SOIL LIQUEFACTION EVALUATION WITH USE OF STANDARD PENETRATION RESISTANCES

by

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SUMMARY

For assessing liquefaction potential the authors propose a simplified method in which a liquefaction resistance factor, F_L, at a given depth of a site and a liquefaction potential factor, P_L, at the site are computed from the conventional soils data. To prove the effectiveness of this method, case studies were conducted at 63 liquefied sites caused by past various earthquakes and at 22 non-liquefied sites. Based on the case studies, the method appears to be valid.

INTRODUCTION

Recently, several methods to evaluate the liquefaction potential of sandy soil due to earthquake motions have been proposed, based on the results of studies done during the past fifteen years. The complexity of these methods varies. In Japan, complex methods to evaluate liquefaction potential from special soil investigations and tests, undisturbed soil samplings, cyclic undrained triaxial tests, etc., and from seismic response analyses, are becoming popular. However, in many cases, the complex methods cannot be used because they require considerable cost and time. Therefore, *in 1978, the authors, Tokida and Yasuda, and Iwasaki and Tatsuoka, proposed* a new simplified method by which the liquefaction potential of sandy soil can be estimated from its N-values, its unit weights, its mean particle diameters, and the maximum acceleration at the ground surface. This simplified method was derived from the results of cyclic undrained triaxial tests on undisturbed samples and seismic response analyses.

The simplified method is herein described and its effectiveness is tested by examining the results of numerous case studies.

OUTLINE OF THE SIMPLIFIED METHOD

In the method, an ability of a soil element at an arbitrary depth to resist liquefaction can be expressed by the factor of liquefaction resistance, F_L, as follows:

$$F_L = \frac{R}{L} \tag{1}$$

where R is the in-situ resistance (or undrained cyclic strength) of a soil element to dynamic loads and can be simply evaluated, based on undrained

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cyclic shear test results, as follows:

$$R = 0.0882\sqrt{\frac{N}{\sigma_v' + 0.7}} + 0.225 \log_{10}\frac{0.35}{D_{50}}, \text{ for } 0.04 \text{ mm} \leqq D_{50} \leqq 0.6 \text{ mm} \qquad 2(a)$$

$$R = 0.0882\sqrt{\frac{N}{\sigma_v' + 0.7}} - 0.05, \text{ for } 0.6 \text{ mm} \leqq D_{50} \leqq 1.5 \text{ mm} \qquad 2(b)$$

where N is the number of blows in 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 estimated by

$$L = \frac{\tau_{max}}{\sigma_v'} = \frac{\alpha_{s_{max}}}{g} \frac{\sigma_v}{\sigma_v'} r_d \qquad (3)$$

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, the reduction factor for dynamic shear stress, indicates the deformation of the ground. In 1971, Seed and Idriss proposed a relationship between r_d and depth. However, in this paper, the relationship,

$$r_d = 1 - 0.015\ Z \qquad (4)$$

where Z is the depth in meters, is used.

It is obvious that the damage to foundations due to soil liquefaction is considerably affected by the severity of liquefaction. As only the ability to resist liquefaction at a given depth can be evaluated by F_L, an index of liquefaction potential, P_L, can be introduced to express the severity of liquefaction as,

$$P_L = \int_0^{20} F \cdot W\ (Z)\ dZ \qquad (5)$$

in which $F = 1 - F_L$ for $F_L \leqq 1.0$ and $F = 0$ for $F_L > 1.0$ as illustrated in Fig. 1(a), and $W\ (Z) = 10 - 0.5Z$ (Z in meters), as illustrated in Fig. 1(b). For the case of $F_L = 0.0$ for the entire range from Z = 0 to Z = 20 m, P_L becomes 100, and for the case of $F_L \geqq 1.0$ for the entire range from Z = 0 to Z = 20 m, P_L becomes 0.0.

CASE STUDIES OF THE NIIGATA EARTHQUAKE AND OTHERS

As is well known, the Niigata Earthquake of 1964 induced severe liquefaction at many sites, and several structures were damaged due to liquefaction. Liquefied sites were exactly located, and, at several sites, the extent of liquefaction could be estimated from the damage to foundations and from the existence of boiled sands. Geotechnical information about these sites was available. Therefore, the liquefaction potentials at several sites in Niigata City were first analysed (Iwasaki, et al., 1978) by the method proposed.

