SOILS AND FOUNDATIONS Vol. 14, No. 3, Sept. 1974
Japanese Society of Soil Mechanics and Foundation Engineering

SHEAR MODULUS AND SHEAR STRENGTH OF COHESIVE SOILS

Akio Hara*, Tokiharu Ohta**, Masanori Niwa*,
Shumpei Tanaka* and Tadashi Banno***

ABSTRACT

The purpose of this paper is to evaluate the relation between initial shear modulus and shear strength of cohesive soils. The initial shear moduli were obtained from the results of the well-shooting tests by means of shear waves, while the shear strength could be obtained from the results of laboratory tests conducted on undisturbed soil samples collected at the same site as the well-shooting tests. Taking into consideration the fact that there have been a number of instances where the well-shooting test and the standard penetration test were conducted on the same sites, the authors conducted their research to seek some relationships between the initial shear modulus and the N-value of the standard penetration test, between the shear strength and the N-value, and between the initial shear modulus and the shear strength. The research works have finally led the authors to finding out several relations among initial shear modulus, shear strength and N-value.

Key words: cohesive soil, elasticity, geophysical exploration, in-situ test, laboratory test, penetration test, shear strength

IGC: D7/D6

INTRODUCTION

In Japanese cities, for lack of suitable building sites, there has recently been a marked tendency to plan construction of large and tall buildings on soft grounds which were never considered in the past as a building site. Every structure is influenced by the local ground behavior to a considerable extent during earthquakes, especially when built on a soft ground. Since any structure built on such a ground is inevitably subject to considerable displacement, particularly at its foundation, it is desirable that an earthquake-proof design follows a dynamic analysis by means of a simulation model of structure-foundation-ground system. Needless to mention, understanding by experiment of the dynamic properties of soils is required for such a dynamic analysis. These properties are characterized by the following items :

(1) Initial shear modulus
(2) Shear strength
(3) Non-linear characteristics
(4) Damping characteristics

* Research Engineers of Kajima Institute of Construction Technology, 19-1, 2-Chome, Tobitakyu, Chofu-Shi, Tokyo.
** Senior Research Engineer of Kajima Institute of Construction Technology, 19-1, 2-Chome, Tobitakyu, Chofu-Shi, Tokyo.
*** Assistant Research Engineer of Kajima Institute of Construction Technology, 19-1, 2-Chome, Tobitakyu, Chofu-Shi, Tokyo.
Written discussions on this paper should be submitted before July 1, 1975.

In order to research the above problems, it is desirable to make the best of the merits of the in-situ test and the laboratory test. By in-situ tests we are able to grasp directly the mechanical properties of soils at natural soil deposit, but the test conditions are not always clear. On the other hand, conditions of the laboratory test are clear, and the laboratory test enables us to obtain the mechanical properties at large deformation, but its weak point is disturbance of soil structure during the sampling and testing processes.

Therefore the in-situ and the laboratory tests have their merits and demerits. In order to take advantage of the merits, the authors took notice of the following items :

(1) There are many cases where soil stiffness increases in general with strength according to the experience of laboratory or in-situ test.

(2) Wilson and Dietrich[1] presented many data on Young's modulus E and undrained shear strength S_u by the laboratory test. Furthermore, D'Apporonia et al.[2] showed a number of data on Young's modulus E obtained from initial settlement curve and undrained shear strength S_u. It is recognized that the values of ratio E/S_u calculated from their data are within a certain limited region.

(3) The shear strength of soil can be calculated from the values of effective overburden pressure, angle of internal friction and cohesion, and the angle of internal friction depends upon the value of plasticity index, that is, it is affected by a number of factors. Furthermore, both dimensions of initial shear modulus and of shear strength are same. It is not without reason, therefore, to consider that initial shear modulus is closely related to shear strength.

The authors have tried, from the above mentioned reasons, to review the relation between initial shear modulus and shear strength of cohesive soils on the basis of the results of their own in-situ test (well-shooting test, standard penetration test) and laboratory test (triaxial compression test).

