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SYNOPSIS : Undrained cyclic triaxial tests have been conducted on specimens of dense diluvial gravelly soils taken from four sites by an in-situ freeze sampling method. The test results and previous test results obtained by other researchers suggest that the dynamic strength of gravelly soils can be evaluated based on both the modified blow count of the penetration tests and on the effective confining pressure. A method for the evaluation by utilizing the large penetration test (LPT) blow counts is proposed in this paper by considering the effect of the effective confining pressure on the dynamic strength.

INTRODUCTION

Dense gravelly ground is usually considered to have sufficient bearing capacity without any accurate evaluation of its mechanical properties because of its large STP blow count (N-value). Recently, as important structures are being constructed increasingly on dense gravelly ground, accurate evaluation of the mechanical properties of gravelly soils is needed more than ever. This paper investigates the relevance of the blow counts of dynamic penetration tests as indexes for evaluating in-situ dynamic strengths of gravelly soils by field and laboratory tests. Field tests were performed at four sites : A, K, T and KJ, where the in-situ freeze sampling was performed along with dynamic penetration tests in nearby boreholes (Kataoka et al., 1989). The dynamic strength of undisturbed gravelly soil samples measured by large-scale undrained cyclic triaxial tests mainly under in-situ effective confining pressure was compared with the penetration resistance to establish their correlations and to study influencing factors on them. In this paper, the dynamic strength is defined as the stress ratio which is required to reach the double axial strain amplitude DA=2% or 2.5% in 20 loading cycles.

UNDRAINED CYCLIC TRIAXIAL TESTS ON GRAVELLY SOILS OBTAINED BY IN-SITU FREEZE SAMPLING

Site A

The soil profile at this site is shown in Fig.1. The gravelly soil of diluvial origin was investigated at depths of 3.3 m to 10.0 m from the ground surface. A Large Penetration Test (LPT), which was first introduced by Kaito et al.(1971) for testing gravelly soils, was performed. The LPT penetration resistance, N_d-value, was about 20 for the upper gravelly layer (GL -3.3 m to -5.0 m), and about 30 for the lower gravelly layer (GL -7.0 m to -10.0 m). 100 mm diameter samples were obtained by in-situ freeze sampling. The groundwater level was 1.0 m from the ground surface. The grain size distribution shown in Fig. 2 indicates that the maximum grain size is about 20 to 40 mm and that the fines content is

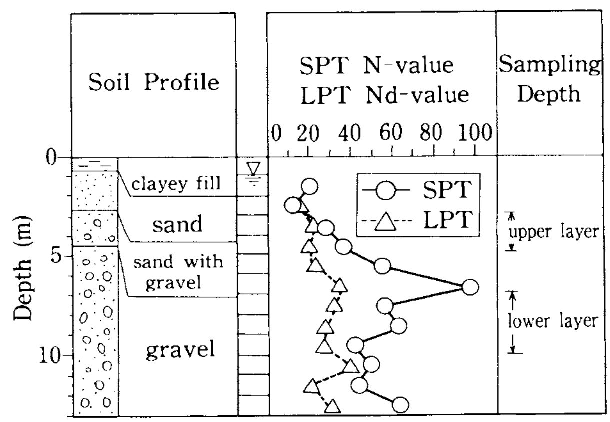


Fig. 1 Soil profile at Site A

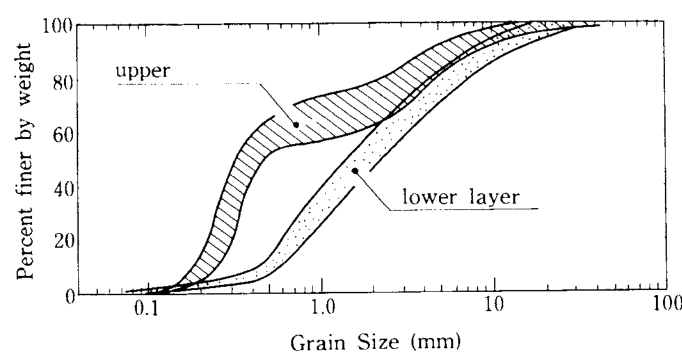


Fig. 2 Typical grain size distributions of Site A gravelly soils sampled by in-situ freeze sampling

less than 2.0%. The results of the undrained cyclic triaxial tests are shown in Fig. 3. For the upper-layer samples, the stress ratios that are required to reach the double axial strain amplitude DA = 2% and 5% in 20 loading cycles are 0.42 and 0.50, respectively. For the lower-layer samples, the same stress ratios were 0.34 and 0.38, respectively.

