AN EXPLORATORY METHOD FOR DYNAMIC PROPERTIES OF GROUND THROUGH BOREHOLE WALL

by

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SYNOPSIS

To investigate the dynamic properties of ground, authors have developed a new equipment for in-situ test, which can generate dynamic pressure on borehole wall through rubber tube at arbitrary depth with a wide range of radial strain amplitude.

The following informations are mentioned here, (a) mechanism and method of the equipment that is dynamic ground modulus exploratory equipment, (b) some test results with this equipment which are the dynamic deformation modulus (Ed), the equivalent viscous damping coefficient (*he*) and the horizontal dynamic coefficient of soil reaction (*Kh*).

INTRODUCTION

Recently, the new pressuremeter attached the self-drilling instrument inside of the shoe which is capable of insertion into the ground with the minimum disturbance has been invented by Wroth et al (1973) and Beguelin et al (1973) and the new theories of in-situ determination of undrained stress strain behavior of sensitive clays with the pressuremeter were proposed by Ladany (1972), Palmer (1972) and Wroth (1973).
Inspite of existence of these investigations for static pressuremeter authors noticed of the accurate dynamic ground coeffcient for the earthquake resistant design of the foundation by seismological observation of the foundation at the Matushiro earthquake swarm (1966). Thus authors have tried to develop the in-situ exploratory equipment of ground through borehole wall.

MECHANISM AND METHOD OF THE EQUIPMENT

The equipment is coposed of dynamic loading part through the rubber tube, a driving and controling part adjoining the loading part, an oil power suply, a load setting instrument and a recoder part, as shown in Fig.1 and 2. The driving and controlling part is composed of a piston cylinder and a servo valbe.

The performance of the equipment is illustrated in table 1. *Kh*, Ed and "*he*" are defined by the loop of pressure and displacement of the wall. Namely *Kh*, Ed and "*he*" are given by follwing equations.

$$Kh = \frac{r}{p}\ ,\ \mathrm{Ed} = (1+\nu)\cdot ro\cdot Kh,\ \text{"}he\text{"} = \frac{1}{2\pi}\,\frac{\text{energy loss of one cycle}}{\text{elastic energy of one cycle}}$$

The dynamic loading is generated by repeated driving of the piston cylinder into and out of the cell of the rubber tube, the movement of the cylinder is controled by the opening and closing of the servo valbe. Before the dynamic loading tests at certain depth, the static tests are performed right above the test point, and from the test result the pressure-displacement curves, static shearing strength and in-situ lateral earth pressure which is often defined to be the base pressure at the dynamic loading test, are obtained. On the otherhand, the approximate shearing stress distribution in the ground during the earthquake which corresponded to the design seismic coefficient at each site is caluculated.

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The dynamic loading tests by the stress control system are performed under 65, 100, 200,300%.....of the shearing stress amplitude during earthquake. The repeated loading numbers are about 200, and repeated velocity of the loading is determined to be same to predominant period of the ground surface. As the length of the loading part incomparison with its diameter is very long, the stress-strain condition of the ground around the loading part is considered to be the two-dimentional state of stress in polar coordinates. Then the radial strain εr,the tangential strain $\varepsilon\theta$, and the axial strain εz are shown respectively by the following equations.

$$\varepsilon r = \frac{du}{dr}\ ,\ \varepsilon\theta = \frac{u}{r}\ ,\ \varepsilon z = 0\ .$$

Where u is the displacement of the radial component at a distance r from the center.
Since the drainage condition during the dynamic loading corresponds that of the undrained shearing test in laboratory, it is considered that the volume change of the surrounding ground of the loading part is negligible small. Hence the radial strain at the borehole wall is given by the following equations

$$\varepsilon ro + \varepsilon\theta o + \varepsilon zo = 0,\quad \varepsilon ro = -\varepsilon\theta o = -\frac{\Delta ro}{ro}$$

where Δro is the displacement of the radial component at the borehle wall and ro is the radius of the rubber tube before the dynamic loading.

