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SYNOPSIS A simplified method based on both a liquefaction resistance factor, F_L and a liquefaction potential factor, P_L has been proposed for evaluating soil liquefaction potential. The factor F_L indicates the liquefaction potential at a given depth of a site, and the factor P_L indicates the one at a site. The effectiveness of the proposed method is investigated by calculating the factors F_L and P_L at both liquefied and non-liquefied sites during past typical earthquakes in Japan, and carrying out shaking table tests.

INTRODUCTION

As a result of various and detail studies on the liquefaction of sandy soils, several simplified and complex methods have been proposed to evaluate the liquefaction potential. The authors, Iwasaki et al. (1978), proposed a simplified method with use of a liquefaction resistance factor F_L and a liquefaction potential factor P_L. Based on the simplified method, the liquefaction potential of sandy soils can be estimated from its N-values, its unit weights, its mean particle diameters, and the maximum acceleration at the ground surface. In this paper, the simplified method is described and to prove the effectiveness of the method, the factors F_L and P_L at 64 liquefied sites and 23 non-liquefied sites during past various earthquakes are calculated according to the proposed simplified method, and also the factor F_L is calculated for the model ground in the shaking table tests on soil liquefaction.

SIMPLIFIED METHOD

In the proposed simplified method an ability to resist the liquefaction of a soil element at an arbitrary depth can be expressed by the factor of liquefaction resistance (F_L).

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

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 cyclic shear test results, as follows :

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

$$R = 0.0882\sqrt{\frac{N}{\sigma_v' + 0.7}} + 0.225\log_{10}\frac{0.35}{D_{50}} \tag{2a}$$

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

$$R = 0.0882\sqrt{\frac{N}{\sigma_v' + 0.7}} - 0.05 \tag{2b}$$

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

$$L = \frac{\tau_{max}}{\sigma_v'} = \frac{\alpha_{s_{max}}}{g}\,\frac{\sigma_v}{\sigma_v'}\,r_d \tag{3}$$

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 kfg/cm^2), and r_d, the reduction factor for dynamic shear stress, accounting for the deformation of the ground. In 1971, Seed and Idriss proposed a relationship between r_d and depth. However, in this paper, the relationship,

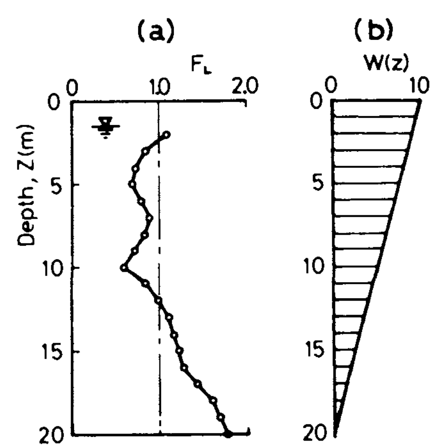

Fig. 1 Integration of F_L

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

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 $F_L < 1.0$ 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 $F_L > 1.0$ for the entire range from $Z = 0$ to $Z = 20$ m, P_L becomes 0.0.

CASE STUDIES OF PAST VARIOUS EARTHQUAKES

Kuribayashi et al. (1974) accumulated data on liquefaction observed during past 44 earthquakes in Japan, and made clear the distribution of liquefied sites and damage of structures due to liquefaction. Furthermore in 1978 liquefaction was observed at about 30 sites during the Miyagi-ken-oki Earthquake (Iwasaki et al. (1980)). Both the liquefaction resistance factor, F_L and the liquefaction potential factor, P_L were calculated for liquefied sites and non-liquefied sites where geotechnical information was available, during each of the following 6 earthquakes : the Nobi Earthquake (1891), the Tonankai Earthquake (1944), the Fukui Earthquake (1948), the Niigata Earthquake (1964), the Tokachi-oki Earthquake (1968) and the Miyagi-ken-oki Earthquake (1978). Calculations were made for 64 liquefied sites and 23 non-liquefied sites. The geotechnical and seismic data at the sites are summarized in Table 1.

Figs. 2 and 3 show the typical variation of F_L with depth at a liquefied and a non-liquefied site, respectively. It can be seen that F_L is, in general, less than 1.0 in the liquefied layers, and greater than 1.0 in the non-liquefied layers. The tendency of F_L at other sites was similar.

