Prediction of liquefaction resistance based on CPT tip resistance and sleeve friction

Prédiction de la résistance à la liquéfaction fondée sur le frottement latéral et la résistance de pointe CPT

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ABSTRACT: The CPT tests are conducted at 68 sites where soil liquefaction is known to have or have not occurred during recent earthquakes. The correlations of the cyclic shear stress ratio, CPT tip resistance and soil behavior type index, q_{t1} and I_c, with the actual soil performance are examined. The critical q_{t1}-value causing liquefaction is found to be independent of I_c if $I_c < 1.65$, while it decreases if $I_c \geqq 1.65$. The boundary line separating liquefiable from non-liquefiable conditions is then presented in terms of q_{t1} and I_c, which is uniquely expressed regardless of the fines content.

RESUME: Les tests de pénétration au cône (CPT) ont été effectués sur 68 sites ayant présenté ou non une liquéfaction des sols au cours de tremblements de terre récents. Les corrélations entre le taux de contrainte tangentielle cyclique, la résistance au cône CPT et les indices du type de comportement du sol, q_{t1} et I_c, et la performance réelle du sol sont examinées. Il en résulte que la valeur critique provoquantla liquéfaction est indépendante des autres paramètres si $I_c < 1{,}65$, tandis qu'elle diminue si $I_c \geqq 1{,}65$. En conséquence, la limite séparant les états liquéfiables des états non liquéfiables est présentée en termes de q_{t1} et I_c, mais ne tient absolument pas compte du contenu en fraction de fines.

1 INTRODUCTION

Several practical methods using in situ testing techniques have been presented for evaluating the liquefaction resistance of a soil deposit subjected to earthquake loading. Among these techniques, the standard penetration test (SPT) has been widely used particularly in Japan and North America. However, because of the variability of the SPT results and the reliability of the cone penetration test (CPT), the CPT has received increasing interest in recent years, and empirical methods using CPT data have been presented and used.

Although several methods based on field performance data have been proposed (e.g., Stark and Olson, 1995), they require physical properties of sand, e.g., mean grain size and fines content, which cannot be obtained from the CPT test. This calls for additional field investigation, and thus impairs the expedience of the CPT for evaluating liquefaction resistance. To overcome this disadvantage, several liqufaction charts have been presented (Olsen et al., 1995, Suzuki et al., 1995) in which field investigation other than CPT test is not required. However, since these charts are based solely on laboratory test results, field verification is needed before they are used in practice.

The object of this paper is to study correlations between liquefaction resistance and CPT tip resistance and soil behavior type index, based on field performance data and field tests, in an attempt to establish a liquefaction chart using CPT data only. For this purpose, the CPT tests were conducted at sites that were known to have or have not liquefied during recent Japanese earthquakes.

2 CPT TEST SITES AND STRONG MOTIONS

The CPT tests were conducted at 68 sites in Japan. For each site, strong motion accelerograms as well as field performance records were available for recent earthquakes, as shown in Table 1. These include the Kushiro-Oki earthquake of January 15, 1993; the Hokkaido-Nansei-Oki earthquake of July 12, 1993; the Hokkaido-Toho-Oki earthquake of October 4, 1994; and the Hyogoken-Nambu earthquake of January 17, 1995. The CPT tests were made with a penetration rate of 2 cm/s, which resulted in continuous records with depth of three components: tip resistance q_t, sleeve friction f_s, and pore water pressure P_w. Modified cone penetration resistance q_{t1} corrected to an effective overburden pressure of 1 kgf/cm² (98 kPa) is given by:

$$q_{t1}=q_t / \sigma_v'^{0.5} \quad (1)$$

The shear stress ratios, τ_d / σ_v', which might have been developed in the field during the earthquakes are estimated from the following equation (Tokimatsu and Yoshimi, 1983):

Table 1. CPT test sites.

Test regions	Number of test sites	Number of liquefied sites
Kushiro City	22	8 (6)*2
Nemuro City	1	1
Hakodate City*1	8	4
Mori Town	2	1
Oshamanbe Town	7	4
Kobe City	24	16
Nishinomiya City	2	2
Amagasaki City	2	2
Total	68	38

*1 : Kamiiso Town in the neighborhood of Hakodate City is involved.
*2 : The Kushiro-Oki earthquake (The Hokkaido-Toho-Oki earthquake)

Table 2. Seismic data at test sites.