INITIAL SHEAR MODULUS OF SOILS

Well-Shooting Method

Miller et al.[3] show that the geophysical exploration test comprises several types of practical methods (cross-hole, down-hole, up-hole, surface refraction and surface-vibration mothods).

The down-hole method is to generate shear waves by hitting horizontally a wooden plate placed on the ground surface, so that velocity transducers clamped in a borehole receive such shear waves. To say more precisely, several velocity transducers are concurrently clamped in the borehole, which allow velocity measurement of a shear wave passing through a section between one transducer and the another.

When the shear wave velocity V_S is measured, shear modulus G can be calculated by the following equation :

$$G=\rho V^2{}_s(\mathrm{kg/cm^2}) \quad (1)$$

where ρ : mass density.

This is the procedure called the well-shooting test by means of shear waves, on which Kitsunezaki[4], Shima et al.[5] and Imai et al.[6] have already published their respective reseach papers.

Shear Strain Amplitude in Soil Deposits

Initial shear modulus can be defined as shear modulus at infinitely small shear strain amplitude. It is therefore necessary to obtain shear strain amplitude in soil deposits when the well-shooting test by means of shear waves is to be conducted.

The result of the well-shooting test which was performed in a soft clay deposit, as an example, is shown in Fig. 1. The authors tried to calculate shear strain amplitude in a soft soil deposit from wave forms shown in Fig. 1 with the following procedure :

(1) Calibration curves of the velocity transducers were firstly determined by shaking table

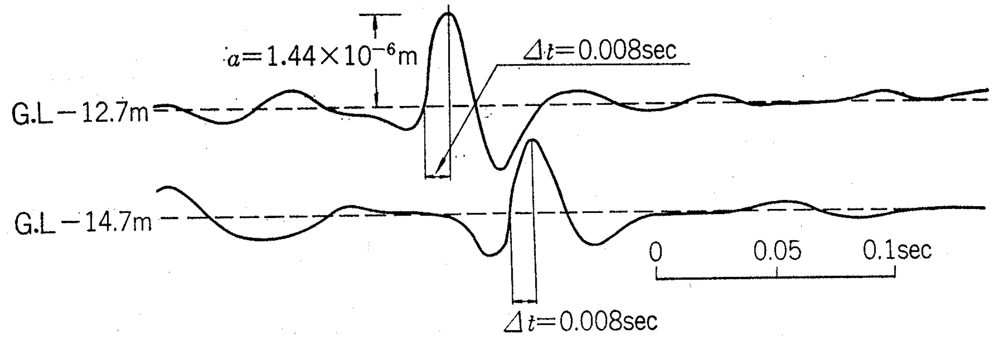

Fig. 1. An example of shear wave velocity measurement

in the laboratory.

(2) On the assumption that principal shear wave should be represented by that of harmonic motion, the maximum shear displacement amplitude Δx was estimated by the use of the calibration curves to be the following value:

$$\Delta x = 0.0000144 \text{ (m)} \qquad (2)$$

(3) On the other hand, thickness of soil deposit ΔH subject to shear displacement of Δx can be calculated as follows:

$$\Delta H = V_s \cdot \Delta t = 90 \times 0.008 = 0.72 \text{(m)} \qquad (3)$$

where V_s: shear wave velocity.

(4) Shear strain amplitude γ can thus be calculated as follows:

$$\gamma = \frac{1.44\times10^{-6}}{0.72} = 2.0\times10^{-6} \qquad (4)$$

Shear strain amplitude obtained in the above procedure are plotted in Fig. 2, which shows that shear strain amplitude thus obtained by the well-shooting test can be assumed to be less than 10^{-3} percent.