RESULTS OF DYNAMIC LOADING TEST

THE RESULTS AT SITE "D"; The ground structure of site "D" is made up of multi laminated alluvial deposits of the flood plain deposits. The soil from surface to GL-5m is coarse sand of which the standard N-value is under ten, and soil between GL-5m and GL-8m is the fine sand which is mixed with pumice and laminated silt. When the relation between Ed and εro is plotted on logarithmic graph, a non-loear relation which is shown in Fig.3, is obtained. The figure indicates that the more the εro become, the less the Ed suddenly.

When the relations between "*he*" and εro are plotted on the semilogarithmic graph, the nonlinear relation showing that the more εro become, the more "*he*" become as shown in Fig.5-a, are obtained.
By the comparison of the test result by this equipment with the exciting test result for piles at same point of site "D", *Kh* with this equipment is slightly larger than *Kh* by the exciting test, but the relation between *Kh* and displacement amplitude is very similar each other as in Fig.4, where *Kh* with this equipment is measured at the depth $(1/\beta)$ of Chang's theory and is adjusted by the following equation. Thus equation is proposed by Yoshinaka (1968) to apply the test result of static pressuremeter to the design of horizontal bearing of pile.

$$Kh = \frac{1}{1.2}\ Ed\ \frac{1}{\sqrt[4]{B^3}}$$

where B is the diameter of pile.

On the contrary, "*he*" with this equipment is smaller than "*he*" by the exciting as in Fig.5-a and 5-b, futher the damping ratio by the seismic prospecting method is about 5%.

THE RESULTS AT SITE "S"; The undulation of the bedrock of mudstone is very extream at this site and the thickness of the formation above it extends from 10m to 30m. This formation is made up of the dredged fill from surface to GL-4m, and the alluvial deposit under it. The object of the test was to investigate the effects of improvement of the ground were performed at three points of the site "S". The first point was treated by the vibrocomposer, the second point was non-treated about two years after dredging,and the third point was at the ground immediatly after dredging.

The effects of improvement of the ground are conspicuous by the static *Kh* and N-value as shown in Fig.6. The relations of these ground between Ed and εro, are showin in Fig.7, and the figures in the curves show the accelarations during the earthquake which correspond to the shearing stress amplitude of the dynamic loading test. In comparison with the test result at same depth, it is appearent that the static and dynamic deformation moduli of the improved ground are the largest among these three points as shown in Fig.7-a, and then the effect of improvement is conspicuous by Ed too. Regarding "*he*" the nonlinear relations which were the same to the results of the test of site "D" were obtained as in Fig.7-b, and futher the damping ratio by the seismic prospecting method were about 5% at this site too.
Fig.8 show influence of load cycle number on Ed and "*he*".
It is shown in Fig.8 that Ed of soft silt becomes suddenly small prior to the first 20 cycles but becomes constant after that time and "*he*" becomes suddenly large before the first 20 cycles but becomes constant after that time. It is also shown that the values of Ed and "*he*" of comparatively hard ground, however, are almost constant from the first to the end under the condition of the small stress amplitude, and of course for any ground, Ed becomes suddenly small and "*he*" becomes large at the stress amplitude which results in large residual deformation.

THE DYNAMIC CHARACTORS OF GROUND BY THE IN-SITU TEST

The deformation modulus and the damping of soil specimen have been investigating by Hardin-Drinevich, Parker Silver, Seed-Chan-Kovacs et al with each dynamic shearing equipment and it is already indicated that strain amplitude, mean principal stress (confined stress) and loading cycles give large influence on these physical properties of soil specimen, but angle of internal friction, void ratio, coeffcient of earth pressure at rest and degree of saturation do little influence on them.
Also Seed et al have collected and arranged the same experimental results and have completed the figures, which give the two relations between G and shearing strain amplitude, and between "*he*" and shearing strain amplitude respectively are plotted on semilogarithmic graph.
On the other hand Osaki-Iwasaki et al have mentioned that Ed or G which are obtained by the seismic propecting method, are relative to the standerd N-value.