Fig. 5 summarize the calculations of F_L with depth at all liquefied and non-liquefied sites in Niigata Earthquake (see Table 1). The liquefied layers shown by black dots in the figure were estimated based on damage to structures (see Fig. 3). It can also be seen that F_L is mainly less than 1.0 in the liquefied layers and the liquefied layers are mostly located at shallower depth than 10 m.

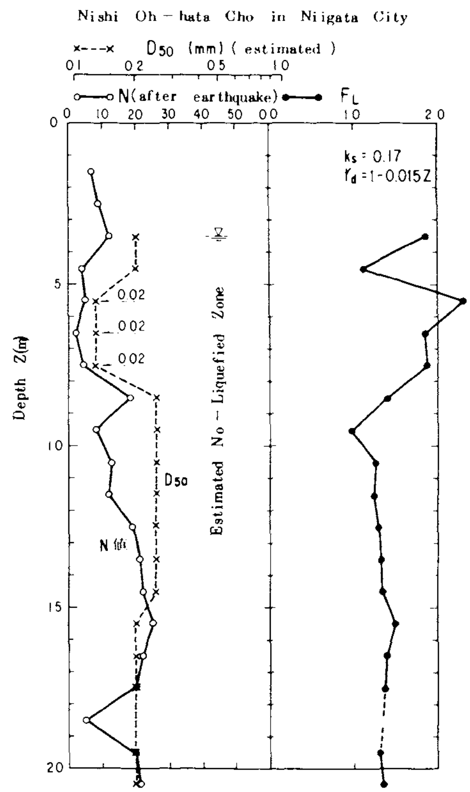

Fig. 2 Relationships between F_L and Z at the Non-Liquefied Site

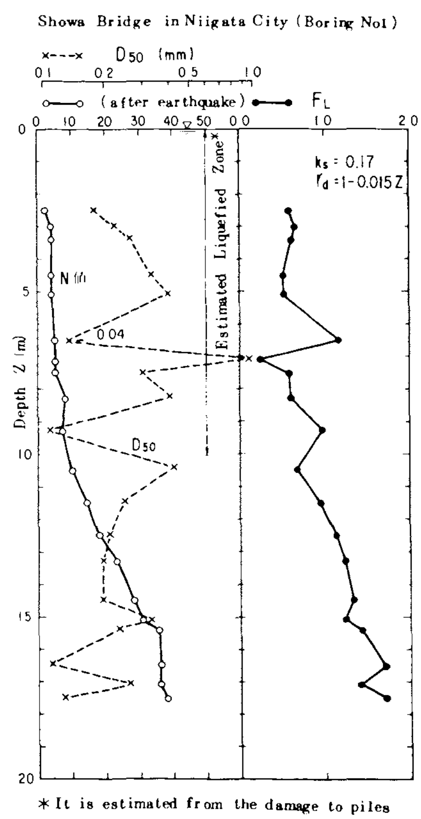

Fig. 3 Relationships between F_L and Z at the Liquefied Site

Table 1 Geotechnical Data at Liquefied Sites and Non-Liquefied Sites

(A) Liquefied Sites

No.	Site			Earthquake	Soil Data*	Depth of Water Table	D_{50}**	Major Soil Type	α_{smax} (gal)	P_L	Ref.
1	Shinano River	1			A	2.0^m	M	Fine to Med. Sand		9.0	
2	Railroad Bridge	2			A	2.5	M	"		24.9	
3	Higashi-Kosen Bridge	Br. 1			A	0.5	E, M	Fine to Coarse Sand		5.8	
4		Br. 2			B	2.5	M	"		1.3	
5		Br. 4			B	2.5	M	"		0.6	
6	Bandai Bridge	Br. 6			A	0.0	M	Medium Sand		20.2	
7	Yachiyo Bridge	Br. 1			B	0.0	E, M	Fine to Med. Sand		18.3	
8		Br. 5			B	0.0	E, M	"		39.5	
9		Br. 7			B	0.0	E, M	"		26.2	(3)
10	Shin-Matsuhama Bri.	Br. 2			A	0.0	M	Med. to Coarse Sand		24.0	
11	Taihei Bridge	Br. 1			A	1.56	M	Coarse Sand		32.9	
12		Br. 2			A	0.0	M	"		19.6	
13	Showa Bridge	Br. 1			A	0.0	E	Coarse Sand		28.2	
14		Br. 2			A	0.0	M	Med. to Coarse Sand		16.5	
15		Br. 3	Niigata City	Niigata, 1964, M=7.5	A	0.0	M	"	170 [1]	14.3	
16		Br. 2			B	0.0	E	"		22.7	
17		Br. 3			B	0.0	E	"		23.2	
18	Niigata Airport				C	0.8	E	"		13.1	
19	Sekiya				A	0.6	M	Coarse Sand		28.6	(4)
20	Niigata Railroad Hospital	Br. 1			B	1.4	M	Fine Sand		20.1	
21		Br. 2			B	1.3	M	Fine to Med. Sand		5.8	(1)
22		Br. 1			A	0.63	M	Fine Sand		13.5	
23		Br. 2			A	1.18	M	"		10.5	
24	Kawagishi-Cho	Br. 1			A	1.25	M	Coarse Sand		18.9	
25		Br. 2			A	1.25	M	"		20.1	(5)
26		Br. 3			A	1.25	M	"		18.9	
27		Br. 4			A	1.25	M	"		14.5	
28	Kawagishi-Cho	BC21-2			B	1.2	E	Medium Sand		5.1	