The in-situ freeze sampling was first conducted in 1984 and again in 1987. In the second (1987) sampling, double-tube core barrels with diamond teeth were used to obtain samples with smooth lateral surfaces, whereas barrels with metal teeth were used in the first (1984) sampling. Figure 4 shows the results of the undrained cyclic triaxial tests on the lower-layer samples obtained in the second sampling. The stress ratios required to reach DA = 2% at Nc = 20 range from 0.40 to 0.60 which is slightly higher than the results shown in Fig. 3(b).

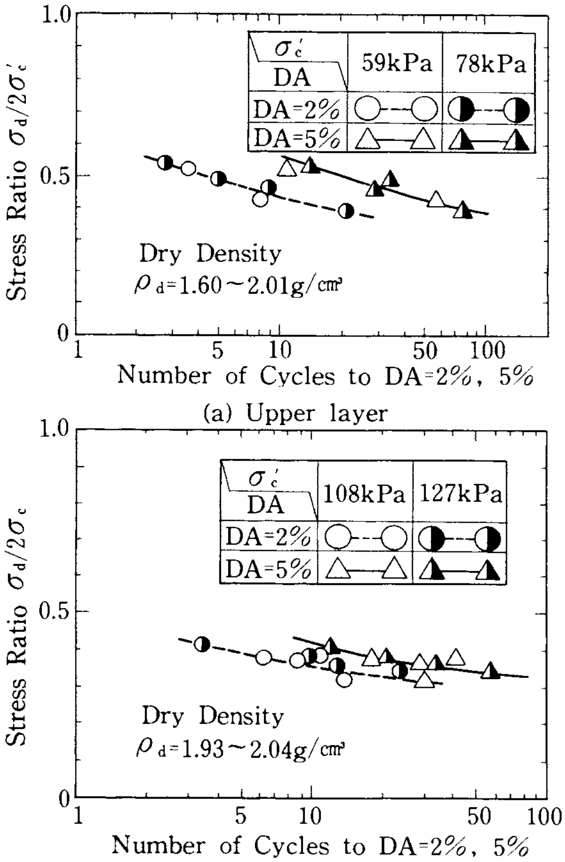


Fig. 3 Dynamic strength of Site A gravelly soils sampled by in-situ freeze sampling

Site K

The gravelly soil at this site is called the Tachikawa Gravel Layer and is a dense well-graded 20,000 year old gravelly deposit. The soil profile is shown in Fig. 5. The sampling depth was 6.0 m to 8.1 m from the ground surface. The penetraion resistance, N-value, of the standard penetration test (SPT) was from 110 to 140. 300 mm diameter samples were obtained by in-situ freeze sampling. The groundwater level was 6.7 m from the ground surface.

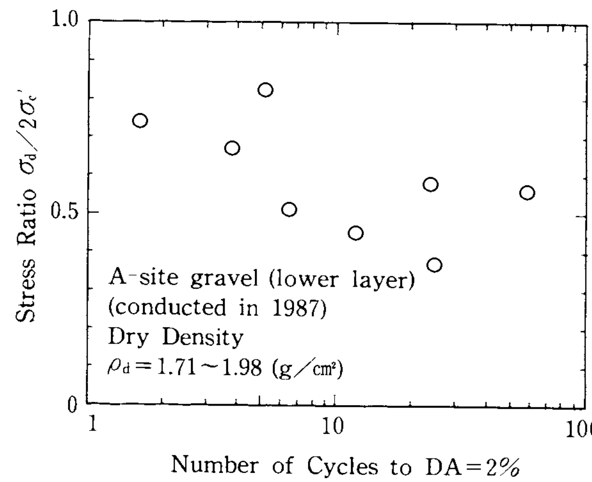


Fig. 4 Dynamic strength of Site A gravelly soils sampled by in-situ freeze sampling (Second sampling conducted in 1987)