Earthquakes	Magnitude	Test sites	a_{max} (gal)
Kushiro-Oki	7.8	Kushiro City	150-410
		Nemuro City	155
Toho-Oki	8.1	Kushiro City	90-290
		Nemuro City	360
Nansei-Oki	7.8	Hakodate City*1	120
		Mori Town	175
		Oshamanbe Town	240
Hyogoken-Nambu	7.2	Kobe City	340-830
		Nishinomiya City	790
		Amagasaki City	470

*1 : Kamiiso Town in the neighborhood of Hakodate City is involved.

$$\left(\frac{\tau_d}{\sigma_v'}\right) = r_N \frac{a_{max}}{g} \left(\frac{\sigma_v}{\sigma_v'}\right)(1-0.015Z) \qquad (2)$$

in which τ_d=amplitude of uniform shear stress cycles equivalent to actual seismic shear stress time history, r_N=0.1(M-1), M= earthquake magnitude, a_{max}=maximum horizontal acceleration at ground surface, σ_v'=initial effective overburden pressure, σ_v=initial overburden pressure, and Z=depth below the ground surface in meters. Table 2 shows the characteristics of the four earthquakes.

The Kushiro-Oki earthquake caused soil liquefaction mainly in the reclaimed land area along the Port of Kushiro in Kushiro City. The strong motion station of the Port and Harbor Research Institute (PHRI) at the West Wharf in the Port of Kushiro registered ground motions (prompt report on strong-motion accelerations No. 41, 1993). Since August, 1994, strong motion stations have been temporarily set at 23 sites in the city and its vicinity to study site effects during earthquakes. Most of these stations together with the PHRI station recorded the ground motions during the Hokkaido-Toho-Oki earthquake. The CPT tests were conducted at 22 sites.

In Nemuro City, the above two earthquakes induced soil liquefaction at the reclaimed land along the Port of Hanasaki. The strong motion station of the PHRI at the Port of Hanasaki recorded ground motions during the Kushiro-Oki earthquake (prompt report No. 41, 1993) and the Hokkaido-Toho-Oki earthquake (prompt report No. 44, 1994). The CPT test was conducted at the reclaimed lands of the Port of Hanasaki near the station.

The Hokkaido-Nansei-Oki earthquake caused soil liquefaction mainly in the reclaimed land area along the Port of Hakodate in Hakodate City, the Port of Mori in Mori Town and fluvial deposits in Oshamanbe Town. Ground motions were recorded at the Chuo Wharf in the Port of Hakodate and in Mori and Oshamanbe Town (prompt report No. 43, 1993). The CPT tests were conducted at 17 sites: 8 in Hakodate City, 2 in Mori Town, and 7 in Oshamanbe Town.

The Hyogoken-Nambu earthquake caused soil liquefaction mainly in the reclaimed land area along the shoreline in Hyogo prefecture, e.g., at the Port of Kobe. Several strong motion stations registered ground motions (prompt report No. 46, 1995). The CPT tests were conducted at 28 sites: 24 in Kobe City, 2 in Nishinomiya City, and 2 in Amagasaki City.

Since the duration of the Kushiro-Oki earthquake was short compared with that for a typical M=7.8 earthquake, a correction factor in terms of earthquake magnitude r_N was assigned a value of 0.58, which corresponds to a M=6.8 earthquake (Seed et al., 1975).

3 CORRELATION BETWEEN q_{t1} AND τ_d/σ_v'

Fig. 1 shows the correlation between shear stress ratio τ_d/σ_v' and modified tip resistance q_{t1} for all data sets. The solid circles represent the liquefied data, the open circles the non-liquefied data, and open triangles the boundary data. The boundary separating liquefiable from non-liquefiable conditions is obscure in Fig. 1, because the effects of fines content FC are neglected.