In addition, when shear strain amplitude is $10^{-3}\%$, shear stress as calculated below is so small that the shear modulus obtained by the well-shooting test can be regarded as initial shear modulus:

$$\tau = \gamma G = 10^{-5}\times110 = 0.0011 \text{(kg/cm}^2\text{)} \qquad (5)$$

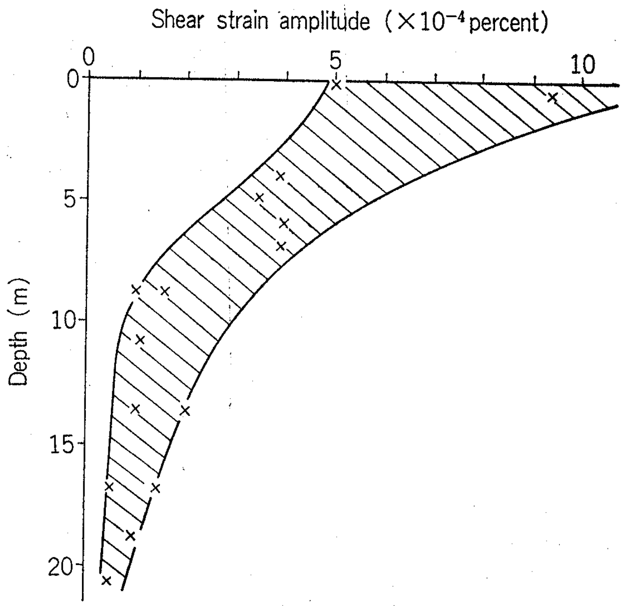

Fig. 2. Estimated shear strain amplitude at various depths

Shear Strength of Soil

The soil element shown in Fig. 3. is deformed due to random vibrations propagating from bedrock to surface during an earthquake. In this case, the deformation excited most significantly is mainly attributable to the shear wave.

The initial stress condition of the soil element is the K_0-condition, which means that the initial effective stresses in vertical and horizontal direction are σ'_v and $K_0\sigma'_v$ respectively.

The shear modulus obtained from the well-shooting test is calculated from the velocity of the shear wave, which propagate through the soil elements vertically in the K_0-condition.