INFLUENCE FACTORS ON THE DAMPING OF GROUND

As the test ground is complex structure which is composed of alluvial deposit and dredged fill, it is not a simple condition to draw a diagram with parameter of the above stated influence factors. But it was already mentioned that the influence of strain amplitude on the damping was very large and the influence of N-value on it was small as shown Fig.7-b.
It was also mentioned that the influence of load cycles from 1 to 20 on the damping of soft ground was very large as shown in Fig.8.
In order to get the influence of the depth or confined stress on "*he*", the relations between "*he*" and εr are plotted with the parameter of the depth as shown in Fig.9. Thus it is shown that the more the depth, the less the "*he*" at same strain amplitude.
On the other hand in order to clarify the difference of "*he*" due to kinds of soil, the relations between the "*he*" and "εr" of silt and fine sand are plotted respectively as shown in Fig.10, which shows that the "*he*" of fine sand is larger than that of silt,and is consistent with the result by Seed

INFLUENCE FACTORS ON THE DYNAMIC MODULUS OF GROUND

It is shown in Fig.3 and 7 that εr, N-value and the load cycles on Ed were very large.

On the other hand, in order to compare with the relation between Ed and N-value which was already mentioned by Osaki-Iwasaki, Ed and N-value are plotted on the logarithmic graph together with Osaki's experimental equation and the equation by static pressuremeter test. Thus the linear relations of which gradients are similar to those of the above stated equations are obtained as shown in Fig.11.

CONCLUSION

From the test results on alluvial deposit and dredged soil the following conclusions are drawn:

(1) The test result by this equipment is compared with that of exciting test on steel piles.

Kh by the former is found to be slighty larger than the latter, while the relation between *Kh* and amplitude of displacement are very much similar each other. On the contrary "*he*" by the former found to be smaller than that of the vibration test.

(2) Acording to the test results on site "S", effects of improvement on the ground are remarkable from the standpoints of Ed, N-value and static bearing capacity, however, are not from that of "*he*".

(3) The dynamic characteristics of the ground are consistent with those by Seed and other's.

Namely, regarding Ed, a nonliear relation is obtained if the relation between Ed and εr is plotted on lagarithmic graph, and also a linear relation is obtained if N-value and Ed are plotted on Semi-logarithmic graph by using parameter of εr. The latter relation between N-value and Ed is similar to those of seismic prospecting and the pressuremeter test.

As to the "*he*" it is influenced by depth but not by N-value.

(4) By above mentioned results of test, it is concluded that this equipment is able to put practical use for in-situ test.

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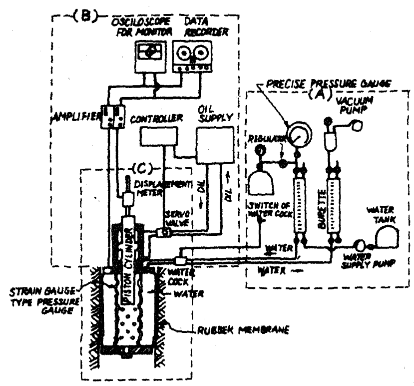