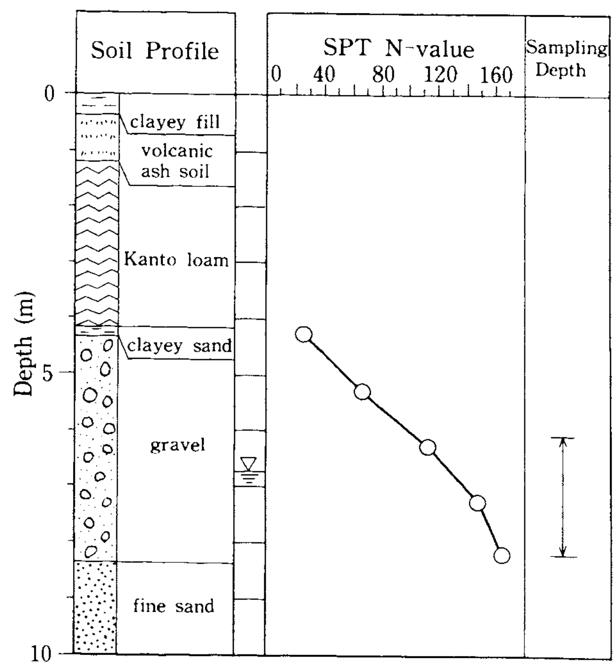


Fig. 5 Soil profile at Site K

The grain size distributions shown in Fig. 6 indicates that the maximum grain size is about 100 to 150 mm and that the fines content is less than 2.0%. The stress ratios for the double axial strain amplitude DA = 2% and 5% in 20 loading cycles measured on the undisturbed samples are 1.13 and 1.50, respectively, as shown in Fig. 7.

<u>Site T</u>
The soil profile at site T is shown in Fig. 8. The age of this gravelly soil deposit was estimated to be about 30,000 years by radio-carbon dating of wooden chips. The sample depth was 10.4 m to 19.1 m from the ground surface. The LPT penetration resistance, N_d-value, was about 30 for the upper-layer (GL -10.4 m to -15.5 m), and about 25 for the lower-layer (GL -15.5 m to -19.1 m). 300 mm diameter samples were obtained by in-situ freeze sampling. The groundwater level was 3.0 m from the ground surface. As Fig. 9 shows, the maximum grain size was about 50 to 200 mm and the fines contents of the upper and lower layers were less than 5.0% and 8.1%, respectively.

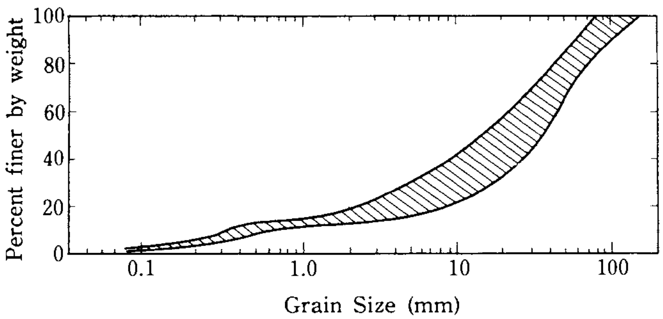


Fig. 6 Typical grain size distributions of Site K gravelly soils sampled by in-situ freeze sampling

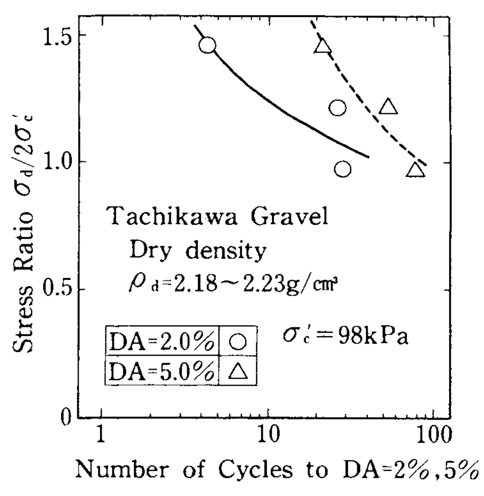


Fig. 7 Dynamic strength of Site K gravelly soils sampled by in-situ freeze sampling