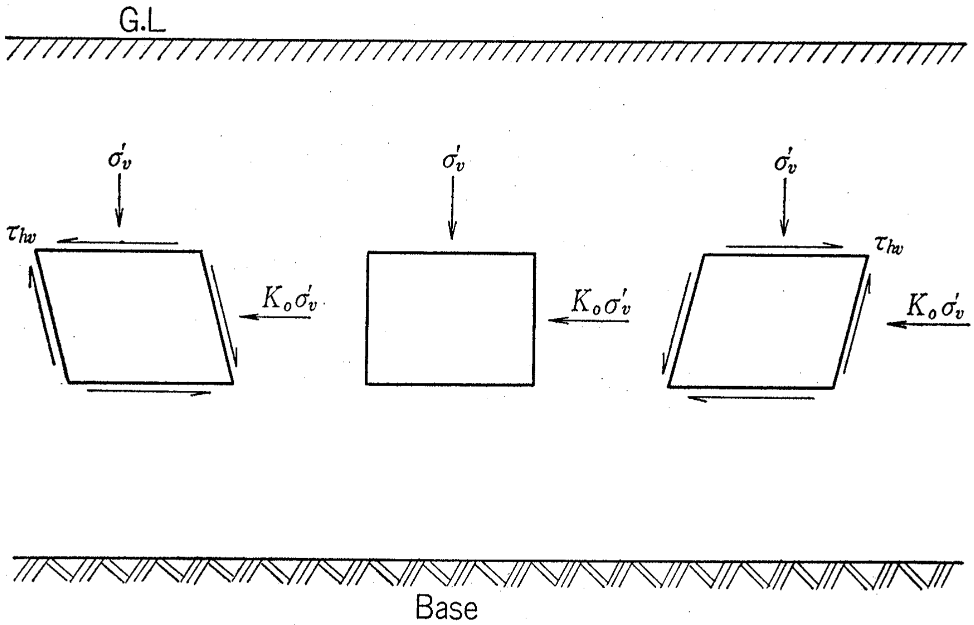

Fig. 3. Conceptual field loading condition

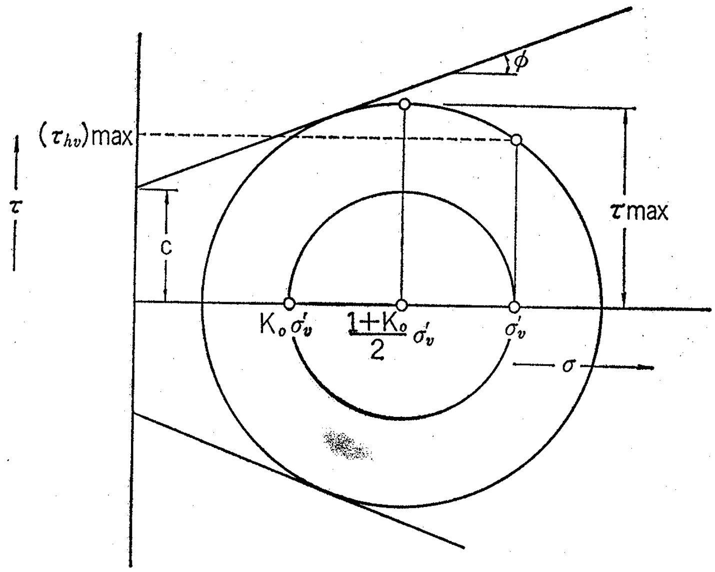

Fig. 4. Mohr's circles and maximum shear stress

Therefore, it is reasonable that the shear strength of a soil deposit should be obtained by taking into account its initial stress condition. These problems have been sufficiently treated by Duncan[7], Seed[8] and Hardin[9]. According to these researches, when the soil element shown in Fig. 3 comes to failure, the maximum lateral shear stress $(\tau_{hv})_{max}$ is given on Mohr's circles as represented in Fig. 4. Taking the geometric relation in Fig. 4 into consideration, the maximum lateral shear stress $(\tau_{hv})_{max}$ will be

$$S_u = (\tau_{hv})_{max} = \left\{(\tau_{max})^2 - \left(\frac{1-K_0}{2}\sigma_v'\right)^2\right\}^{1/2} \qquad (6)$$

where K_0 : coefficient of earth pressure at rest
σ_v' : effective overburden pressure.

In addition, if the cohesion c and the angle of internal friction ϕ of soil are given, the maximum shear stress τ_{max} can be expressed by the following equation :

$$\tau_{max}=\frac{1+K_0}{2}\sigma_v'\sin\phi+c\cos\phi \tag{7}$$

Substituting Eq. (7) into Eq. (6), the shear strength S_u of the soil element shown in Fig. 4 can be given as follows :

$$S_u=\left\{\left(\frac{1+K_0}{2}\sigma_v'\sin\phi+c\ \mathrm{con}\,\phi\right)^2-\left(\frac{1-K_0}{2}\sigma_v'\right)^2\right\}^{1/2} \tag{8}$$

However, the actual shear strength in soil deposit cannot be expressed so easily as above. For example, the process of development of the pore water pressure in the soil element vary with respective earthquakes, so that the effective overburden pressure and coefficient of earth pressure at rest are not constant, varying from the state of rest to the state of failure. The shear strength in a soil deposit cannot be exactly obtained for the above reason. Accordingly, it is realistic that the shear strength of a soil is calculated based on the assumption that the initial stress condition continues to the state of failure. The soil parameters in Eq. (8) are determined by the following conditions :

(1) The cohesion c and the angle of internal friction ϕ are determined by the expression of total stress of the unconsolidated-undrained triaxial compression test.