Fig. 2 shows the relationship between fines content FC and soil behavior type index I_c (Robertson et al., 1995) defined as:

$$I_c = \{(3.47-\log Q)^2+(\log F+1.22)^2\}^{0.5} \qquad (3)$$

$$Q= (q_t - \sigma_v)/\sigma_v' \qquad (4)$$

$$F = f_s/(q_t - \sigma_v)\times 100 \quad (\%) \qquad (5)$$

The soil behavior type index I_c tends to increase with increasing fines content FC, and thus could be used in place of fines content FC. The soil behavior type indexes I_c of 1.5 and 1.75 roughly correspond to fines contents FC of 5% and 10% respectively. This suggests the possibility that the effects of fines content FC may be replaced by those of I_c, and thus the occurrence of liquefaction may be characterized by the combined effects of q_{t1} and I_c.

Fig. 3 shows the combined effects of q_{t1} and I_c on the occurrence of liquefaction. The q_{t1}-value of the liquefied soils decreases with increasing I_c and no soils with $I_c>2.4$ liquefied. This indicates that soils invulnerable to liquefaction can be identified using I_c. However, the boundary is still obscure in Fig. 3.

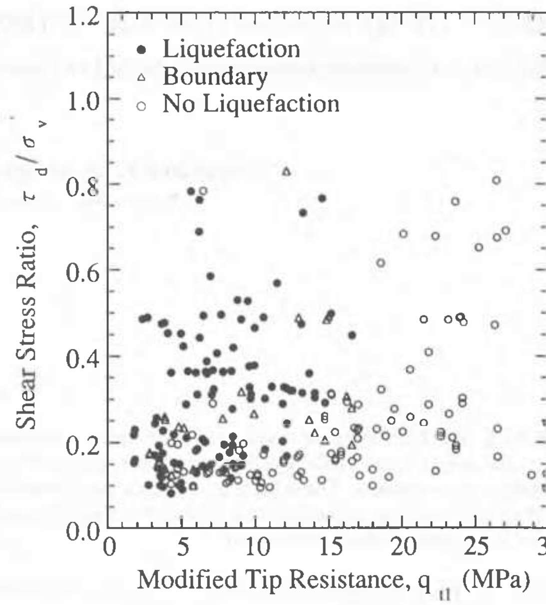

Fig. 1. Correlation between shear stress ratio and modified tip resistance.

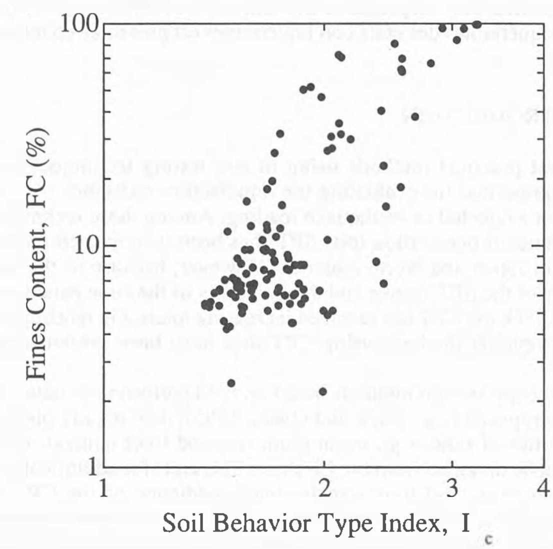

Fig. 2. Correlation between soil behavior type index and fines content.

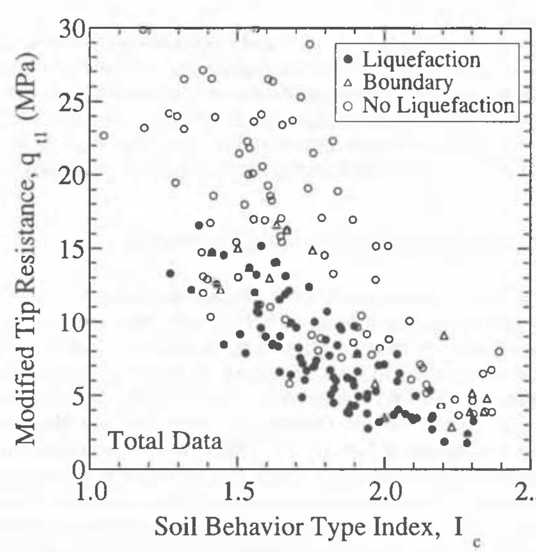

Fig. 3. Correlation between modified tip resistance and soil behavior type index.