Figs. 2 and 3 compare the variation of F_L with depth at a site where liquefaction was observed with its variation at a site where liquefaction was not observed, respectively. It can be seen that F_L is, in general, less than 1.0 in the liquefied zones, and greater than 1.0 in the non-liquefied zones. The tendancy of F_L at other liquefied and non-liquefied sites was similar.

P_L was then calculated for sites where liquefaction occurred and also for sites where it did not during each of the following earthquakes: the Niigata Earthquake, the Tokachi-oki Earthquake (1968), the Fukui Earthquake (1948), The Tonankai Earthquake (1944) and the Nobi Earthquake (1891), and the values were combined to form Fig. 4. Calculations were made for 45 liquefied sites and for 10 non-liquefied sites, of which, 31 liquefied sites and 9 non-liquefied sites were from the Niigata Earthquake. Fig. 4 shows the distribution of P_L values at both kinds of sites. All the values of P_L at non-liquefied sites are less than 20, and at seventy percent of such sites, the values of P_L are less than 5. On the other hand, the values of P_L at eighty percent of the liquefied sites are larger than 5 and at fifty percent of such sites, the values of P_L are larger than 15. Therefore, it is likely that severe liquefaction will not occur at sites with P_L values of less than 5, and moderate or severe liquefaction can be expected at sites with P_L values larger than 15 or 20.

CASE STUDY OF THE MIYAGI-KEN-OKI EARTHQUAKE

Liquefaction was observed at about 30 sites during the Miyagi-ken-oki Earthquake (Iwasaki and Tokida, 1980), and several river dikes, bridges, and port facilities were damaged due to liquefaction. The values of F_L and P_L at 18 liquefied sites and at 12 non-liquefied sites where soil investigations had been conducted were calculated. All non-liquefied sites selected were close to liquefied sites, and the soil at all sites was predominantly sandy. The values of P_L and geotechnical data gathered at the sites are summarized in Table 1. The mean particle diameter, D_{50}, and/or unit weight, γ_t, were not measured at several sites, though the N-values were measured. For these sites, as indicated in the footnote of Table 1, D_{50} and γ_t were assumed to be equal to the average values shown in Table 2. The maximum accelerations at the ground surface, $\alpha_{s_{max}}$, were estimated from epicentral distances, Δ, using the mean relationship between Δ and $\alpha_{s_{max}}$ shown in Fig. 5, which was determined from records of measured acceleration during the Miyagi-ken-oki Earthquake (Kuribayashi and Iwasaki, 1980).

Figs. 6 and 7 show the variation of F_L with depth at a site where liquefaction occurred and where it did not, respectively. The distance between the sites was only 69 m, and both sites were located at the base of the same river dyke, the right-bank dyke of the Notori River. Most of the calculated values of F_L at the liquefied site are less than 1.0 from the ground water table to about 7 m in depth. On the contrary, the calculated values of F_L at the non-liquefied site are generally larger than 1.0. This tendancy was observed at other liquefied and non-liquefied sites and reinforces the conclusions based on the results of the previous case studies.

Fig 8 shows the cumulative incidences, in percentages, of P_L values at liquefied sites and at non-liquefied sites and the distributions of P_L values at both kinds of sites. By comparing Fig. 8 with Fig. 4, it is found that the values of P_L at liquefied sites in Fig. 8 are slightly lower, on the average, than those in Fig. 4. This is attributed to a difference in the severity of liquefaction: liquefaction caused by the Niigata Earthquake was far severer than that caused by the Miyagi-ken-oki Earthquake. The values of P_L at non-liquefied sites in Fig. 8 seem to be almost the same as those in Fig. 4, if allowance is made for the rather fortuitous nature of selecting non-liquefied sites as closely as possible to liquefaction sites.

Fig. 9 combines the data in Figs. 4 and 8 and shows the cumulative incidence, in percentages, of all values of P_L and their distributions. Again, it can be concluded from Fig. 9 that moderate or severe liquefaction is probable at sites with P_L values larger than 15, and that severe liquefaction is not probable at sites with P_L values of 5 or less.