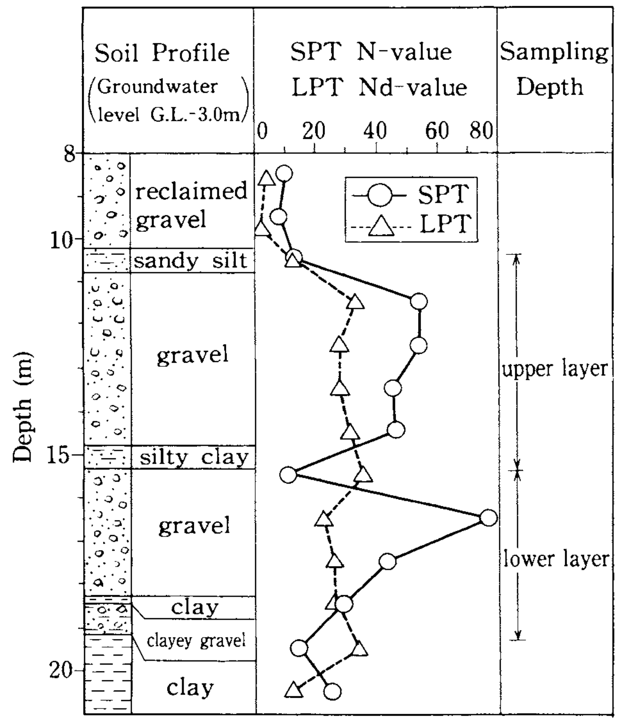


Fig. 8 Soil profile at Site T

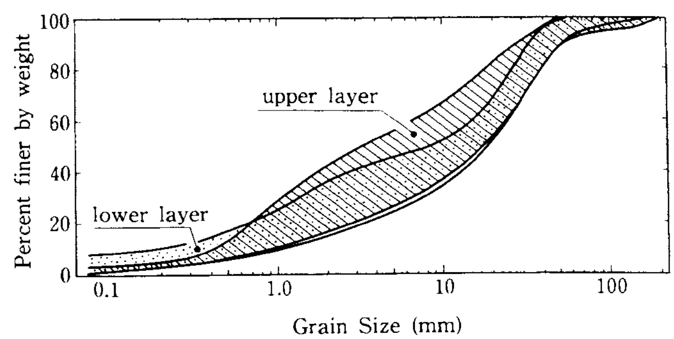
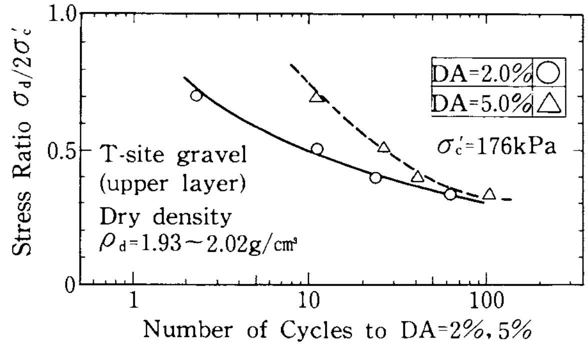
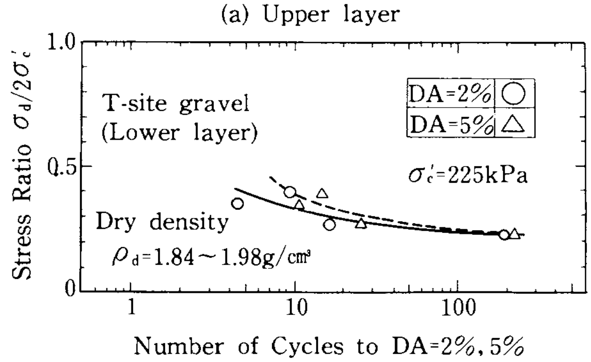


Fig. 9 Typical grain size distributions of Site T gravelly soils sampled by in-situ freeze sampling

Figure 10 shows the results of the undrained cyclic triaxial tests. For the upper-layer samples, the stress ratios for the double axial strain amplitude DA = 2% and 5% in 20 loading cycles were 0.42 and 0.56, respectively. Whereas, for the lower-layer samples, the same stress ratios were 0.29 and 0.33, respectively. The results of the undrained cyclic triaxial tests, in which the upper-layer samples were consolidated under an isotropic stress of 441 kPa and which is higher than the effective overburden pressure, are shown in Fig. 11. The same stress ratios in this case were 0.31 and 0.35, respectively. Both the test results of σ'_c = 176 kPa and 441 kPa are plotted on the same graph shown as Fig. 12 The stress ratio seems to decrease as the confining pressure increases. This trend has also been noted for undisturbed dense Niigata sand by Yoshimi et al. (1984).