29		BC21-3			B	1.2	E	"		9.7	(1)
30		BC104			A	1.2	E	"		14.2	
31		BC14			A	1.35	E	Fine to Med. Sand		4.4	
32	Nanae Beach	Br. 1			A	0.95	M	Fine Sand		0.0	
33		Br. 2	Hakodate City	Tokachi-Oki, 1968, M=7.9	A	0.95	M	"	200 [2]	0.0	(6)
34		Br. 3			A	1.1	M	"		5.8	
35	Hachinohe City				A	0.57	E	"	235 [3]	26.7	(7)
36	Gifu City				A	0.9	M	Medium Sand	255 [4]	36.1	
37	Unuma	Kagamigahara	Gifu Pref.	Nobi, 1891, M=8.0	A	0.75	M	Gravelly Sand	210 [4]	13.0	
38	Ogaseike				A	2.1	M	"	210 [4]	8.8	
39	Mangoku, Ohgaki				A	1.2	M	Sand to Grav. Sand	270 [4]	28.7	
40	Meikodori				A	0.6	M	Medium Sand		19.5	(2)
41	Kohmei		Nagoya City	Tonankai, 1944, M=8.0	A	0.9	M	Coarse Sand	200 [5]	26.3	
42	Inaei				A	0.23	M	Silty to Fine Sand		17.7	
43	Takaya 45		Fukui Pref.	Fukui, 1948, M=7.3	A	4.2	M	Med. to Coarse Sand	325 [4]	22.7	
44	Maruoka No. 2				A	1.8	M	Med. Sand to Gravel	315 [4]	24.9	
45	Takaya 2-168				A	4.2	M	Sandy Silt to Med. Sand	325 [4]	11.4	
46	Abukuma Bridge	Br. 4			A	0.0	M	Coarse Sand	175 [6]	12.2	
47	Morth of Abukuma River				B	0.0	E	"	180 [6]	20.4	
48	Yuriage-kami	Y-1			A	1.32	M	Silt to Coarse Sand	180 [6]	10.3	
49	"	Y-2			A	0.85	M	Fine to Coarse Sand	180 [6]	9.4	
50	Yuriage Bridge	No. 1			A	1.7	M	Fine to Med. Sand	185 [6]	7.0	
51	"	No. 2			A	1.3	M	Med. to Coarse Sand	185 [6]	0.5	
52	"	No. 3			A	0.26	M	"	185 [6]	21.8	
53	Yamazaki		Miyagi Pref.	Miyagi-ken-Oki, 1978, M=7.4	A	0.87	E	Sandy Silt to Med. Sand	190 [6]	13.0	
54	Oiri (1)	No. 1			A	4.3	M	Clay to Med. Sand	210 [6]	4.1	(8)
55	"	No. 2			A	2.4	M	"	210 [6]	4.1	
56	Uomachi	B-1			B	0.0	E	Med. to Silty Sand	230 [6]	39.1	
57	"				B	0.0	E	"	230 [6]	36.5	
58	Rifu	No. 12			B	2.72	E	Clay to Coarse Sand	185 [6]	20.4	
59	Shiomi	No. 1			B	0.0	E	Fine to Med. Sand	225 [6]	14.1	
60	"	No. 2			B	0.0	E	Fine Sand	225 [6]	27.1	
61	"	No. 3			B	0.0	E	"	225 [6]	18.2	
62	Nakamura	N-4			A	0.5	M	Sandy Silt to Med. Sand	180 [6]	12.3	
63	"	N-5			A	1.3	M	Fine to Coarse Sand	180 [6]	5.6	
64	Wabuchi	W-2			A	2.45	M	Silt to Fine Sand	295 [7]	21.2	