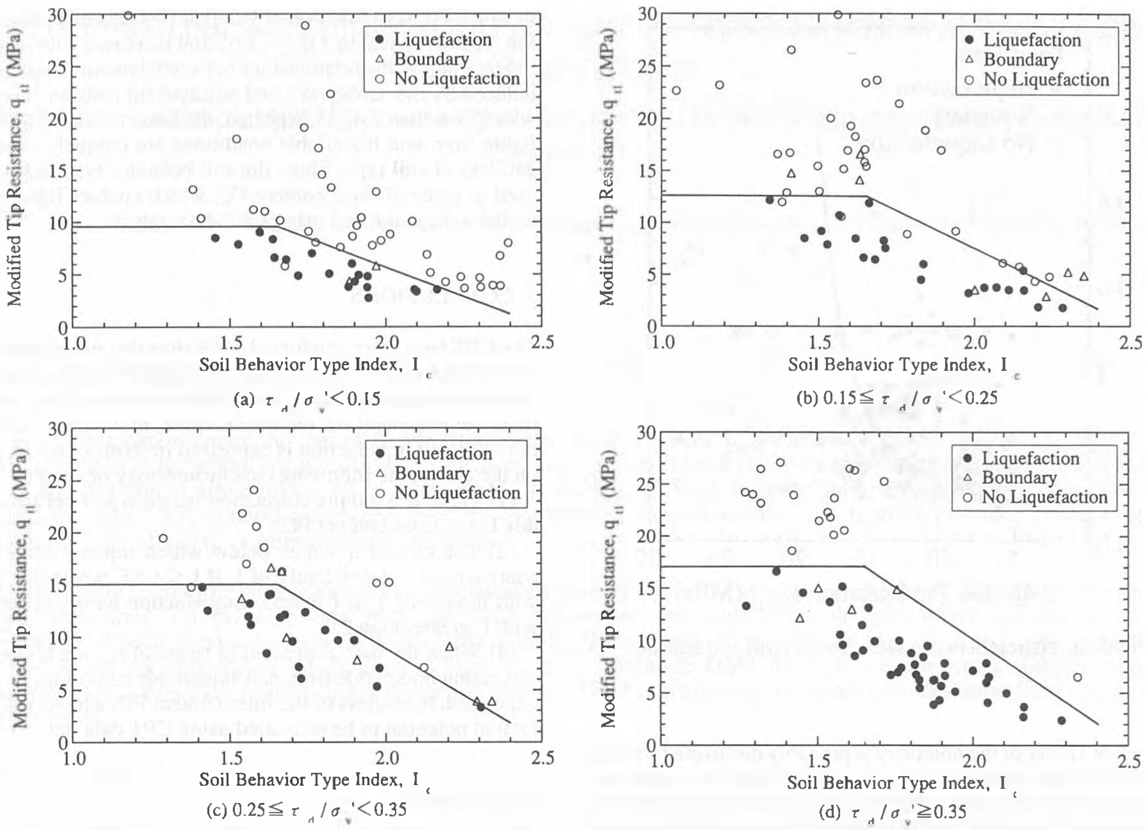

Fig. 4. Field correlation between modified tip resistance and soil behavior type index in which the data points are classified on the basis of the shear stress ratio.