(a) Upper layer



(b) Lower layer

Fig. 10 Dynamic strength of Site T gravelly soils sampled by in-situ freeze sampling

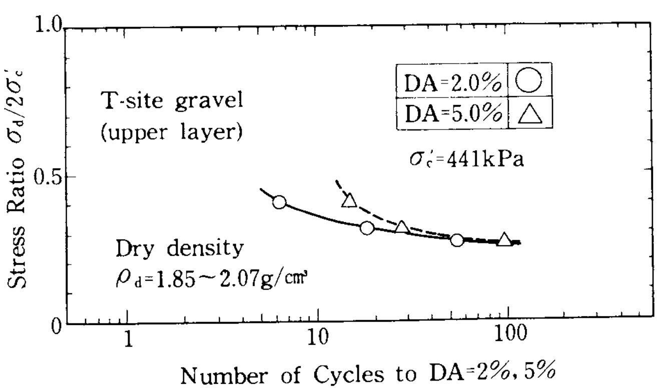


Fig. 11 Dynamic strength of Site T gravelly soils sampled by in-situ freeze sampling (for large confining pressure)

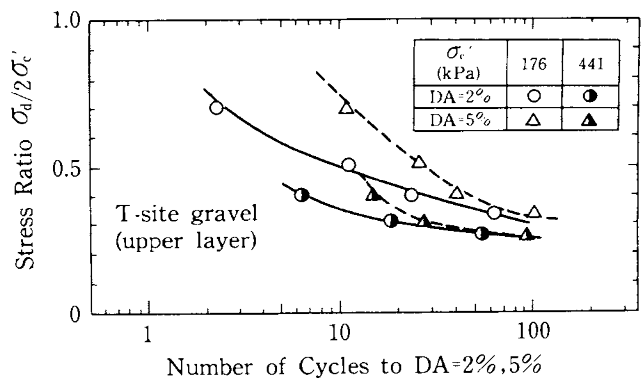


Fig. 12 The effects of confining pressure on the dynamic strengths of Site T gravelly soils

Site KJ

The soil profile at site KJ is shown in Fig. 13. This gravelly deposit of diluvial origin was investigated at depths of 15.6 m to 17.5 m from the ground surface. The LPT penetration resistance, N_d-value, of the gravelly layer was 100. 300 mm diameter samples were obtained by in-situ freeze sampling. The groundwater level was 1.1 m from the ground surface. As Fig. 14 shows, the maximum grain size was about 40 to

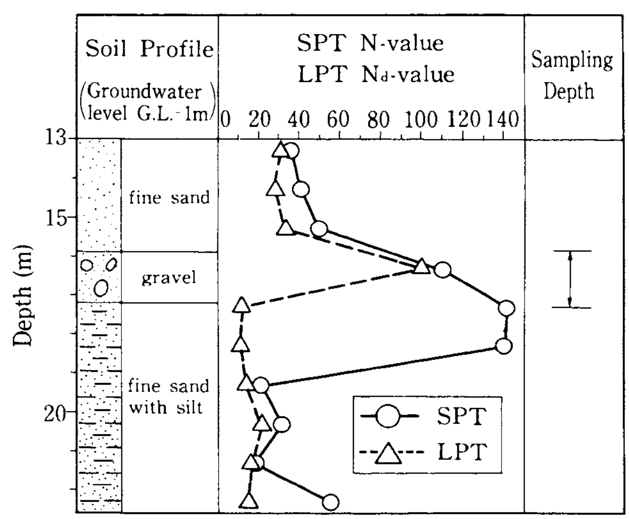


Fig. 13 Soil profile at Site KJ

60 mm and the fines content was less than 3.9%. Figure 15 shows, the results of the undrained cyclic triaxial tests. The stress ratios for the double axial strain amplitude DA = 2% and 5% in 20 loading cycles were 1.27 and 1.40, respectively.

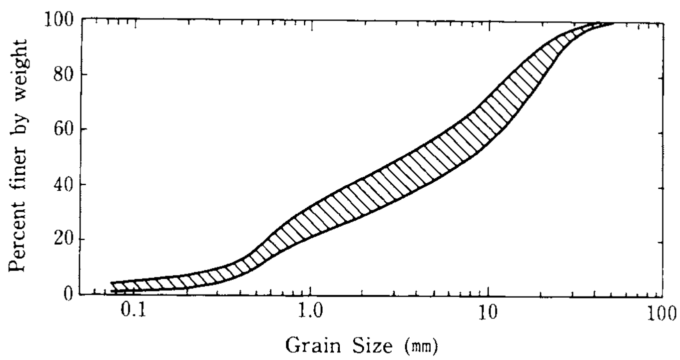


Fig. 14 Typical grain size distributions of Site KJ gravelly soils sampled by in-situ freeze sampling