(B) Non-Liquefied Sites

No.	Site			Earthquake	Soil Data*	Depth of Water Table	D_{50}**	Major Soil Type	α_{smax} (gal)	P_L	Ref.
1	Jindoji				A	2.3^m	E	Fine Sand		0.0	
2	Kogane-cho				A	5.2	E	Medium Sand		0.0	
3	Higashi Kosen	Br. 5	Niigata City		B	2.5	M	Fine to Coarse Sand		0.0	
4	Shin Matsuhama Bri.	Br. 1		Niigata, 1964, M=7.5	A	1.2	M	Med. to Coarse Sand		18.6	(3)
5					A	0.0	M	Fine Sand	170 [1]	8.2	
6	Showa Bridge	Br. 4			A	0.0	E	Medium Sand		4.6	
7	Nishi Oh-Hata-Cho				A	3.5	E	Fine to Med. Sand		0.4	
8	Gotanda Bridge	Br. 1	Kamo City		A	0.1	M	Fine to Silty Sand		5.3	
9	"	Br. 2			A	4.3	M	Fine Sand to Silt		0.0	
10	Maruoka		Fukui Pref.	Fukui, 1948, M=7.3	A	1.8	M	Silt to Med. Sand	295 [4]	18.7	(2)
11	Nakamura	N-1			A	0.85	M	Fine to Med. Sand	180 [6]	0.3	
12	"	N-2			A	0.90	M	Silty to Coarse Sand	180 [6]	1.0	
13	Yuriage-kami	Y-3			A	2.15	M	"	180 [6]	0.9	
14	Kitakami River	No. 10			A	1.55	E	Fine Sand	230 [6]	1.5	
15	Natori River, 3.2 Km			Miyagi-ken-Oki, 1978, M=7.4	A	2.50	E	Clay to Coarse Sand	180 [6]	0.8	
16	Kinnou Bridge	P8	Miyagi Pref.		A	4.0	M	Silt to Silty Sand	195 [6]	0.6	(8)
17	Abukuma Bridge	Br. 1			A	4.3	M	Med. to Coarse Sand	175 [6]	0.0	
18	"	Br. 2			A	3.4	M	"	175 [6]	0.7	
19	Eai Bridge	No. 1			A	8.0	E	Fine Sand to Clay	175 [6]	0.0	
20	Minami Sendai	No. 2			B	0.8	E	Sandy Silt to Gravel	180 [6]	0.0	
21	Uomachi +	A-1			B	0.0	E	Med. to Silty Sand	230 [6]	13.9	
22	" +	A-2			B	0.0	E	"	230 [6]	17.1	
23	Wabuchi	W-3			A	3.35	M	Fine to Med. Sand	260 [7]	3.7	

* B : Soil Data before Earthquake A : Soil Data after Earthquake

** E : Estimated by Using Table 2 M : Measured

\+ Compacted by Sand Compaction Piles, near Uomachi B-1 and B-2

1 Recorded Value at Kawagishi-Cho

2 Estimated from Recorded Value at Other Sites

3 Recorded Value at Hachinohe

4 Estimated by the Empirical Equation for Alluvial Deposits (

5 Estimated from Damage to Structures

6 Estimated by Using Fig. 4

7 Estimated by Dynamic Response Analyses of Ground

Reference

(1) BRI (1965)

(2) BRI (1969)

(3) Japanese Society of Civil Engineers

(4) Ishihara (1976)

(5) J.S.S.M.F.E. (1976)

(6) Kishida (1970)

(7) Ohashi et al. (1977)

(8) Yasuda et al. (1980)

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	1.95	0.15
Medium Sand	2.0	0.35
Coarse Sand	2.0	0.6
Gravel	2.1	2.0