(2) Brooker and Ireland[10] have reported that the close relation were observed among the coefficient of earth pressure at rest K_0, the plasticity index I_p and the overconsolidated ratio OCR as shown in Fig. 5. The value of K_0 to be required in Eq. (8) was determined by the relation between I_p and OCR shown in Fig. 5.

Soil Classification

The data on initial shear modulus, shear strength and N-value of the standard penetration test were obtained from 25 sites, which consisted of 15 alluvial deposits, 9 diluvial deposits, and 1 tertiary deposit.

In order to describe the physical properties of soils used for this investigation, the triangular classification chart for the results of mechanical analysis is shown in Fig. 6, the results of plasticity and liquid limit test are shown in Fig. 7 by the plasticity chart, while the relation between overconsolidation ratio and plasticity index are shown in Fig. 8, and the relation between degree of saturation and void ratio in Fig. 9.

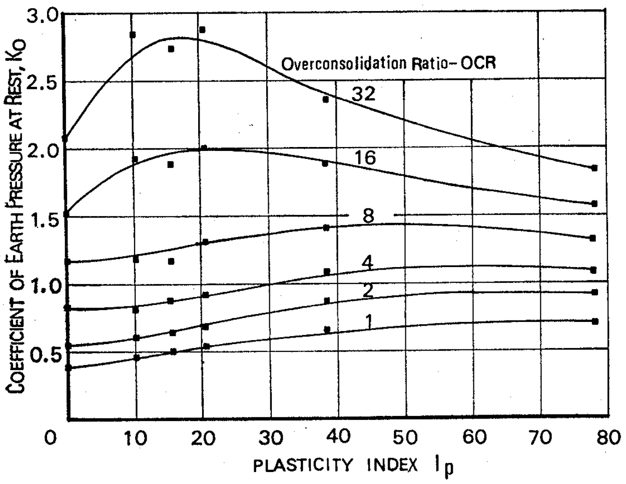

Fig. 5. Relationship between K_0, I_p and OCR (From Brooker and Ireland, 1965)