Fig.1 DYNAMIC GROUND MODULUS MEASUREMENT EQUIPMENT MECHANSM

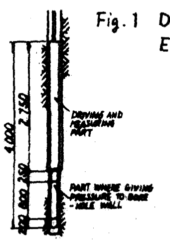

Fig.2. SIZE AND FORM OF PART WHERE GIVING PRESSURE TO BOREHOLE WALL

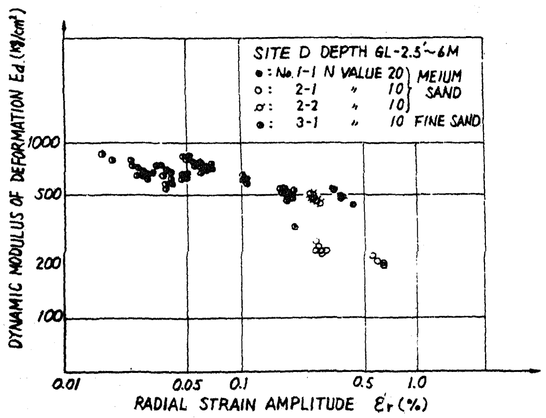

Fig.3 THE RELATION BETWEEN E_d AND RADIAL STRAIN AMPLITUDE ε_r

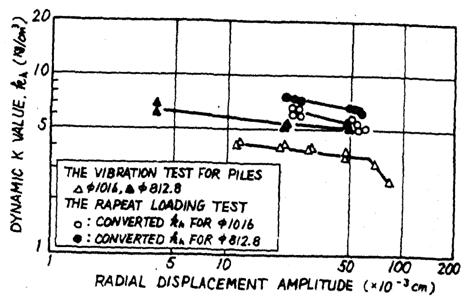

Fig. 4 COMPARISION BETWEEN CONVERTED k_h BY THE REPEATLODING TEST AND k_h BY THE VIBRATION TEST FOR PILES AT SAITE D

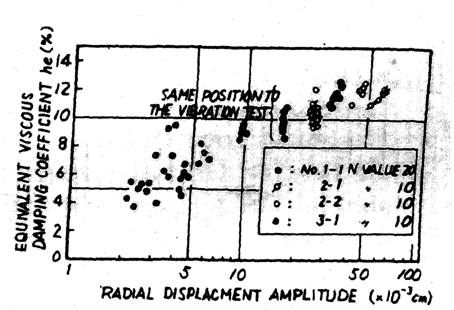

Fig.5-a THE RERATION BETWEEN h_e AND RADIAL DISPLACEMENT AMPLITUDE BY THE REPEAT LOADING TEST FOR BOREHOLE WALL AT SAITED

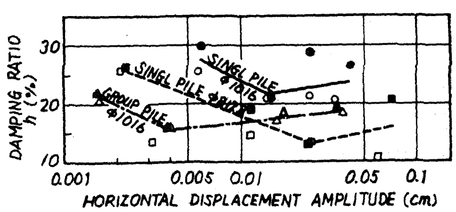

Fig.6-b THE RELATION BETWEEN h AND DISPLACEMENT AMPLITUDE BY THE VIBRATION TEST FOR PILES AT SAITE D

TABLE - 1 EQUIPMENT PERFORMANCE

PRESSURE ABILITY (kg/cm^2)	0 ~ 30
KINDS OF PRESSURE WAVES	SIN, TRAIANGLE, RECTANGLE
FREQUENCY ABILITY (Hz)	0.5 ~ 20
MAXIMUM DEPTH (M)	50
SIZE OF RUBBER MEMBRANE (M·M)	L TYPE φ80 x 800 S TYPE φ80 x 400