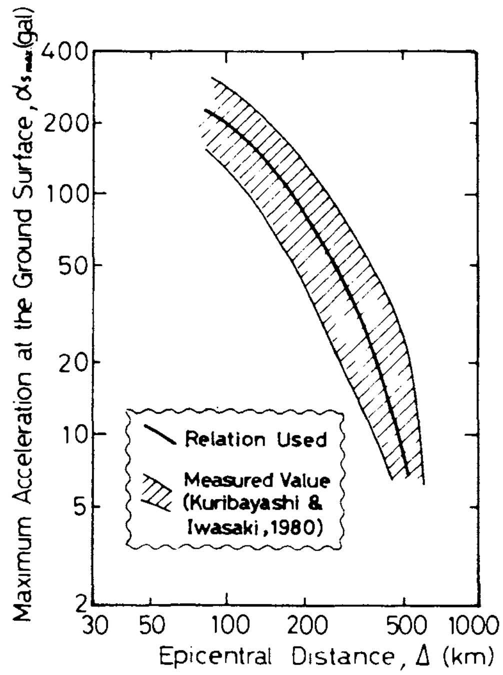

Fig. 4 Relation between Maximum Acceleration at the ground surface, $\alpha_{s_{max}}$, and Epicentral Distance, Δ, during the Miyagi-ken-oki Earthquake

Fig. 6 shows the frequency and accumulative incidences of F_L values for both liquefied and non-liquefied layers at all sites in Table 1. Hereupon the liquefied layers were estimated based on damage to structures or if not estimated by soil conditions, i.e., the saturated sandy layers whose N-value is less than 15 and whose D_{50} ranges from 0.02 mm to 2.0 mm were regarded as liquefied layers. The distribution of F_L at liquefied layers is very different from that at non-liquefied layers. At liquefied layers most (about 87 %) of F_L values distribute in the range less than 1.0, and while at non-liquefied layers most (about 89 %) of F_L values distribute in the range more than 1.0. However it must be noticed that about 13 % of F_L values exceed 1.0 at liquefied layers and about 11 % of F_L values indicate less than 1.0 at non-liquefied layers.

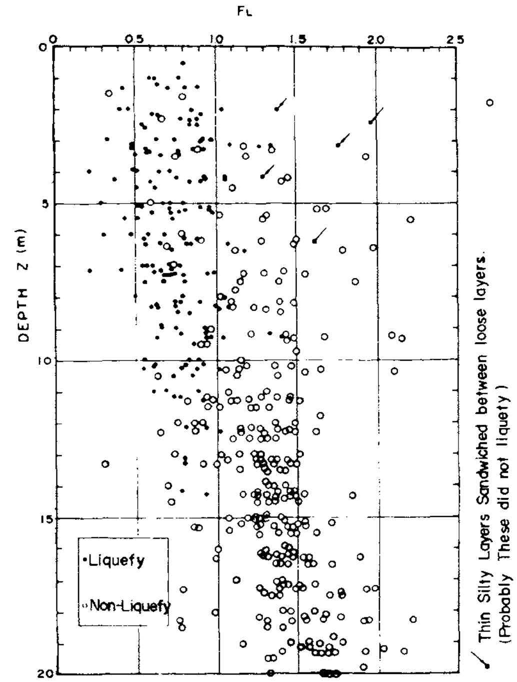

Fig. 5 Relationship between F_L and Z at the Liquefied and Non-Liquefied Sites during Niigata Earthquake

Fig. 7 summarize the calculations of P_L at all liquefied and non-liquefied sites in Table 1, i.e., both relation between number of case and P_L and relation between accumulative percentage of P_L and P_L. According to Fig. 7, it is found at non-liquefied sites P_L is less than 20 and the probability that P_L is less than 5 is 70 %, on the other hand at liquefied sites the probability that P_L is less than 5 is only 20 % and 50 % of the sites range more than 15. Based on the above results, the assessment for soil liquefaction potential using P_L can be done as follows.

$P_L = 0$ ---- Liquefaction potential is very low and detail investigations on soil liquefaction aren't needed in general.

$0 < P_L \le 5$ --- Liquefaction potential is low but detail investigations on soil liquefaction are needed only for specially important structures.

$5 < P_L \le 15$ -- Liquefaction potential is rather high and detail investigations on soil liquefaction are needed for important structures and countermeasures of soil liquefaction are neededed in general.

$15 < P_L$ ---- Liquefaction potential is very high and detail investigations and countermeasures on soil liquefaction are needed.

As mentioned in the above, it has been shown that the liquefaction potentional factor P_L may be used to assess the liquefaction potential at a certain site reasonably. Moreover the necessity for detail investigations on soil liquefaction also can be judged based on the factor P_L calculated by the proposed simplified method.