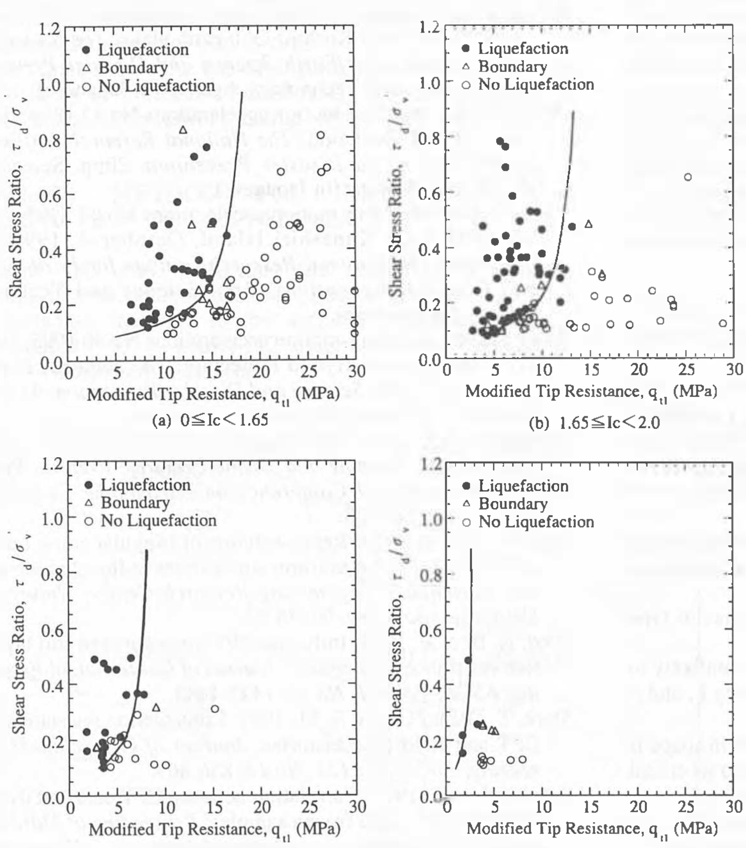

Fig. 5. Field correlation between shear stress ratio and modified tip resistance in which the data points are classified on the basis of the soil behavior type index.

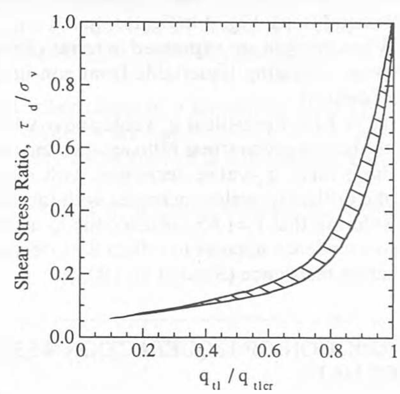

Fig. 6. Correlation between τ_d/σ_v' and the q_{t1}-value normalized with the critical q_{t1cr}-value.

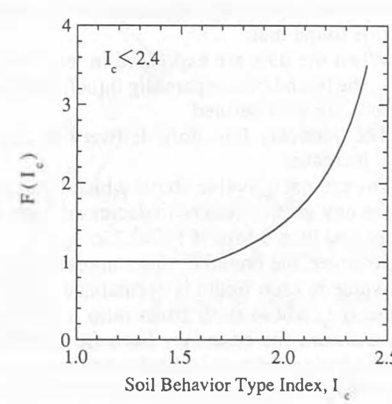

Fig. 7. Correlation between soil behavior type index and adjusted function.

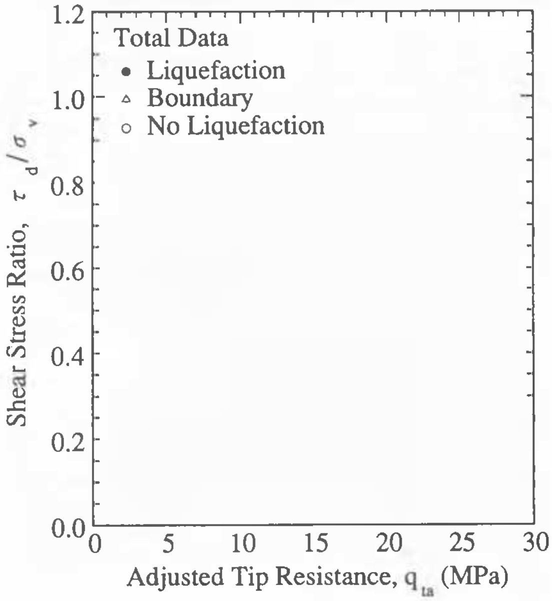

Fig. 8. Field correlation between shear stress ratio and adjusted tip resistance.