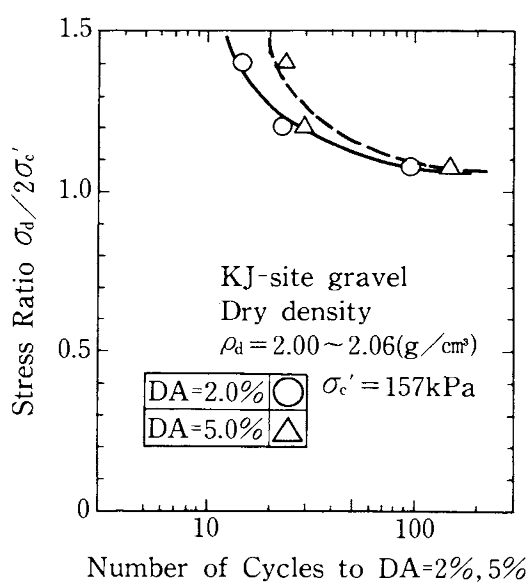


Fig. 15 Dynamic strength of Site KJ gravelly soils sampled by in-situ freeze sampling

RELATIONSHIP BETWEEN DYNAMIC STRENGTH AND PENETRATION RESISTANCE

EMPIRICAL CORRELATIONS BASED ON SPT AND LPT

As described previously, we conducted a series of field and laboratory tests, including undrained triaxial tests on undisturbed samples and penetration tests on the same soils. Several similar tests have been also conducted recently by other researchers on some kinds of gravelly soil deposits. Table 1 summarizes the results of all these field tests in which in-situ freeze sampling as well as penetration tests at nearby boreholes were carried out.

Figure 16 represents the relationships between the dynamic strengths and the penetration resistance. The vertical axis corresponds to the dynamic strength in terms of the stress ratio for DA = 2% or 2.5% in 20 loading cycles ; the horizontal axis is to the modified SPT blow count, N_1. N-value was modified in terms of the effective overburden pressure to give N_1 according to the equation

$$N_1 = N/(\sigma_v'/P_1)^{0.5}, \quad P_1 = 98\,kPa \qquad (1)$$

proposed by Liao et al. (1986). The effective confining pressures, σ_c', of 69 to 127 kPa in the triaxial tests on these gravels, were all assumed to equal the in-situ effective overburden pressures. Similar data for sands

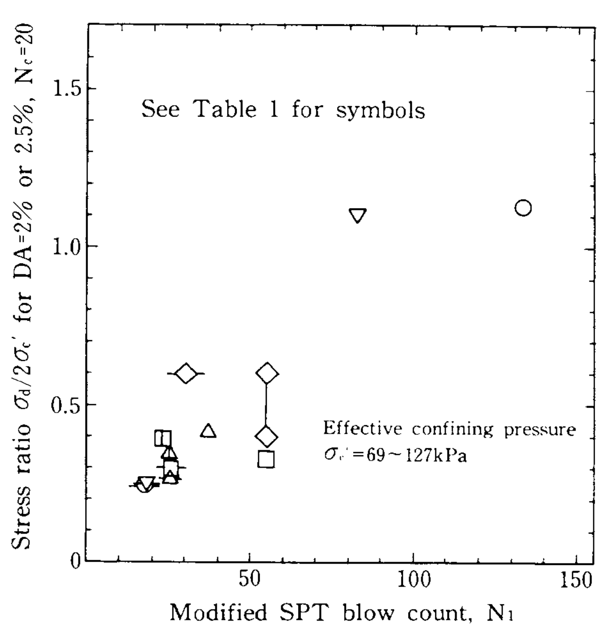


Fig. 16 Dynamic strength vs. SPT N_1-value (σ_c' = 69 ~ 127 kPa)

Table 1. Symbols in Figs. 16, 17 and 18

Soils	Soil sample description		Refferences	σ_c' kPa	Symbols
Gravelly Soils	Site K		This investigation	98	○
	Site T	upper layer		176	◭
		lover layer		225	◨
	Site KJ			157	▼
	Site A (conducted in 1984)	upper layer		69	△
		lower layer		118	□
	Site A (conducted in 1987)	lower layer		98	◇
	Mandano Gravel		Tamaoki, et al. (1986) Goto, et al. (1987)	118	▽
	Tokyo Gravel		Hatanaka, et al. (1986) Hatanaka, et al. (1988) (1989b)	294	●
	Samples obtained near the Tonegawa river	upper layer	Nishio, et al. (1989)	78	◬
		lower layer	Hatanaka, et al. (1989a)	127	◫
Sands	Niigata sand		Yoshimi, et al. (1984)	98	-◇-
	Meike site	C_1	Yoshimi, et al. (1989)	78	-○-
		C_2		88	-△-
		C_3		98	-□-
	Showa Bridge site			98	-▽-