CONCLUSIONS

The liquefaction resistance factor, F_L, and the liquefaction potential factor, P_L, at 63 liquefied sites and 22 non-liquefied sites during the Niigata Earthquake, the Tokachi-oki Earthquake, the Nobi Earthquake, the Tonankai Earthquake, the Fukui Earthquake, and the Miyagi-ken-oki Earthquake were calculated following the simplified procedure proposed by Iwasaki, Tatsuoka, Tokida, and Yasuda. From these case studies, it was found that most values of F_L are less than 1.0 at liquefied zones, and are larger than 1.0 at non-liquefied zones. Further, the values of P_L and their incidences at liquefied sites vary significantly from the values and incidences at non-liquefied sites. Therefore, by determining the values of both F_L and P_L at a site, the liquefaction potential at that site can be predicted quite accurately. Furthermore, as the value of P_L increases, the liquefaction potential also increases.

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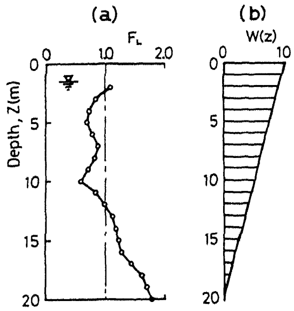

Fig. 1 Integration of F_L

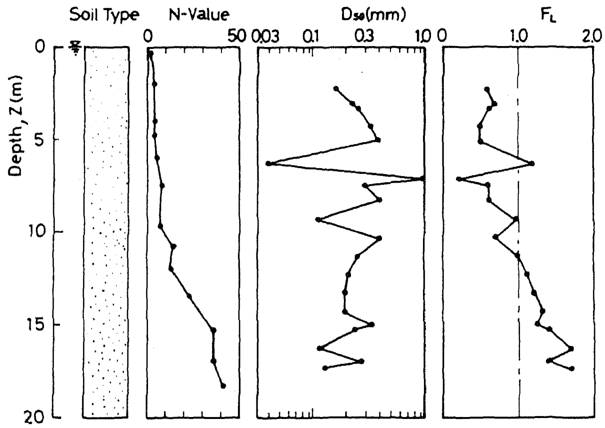

Fig. 2 F_L at a Liquefied Site, Showa Bridge, Br. No. 2, Niigata City (Iwasaki, et al., 1978)

Legend of Figs. 2, 3, 6, and 7

- Sand
- Sandy Silt
- Gravelly Sand
- Peat

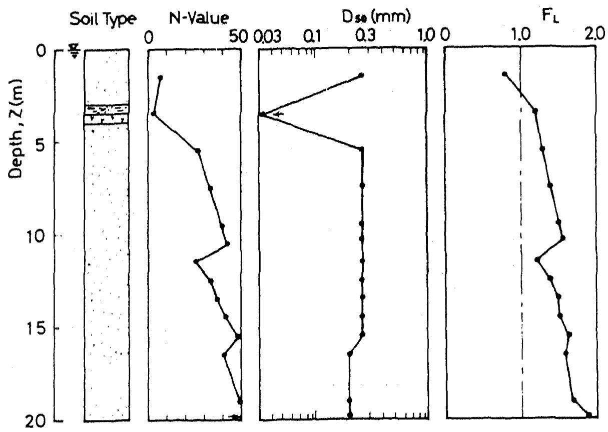

Fig. 3 F_L at a Non-liquefied Site, Showa Bridge, Br. No. 4, Niigata City (Iwasaki, et al., 1978)

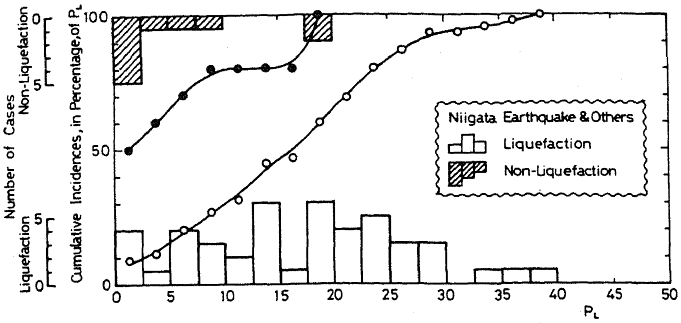

Fig. 4 Distribution of P_L Values and Their Cumulative Incidences, in percentage, at Sites of Liquefaction and of Non-Liquefaction (Iwasaki, et al., 1978)