The lack of clarity of the boundary is probably due to the fact that the effects of shear stress ratio τ_d/σ_v' induced by the earthquake are neglected. Thus, the data shown in Fig. 3 are classified into four levels of τ_d/σ_v' and shown in Fig. 4. Fig. 4 (a) to (d) correspond to the data with $\tau_d/\sigma_v'<0.15$, $0.15\leqq\tau_d/\sigma_v'<0.25$, $0.25\leqq\tau_d/\sigma_v'<0.35$, $0.35\leqq\tau_d/\sigma_v'$, respectively. Also shown in each figure is the boundary line that separates liquefiable from non-liquefiable conditions. Fig. 4 indicates that:

(1) When the data are expressed in terms of shear stress ratio, the boundaries separating liquefiable from non-liquefiable conditions are well defined.

(2) If $I_c<1.65$, the critical q_{t1} value above which liquefaction did not occur for the given stress ratio appears independent of I_c. If $I_c\geqq 1.65$, the critical q_{t1}-value decreases with increasing I_c. In both cases, the critical q_{t1}-value increases with increasing τ_d/σ_v'.

Considering that I_c=1.65 corresponds to a fines content of 5%, the above tendency appears to reflect the effects of fines content on liquefaction resistance (Seed et al, 1985).

4 PREDICTION OF LIQUEFACTION RESISTANCE BASED ON CPT DATA

Based on the above discussions, the data shown in Fig. 1 are replotted in Fig. 5 (a) to (d), in terms of I_c, i.e., $I_c<1.65$, $1.65\leqq I_c<2.0$, $2.0\leqq I_c<2.25$ and $2.25\leqq I_c<2.4$. Also shown in each figure is the boundary line separating liquefiable from non-liquefiable conditions. It is found that:

(1) When the data are expressed in terms of soil behavior type index I_c, the boundaries separating liquefiable from non-liquefiable conditions are well defined.

(2) The boundary line shifts leftward as the soil behavior type index I_c increases.

(3) The critical q_{t1}-value above which liquefaction is unlikely to occur for any given stress ratio decreases with increasing I_c, and it becomes less than 3 Mpa if $I_c\geqq 2.25$.

(4) However, the boundary lines appear to be similar in shape if the q_{t1}-value in each figure is normalized with respect to its critcal q_{t1}-value, q_{t1cr}, whose shear stress ratio τ_d/σ_v' is equal to 1.

Fig. 6 shows the boundary lines from four cases in Fig. 5 in which the horizontal axis is the normalized q_{t1}-value defined as q_{t1}/q_{t1cr}. It is found that all the boundary lines become to fall within a narrow band, regardless of the soil behavior type index. Thus, the adjusted tip resistance, q_{ta}, can be defined as:

$$q_{ta}=q_{t1}\times F(I_c) \qquad (6)$$

in which $F(I_c)$ is a function of I_c defined by the correlation shown in Fig. 7, and is equal to 1 if $I_c<1.65$ and increases with increasing I_c.

Fig. 8 shows the relationships between dynamic shear stress ratio induced by the earthquake and adjusted tip resistance for the data with I_c less than 2.4. As expected, the boundaries separating liquefiable from non-liquefiable conditions are uniquely expressed regardless of soil type. Thus, the soil behavior type index I_c can be used in place of fines content FC, which enables liquefaction potential to be estimated using CPT data only.

5 CONCLUSIONS

The CPT tests were conducted at 68 sites that were known to have or have not liquefied during recent Japanese earthquakes. Empirical correlations separating liquefiable from non-liquefiable conditions are presented for the above cases, in which the shear stress ratio causing liquefaction is expressed in terms of q_{t1} and I_c. Based on the above, the following conclusions may be drawn:

(1) There is a unique correlation between soil behavior type index I_c and fines content FC.

(2) The critical q_{t1}-value below which liquefaction occurs appears almost independently of I_c if $I_c<1.65$, whereas it decreases with increasing I_c if $I_c\geqq 1.65$. Liquefaction hardly occurs in soils with I_c greater than 2.4.

(3) When the data expressed in terms of q_{ta}, the boundary lines separating liquefiable from non-liquefiable conditions are uniquely expressed, regardless of the fines content FC, which enables liquefaction potential to be estimated using CPT data only.

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