obtained from field and laboratory tests under confining pressures, σ_c', of 78 to 98 kPa are plotted on the same graph. Although these plots indicate a correlation between the dynamic strength and N_1 for gravelly soils, the scatter of the data is rather large. This may be attributed partially to problems associated with the Standard Penetration Test in gravelly soils where the penetration resistance is sensitive to grain size. Another problem is that the size of the test probe and the driving energy are insufficient to match the penetration resistance of gravelly soils.

To overcome these difficulties, the Large Penetration Test is used occasionally in Japan to investigate gravelly deposits. Figure 17 plots the dynamic strengths on the vertical axis against the modified penetration resistances, N_{d1}, of the Large Penetration Test on the horizontal axis instead of N_1. At some points in this graph where LPT data were not available, N-value was converted based on the empirical relationship (Yoshida et al., 1988). N_{d1} was derived for the effective overburden pressure, σ_v', of 98 kPa following the same equation used for the N-value modification. Unlike the SPT-based correlation shown in Fig. 16, the LPT-based correlation appears to be more consistent between gravels and sands and also between different gravels. This may indicate that the Large Penetration Test is less sensitive to increaseing grain size and is more powerful than the Standard Peneration Test and is thus better suited to investigation of gravelly soils.

THE EFFECT OF CONFINING PRESSURE

As mentioned previously, Figure 12 shows that the dynamic strength of undisturbed gravelly soil samples decreases as the effective confining pressure σ_c' increases beyond the effective overburden pressure. Since the samples, the test results of which are shown in Fig. 12, are taken from the same soil layer, the modified LPT blow counts N_{d1} of the samples are the same. This indicates that under different confining pressures no unique relationship between the modified LPT blow count N_{d1} and the dynamic strength exists any more. Since the relationship between the modified LPT blow count and the dynamic strength is affected by the effective confining pressure, the establishment of this relationship must be considered for the same effective confining pressure.

The plots in Fig. 17 represent data for the effective confining pressure, σ_c' of 67 kPa to 127 kPa. As previously noted, the relationship between the dynamic strength and the modified LPT blow count, N_{d1}, will be influenced for dense sandy and gravelly soils by the effective confining pressure. This suggests that the in-

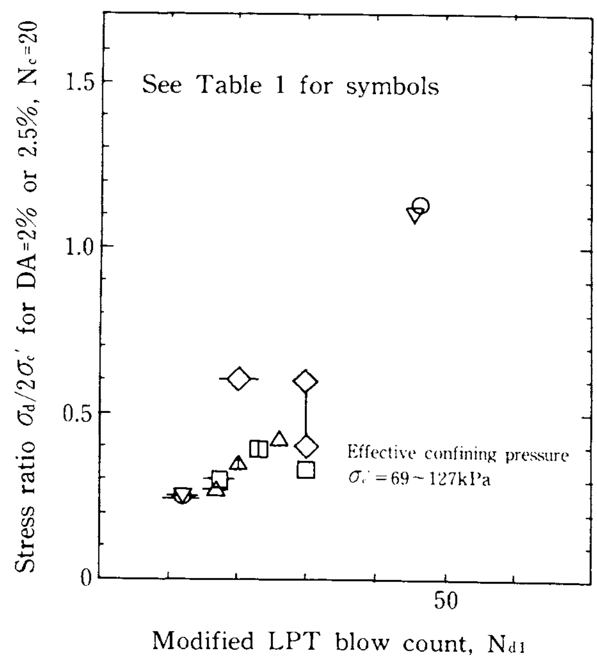


Fig. 17 Dynamic strength vs. LPT N_{d1}-value (σ_c' = 69 ~ 127 kPa)

situ dynamic strength of dense soils may be overestimated if the relationship in Fig. 17 is applied to deeper layers. It is therefore necessary to estimate the variations in the relationship caused by changes in the effective confining pressure.