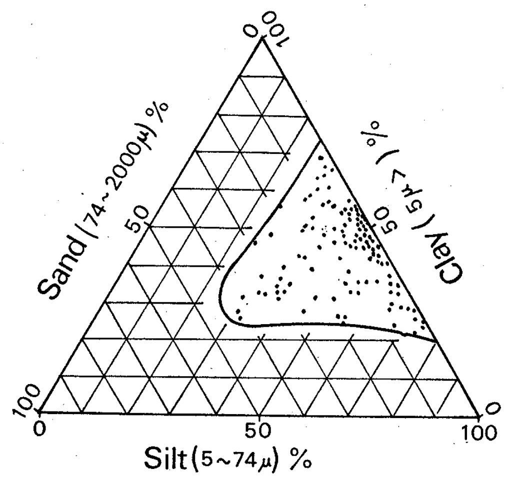

Fig. 6. Trianguler classification chart

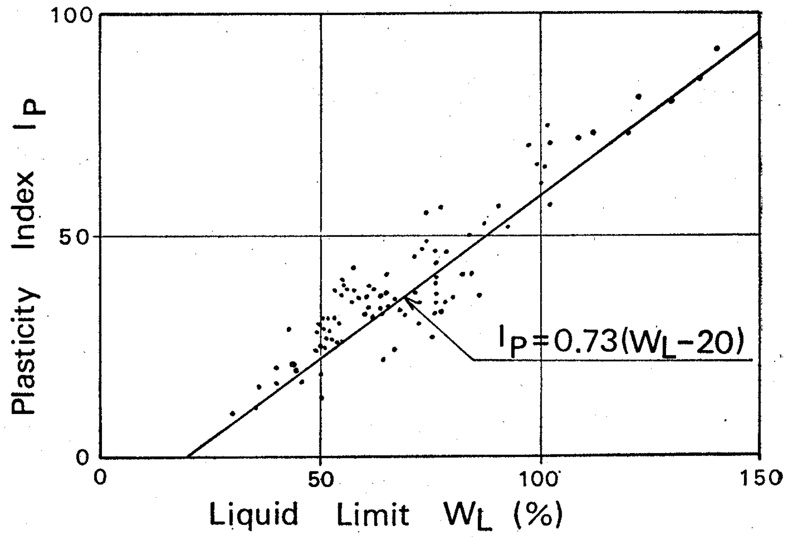

Fig. 7. Relation between I_p and W_L

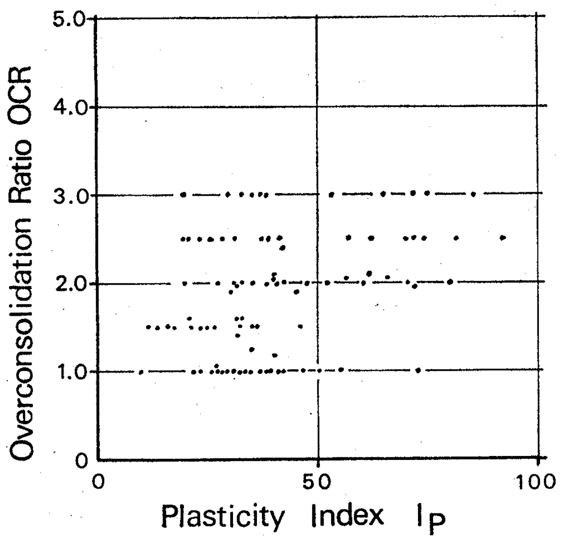

Fig. 8. Relation between OCR and I_p

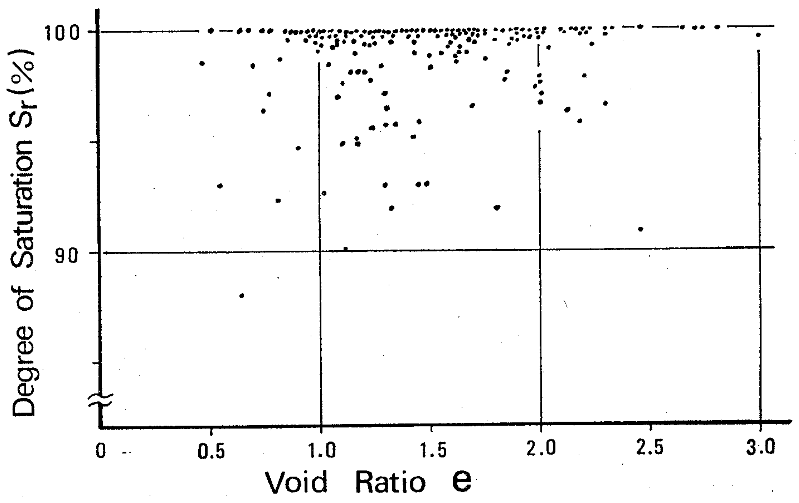

Fig. 9. Relationship between S_r and e

The physical properties of the soil used in this paper are summarized as follows from Figs. 6, 7, 8 and 9:

(1) Soil belongs to the category of cohesive soil.

(2) The plasticity index has a wide range of distribution.

(3) The values of overconsolidation ratio range from 1.0 to 3.0.

(4) The values of void ratio range from 0.5 to 3.0, and most of the data are from 1.0 to 2.0.

(5) The values of the degree of saturation range from 90% to 100%, and most of the data nearly 100%.

INITIAL SHEAR MODULUS AND N-VALUE

The well-shooting test is gradually being adopted widely, because it is proven to be provided with such advantageous features that:

(1) eliminate any necessity of securing an extensive area for measurement;

(2) ensure higher accuracy of measurement, compared with surface refraction method;

(3) allow wave velocity measurement in a soft soil layer, even if there exists a hard layer above it.