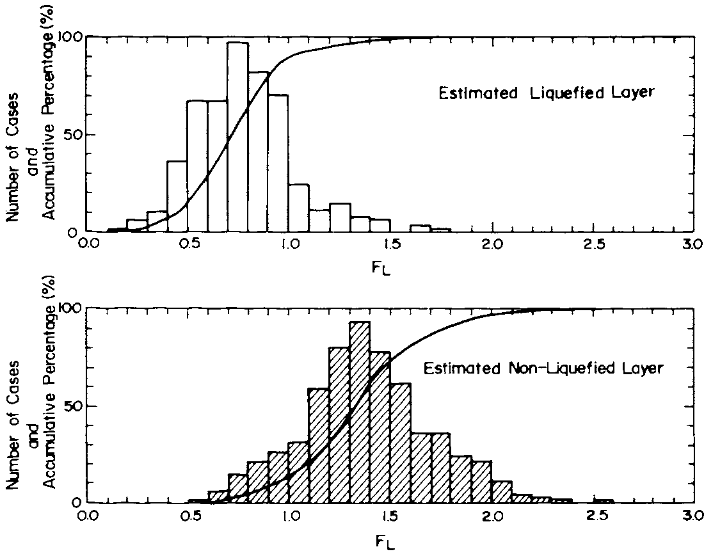

Fig. 6 Distribution of F_L Values and Their Accumulative Incidences, in Percentage, Comparing Liquefied Sites with Non-Liquefied Sites in Table 1

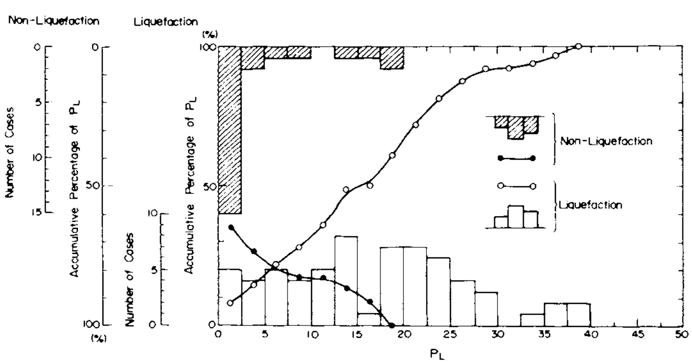

Fig. 7 Distribution of P_L Values and Their Accumulative Incidences, in Percentage, Comparing Liquefied Sites with Non-Liquefied Sites in Table 1

SHAKING TABLE TESTS ON F_L

The authors carried out shaking table tests to clarify the effectiveness of F_L for assessing liquefaction potential of sandy soil. A loose saturated sandy ground model with about 0.95 m depth, 6 m length and 3 m width was prepared on shaking table and shaked by sinusoidal wave. The frequency of the inputted motion was 7 Hz and the magnitude of the table acceleration ranged from 30 gals to 250 gals. The acceleration and pore water pressure of ground model during shaking were measured.

Figs. 8 and 9 shows the relationships between ground acceleration, pore water pressure and F_L for non-liquefied and liquefied test, respectively. In these figures, F_L values were estimated by equations (2) and (3) based on test results. It can be seen that according to the increase of pore water pressure, F_L values decrease to less than 1.0 in the liquefied layers (see Fig. 9) and on the other hand F_L values are more than 1.0 in the non-liquefied layers (see Fig. 8).

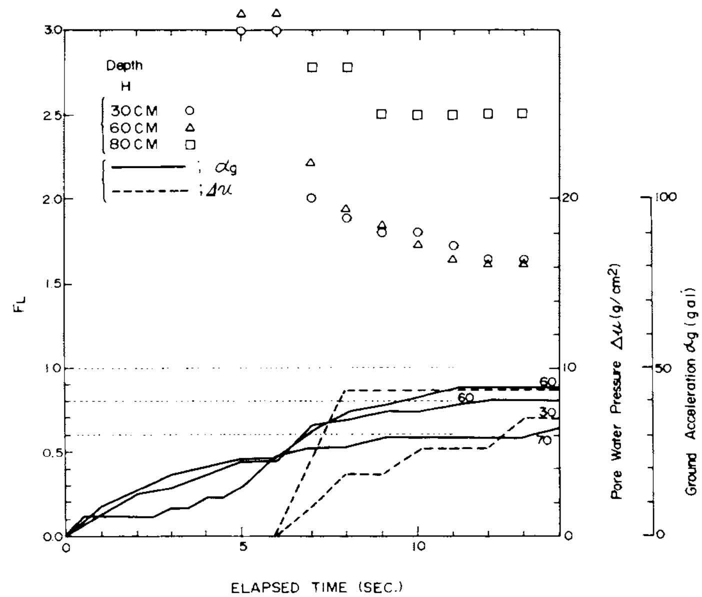

Fig. 8 Relationships between Pore Water Pressure and Acceleration of Non-Liquefied Sand Layers and F_L Values in Shaking Table Tests

