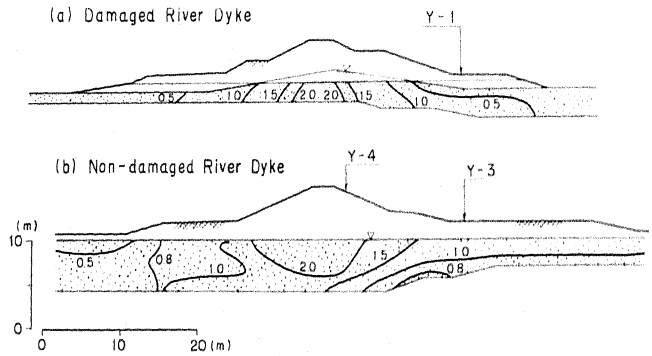


Fig. 9 Distribution of F_L-values Comparing Liquefied River Dyke with Non-Liquefied River Dyke

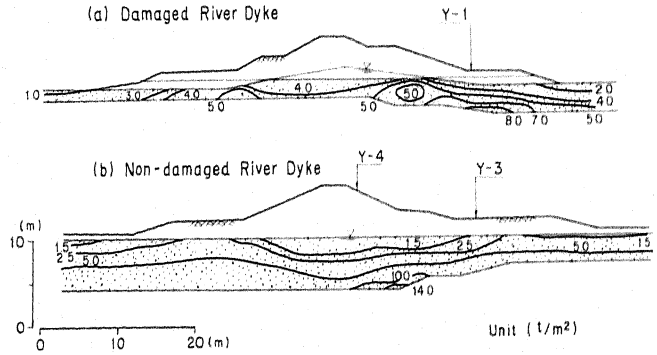


Fig.10 Distribution of Excessive Pore Water Pressure Comparing Liquefied River Dyke with Non-Liquefied River Dyke