Table 1 Geotechnical Data at Sites of Liquefaction and at Sites of Non-Liquefaction (Miyagi-ken-oki Earthquake)

Lique-faction	Site	*Soil Data	Depth of Water Table	**D_{50}	Major Soil Type	$\alpha_{s\,max}$ (gal)	P_L
Lique-faction	Abukuma River, Abukuma Bridge, Dr-4	A	0.00	M	Coarse Sand	175	12.2
	Mouth of Abukuma River	B	0.00	E	"	180	20.4
	Natori River, Yuriage-kami, Y-1	A	1.82	M	Silt to Coarse Sand	180	10.3
	" Y-2	A	0.85	M	Fine to Coarse Sand	180	9.4
	Natori River, Yuriage Bridge, No. 1	A	1.70	M	Fine to Med. Sand	185	7.0
	" No. 2	A	1.30	M	Med. to Coarse Sand	185	0.5
	" No. 3	A	0.26	M	"	185	21.8
	Yoshida River, Yamazaki, 15.9 km	A	0.87	E	Sandy Silt to Med. Sand	190	13.0
	Old Kitakami River, Oiri (1), No. 1	A	4.30	M	Clay to Med. Sand	210	4.1
	" No. 2	A	2.40	M	"	210	4.1
	Ishinomaki Port, Uomachi, B-1	B	0.00	E	Med. to Silty Sand	230	39.1
	" B-2	B	0.00	E	"	230	36.5
	Rihu, No. 12	B	2.72	E	Clay to Coarse Sand	185	20.4
	Ishinomaki Port, Shiomi, No. 1	B	0.00	E	Fine to Med. Sand	225	14.1
	" No. 2	B	0.00	E	Fine Sand	225	27.1
	" No. 3	B	0.00	E	"	225	18.2
	Natori River, Nakamura, N-4	A	0.50	M	Sandy Silt to Med.Sand	180	12.3
	" N-5	A	1.30	M	Fine to Coarse Sand	180	5.6
Non-Lique-faction	Natori River, Nakamura, N-1	A	0.85	M	Fine to Med. Sand	180	0.3
	" N-2	A	0.90	M	Silty to Coarse Sand	180	1.0
	Natori River, Yuriage-kami, Y-3	A	2.15	M	"	180	0.9
	Mouth of Kitakami River, No. 10	A	1.55	E	Fine Sand	230	1.5
	Natori River, 3.2 km	A	2.50	E	Clay to Coarse Sand	180	0.8
	Kitakami River, Kinnou Bridge, P_8	A	4.00	M	Silt to Silty Sand	195	0.6
	Abukuma River, Abukuma Bridge, Br. 1	A	4.30	M	Med. to Coarse Sand	175	0.0
	" Br. 2	A	3.40	M	"	175	0.7
	Eai River, Eai Bridge, No. 1	A	8.00	E	Fine Sand to Clay	175	0.0
	Minami Sendai, No. 2	B	0.80	E	Sandy Silt to Gravel	180	0.0
	***Ishinomaki Port, Uomachi, A-1	B	0.00	E	Med. to Silty Sand	230	13.9
	*** " A-2	B	0.00	E	"	230	17.1

Note: *B: Soil Data before Earthquake A: Soil Data after Earthquake
**E: Estimated by Using Table 2 M: Measured
***: Compacted by Sand Compaction Piles, near the Ishinomaki Port, Uomachi, B-1, B-2

Table 2 Average Values of the Unit Weights and Mean Particle Diameters of Different Type of Soil (This table was used only when these values were not tested)

Soil Type	Unit Weight, γ_t(t/m^3)	Mean Particle Diameter, D_{50} (mm)
Surface Soil	1.7	0.02
Silt	1.75	0.025
Sandy Silt	1.8	0.04
Silty Sand	1.8	0.07
Very Fine Sand	1.85	0.1
Fine Sand	1.75	0.15
Medium Sand	2.0	0.35
Coarse Sand	2.0	0.6
Gravel	2.1	2.0