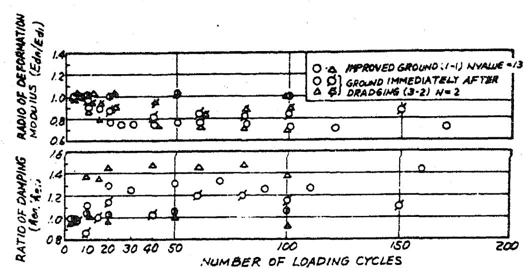

Fig 8 THE CHANGE OF DYNAMIC PROPERTY OF SOIL WITH NUMBER OF CYCLES

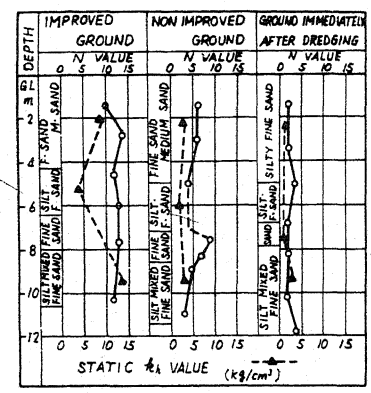

Fig. 6 BORING LOG OF SITE S

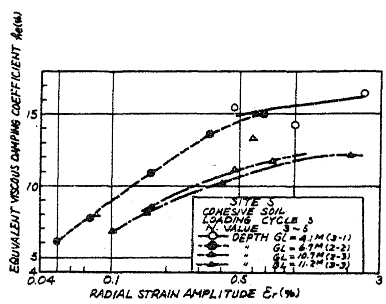

Fig. 9 THE EFFECT OF DEPTH TO THE RELATION BETWEEN h_e AND ε_r

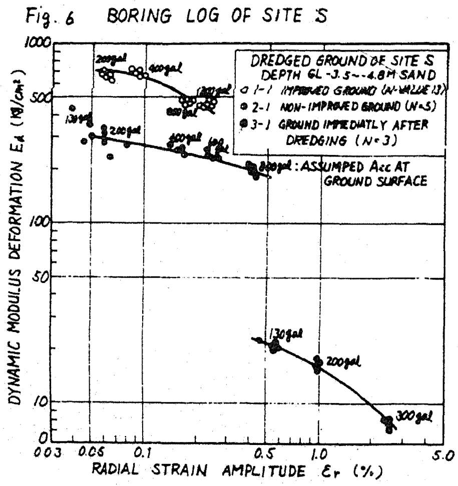

Fig 7-a EFFECT OF IMPROVEMENT ON GROUND

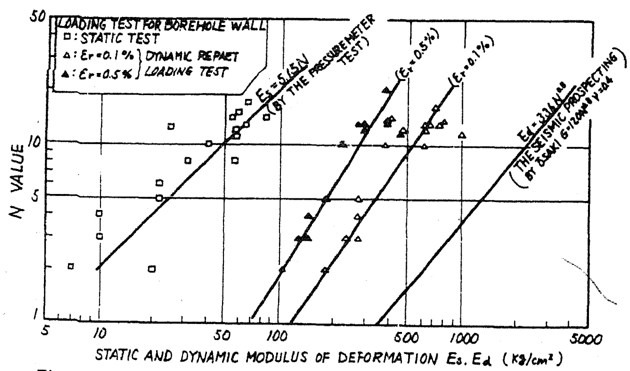

Fig 11 THE RELATION BETWEEN N VALUE AND MODULUS OF DEFORMATION

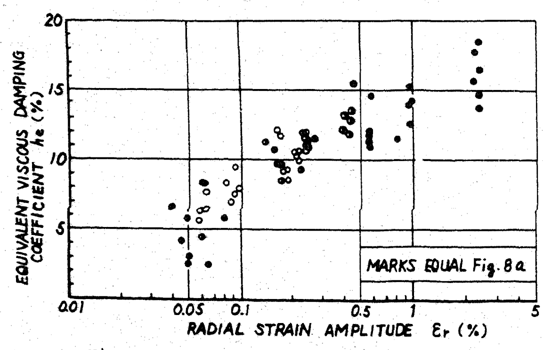

Fig. 7-b EFFECT OF IMPROVEMENT ON GROUND

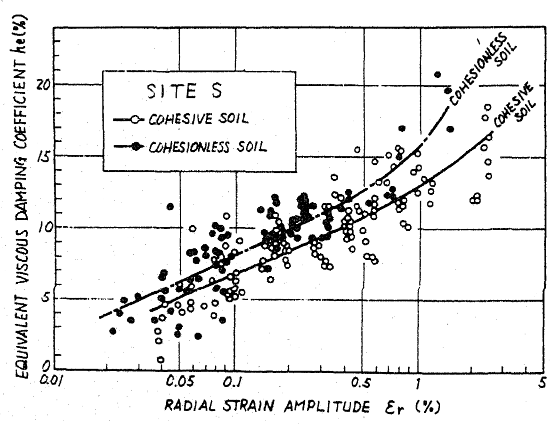

Fig 10 THE RELATION BETWEEN h_e AND RADIAL STRAIN ε_r AT SITES