Based on a hypothesis that the dynamic strength of sand is closely related to the shape of the effective stress path in static undrained triaxial compression tests, Imai et al. (1975) demonstrated that the relationships between the dynamic strength and negative dilatancy characteristics are not affected by the type of sand or relative density. Assuming that Imai's hypothesis holds for the shape of the stress path in static undrained triaxial compression tests of gravelly soils under a different magnitude of effective confining pressure, the effect of the confining pressure on the dynamic strength can be estimated. A final formula that can estimate the effect quantitatively is expressed in Eq. (2), the derivation of which is available in other literature (Tanaka et al., 1988, Tanaka et al., 1990).

$$\sigma_d/2\sigma_c' = f\{(\frac{\sigma_c'}{P_1})^{-\alpha} \cdot N_{d1}\} \qquad (2)$$

$\sigma_d/2\sigma_c'$ in Eq. (2) denotes the dynamic strength. f( ) is a function of the LPT blow count and the effective confining pressure. P_1 is equal to 98 kPa. N_{d1} is the modified LPT blow count, and α is a constant determined from the confining pressure dependency on the shear modulus, the rebound modulus and the penetration resistance of gravelly soils (Tanaka et al., 1988, Tanaka et al., 1990).

Eq. (2) indicates that both the increase in σ_c' by Q times and the decrease in N_{d1} by Q^{α} times have the same effects on the dynamic strength. On the basis of experimental data from natural deposits, the constant α can be approximated by 0.5. By using Eq. (1), Eq. (2) with α = 0.5 can be rewritten as follows:

$$\sigma_d/2\sigma_c' = f\{(\frac{\sigma_c'}{P_1})^{-1} \cdot N_d\} \qquad (3)$$

Eq. (3) means that the dynamic strength of gravelly soils was solely dependent on the value of $N_d/(\sigma_c'/P_1)$. Thus the dynamic strengths of the soils listed in Table 1 are all plotted in Fig. 18 against $N_d/(\sigma_c'/P_1)$. It may be possible to draw an empirical curve as the solid line shown in Fig. 18. The curve in Fig. 18 is expressed by the following equation.

$$\sigma_d/2\sigma_c' = 0.15 + 0.0059((\frac{\sigma_c'}{P_1})^{-1} \cdot N_d)^{1.3} \qquad (4)$$

Fig. 18 indicates that the equation (4) can be used as a powerful tool for estimating in-situ dynamic strengths of dense gravelly soils for a wide range of effective confining pressures based on the penetration test.

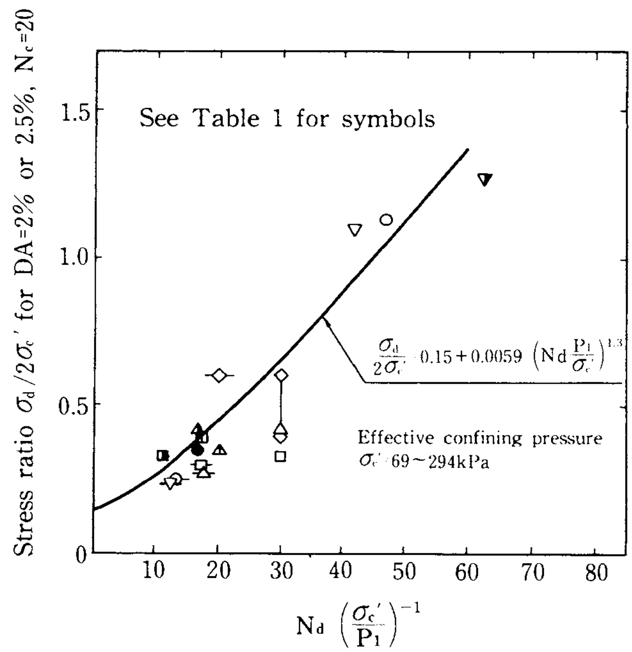


Fig. 18 Dynamic strength vs. $N_d(\sigma_c'/P_a)^{-1}$ (σ_c' = 69 ~ 294 kPa)

CONCLUSIONS

On the grounds that reliable in-situ dynamic strengths of dense gravelly soils without significant disturbance can be evaluated by in-situ freeze sampling, a series of field and laboratory tests, including undrained triaxial tests on undisturbed samples and penetration tests on the same soils, was conducted to establish an empirical correlation for estimating in-situ dynamic strengths of gravelly soils from penetration resistance. Of the two penetration tests, the LPT was clearly more applicable than the SPT to dense gravelly soils. Also, the effective confining pressure was found to be deeply involved in the correlation. It was shown that the proposed empirical correlation can satisfactorily evaluate the dynamic strength based on the penetration resistance.

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