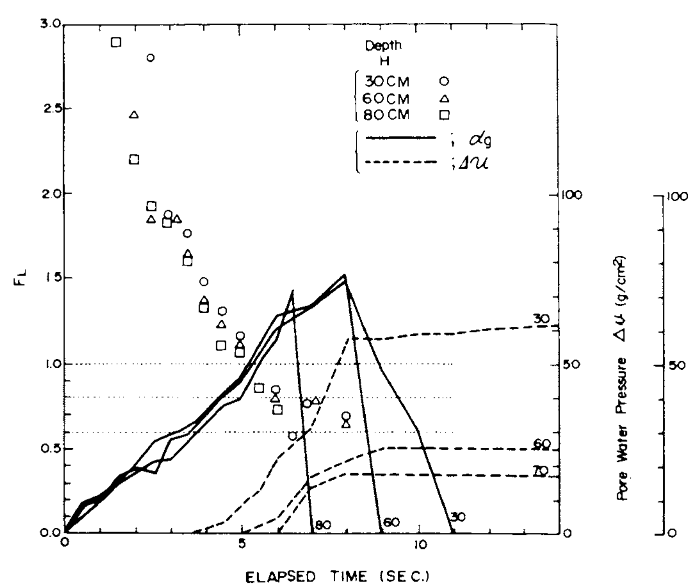

Fig. 9 Relationships between Pore Water Pressure and Acceleration of Liquefied Sand Layers and F_L Values in Shaking Table Tests

Fig. 10 summarize the relation between F_L and the rate of ground liquefaction, $\Delta u / \sigma_v'$ (Δu : an excessive pore water pressure, σ_v' an effective overburden pressure) for liquefied layers. F_L decreases according to the increase in $\Delta u / \sigma_v'$, and on the average F_L is less than 1.0 when $\Delta u / \sigma_v'$ is more than 0.5 and when $\Delta u / \sigma_v'$ is 1.0, i.e., the sand layer liquefy perfectly, F_L decreases to less than about 0.6.

As mentioned in the above, it's been clarified in these shaking table tests that F_L is adequately equivalent to the liquefaction phenomena and may be used to estimate the soil liquefaction potential of saturated sandy layers.

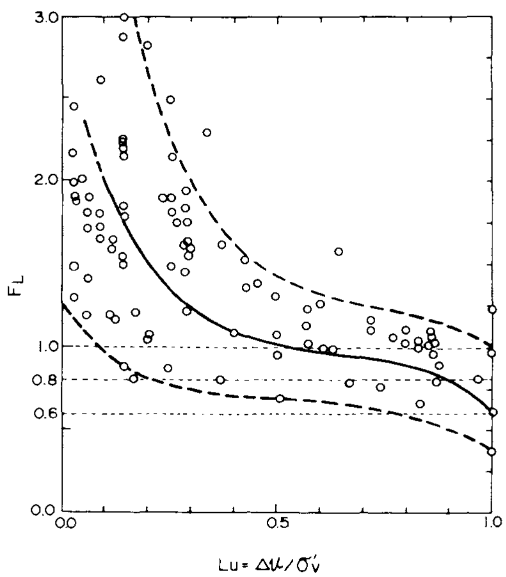

Fig. 10 Relationships between F_L Values and the Rate of Ground Liquefaction $\Delta u / \sigma_v'$ in Shaking Table Tests

CONCLUSIONS

The simplified method based on the liquefaction resistance factor, F_L and the liquefaction potential factor, P_L proposed to assess the liquefaction potential was investigated by calculating the factors at 64 liquefied and 23 non-liquefied sites during past 6 Earthquakes in Japan and shaking table tests. From these studies, it was found that most values of F_L are less than 1.0 at liquefied layers, and are larger than 1.0 at non-liquefied layers. Further, the values of P_L and their incidences at liquefied sites differ from the ones at non-liquefied sites. Therefore, the liquefaction potential can be predicted reasonably by calculating the factors F_L and P_L .

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