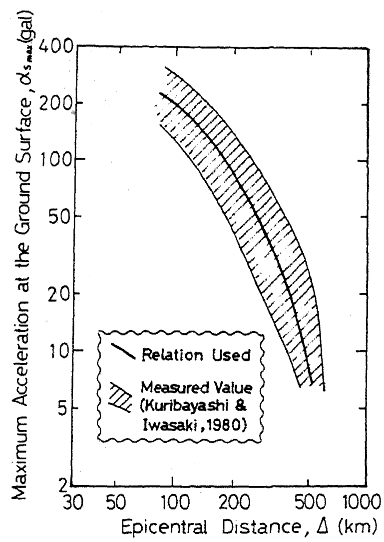

Fig. 5 Relation between Maximum Acceleration at the ground surface, $\alpha_{s_{max}}$, and Epicentral Distance, Δ, during the Miyagi-ken-oki Earthquake

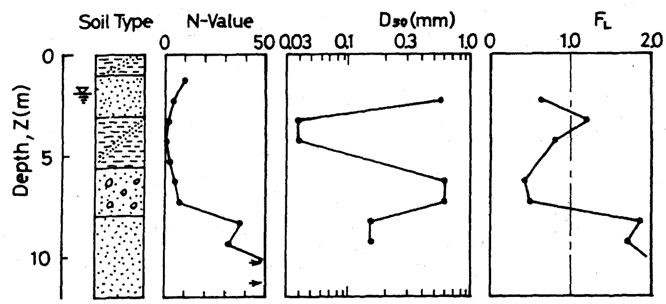

Fig. 6 F_L at a Liquefied Site, Miyagi-ken-oki Earthquake, Yuriage-kami, Y-1

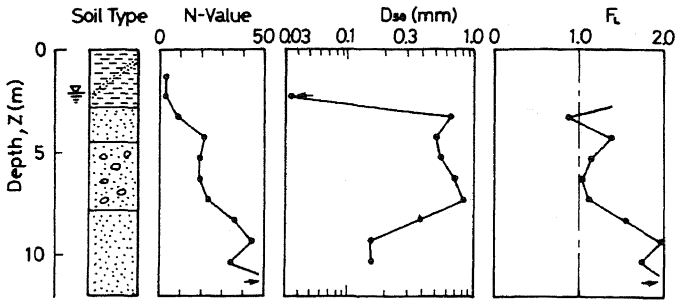

Fig. 7 F_L at a Non-Liquefied Site, Miyagi-ken-oki Earthquake,Yuriage-kami, Y-3

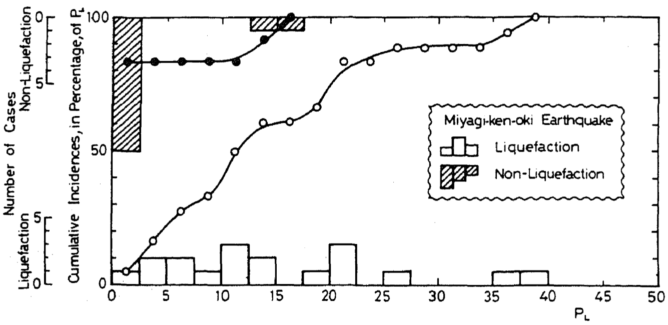

Fig. 8 Distribution of P_L Values and Their Cumulative Incidences, in percentage, at Sites of Liquefaction and of Non-Liquefaction, Miyagi-ken-oki Earthquake

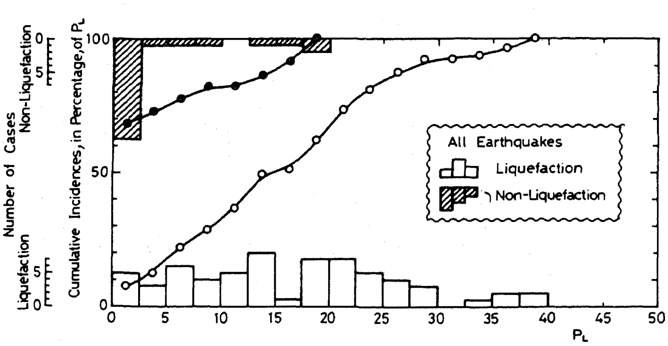

Fig. 9 Distribution of P_L Values and Their Cumulative Incidences, in percentage, at Sites of Liquefaction and of Non-Liquefaction, All Earthquakes