A review of the data on the well-shooting tests conducted in the past has led the authors to take notice that there have been a number of instances where the well-shooting test and the standard penetration test were conducted on the same site. It is therefore necessary to make an investigation into the relationship between the shear modulus and the N-value of soil deposits obtained respectively by the well-shooting test and the standard penetration test, so as to get a better knowledge of elastic properties of soil. When the value of initial shear modulus and N-value are to be sampled from the existing data, it is necessary, however, to set up certain criteria to avoid any partial data sampling.

Criteria of Data Sampling

In the case of the authors' investigation, criteria of data sampling were set up so that such data as might meet the following requirements could only be adopted:

(1) Soil has to fall under the category of cohesive soil.

(2) Shear wave velocity has to be measured by way of the well-shooting test.

(3) Uniform section of shear wave velocity values and N-values has to be more than 2.0 m, as shown in Fig. 10 "An Example of Data Sampling."

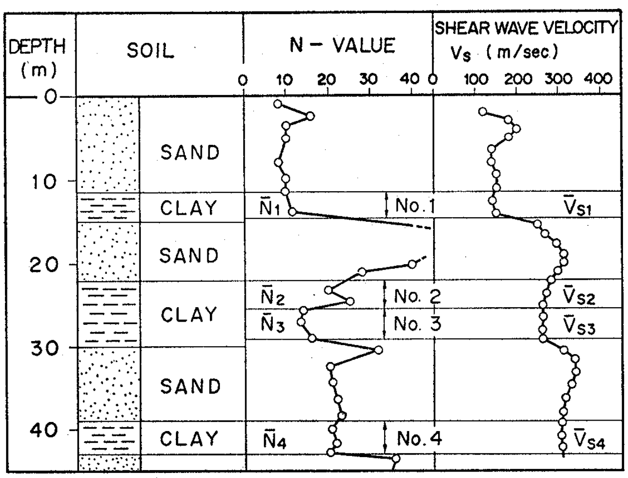

Fig. 10. An example of data sampling

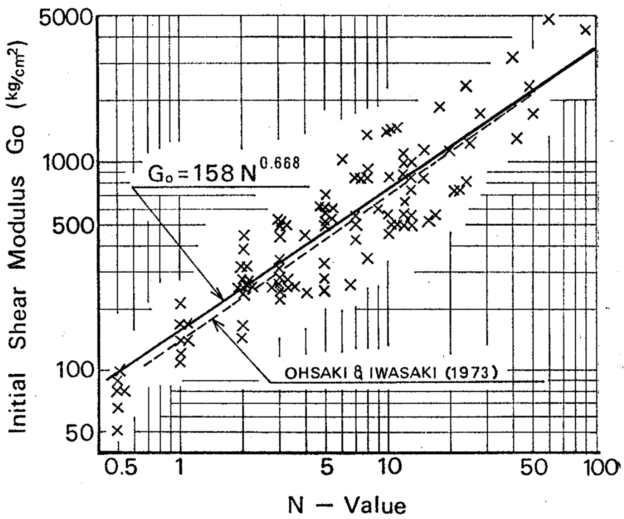

Fig. 11. Relationship between G_0 and N-value

(4) The well-shooting test and the standard penetration test have to be conducted on the same site.

Relation between Initial Shear Modulus and N-Value

The data on initial shear modulus G_0 and N-value thus sampled in conformity with the above criteria of data sampling are plotted on a logarithmic graph as shown in Fig. 11. Fig. 11 shows that G_0 and N-value is in approximate proportion to each other on the logarithmic graph.

Accordingly, on the assumption that the first order equation should be $G_0=aN^b$, the following empirical formula was obtained by calculating a and b by the least squares method :

$$G_0=158N^{0.668}(\mathrm{kg/cm^2}) \tag{9}$$

where coefficient of correlation $\rho_{xy}=0.88$.

Comparison with Other Investigations

Ohsaki and Iwasaki[11] have evaluated the relation between G_0 and N-value by arranging the data obtained from well-shooting tests and standard penetration tests, which were collected by the Evaluation Committee on High-Rise Building Structures, the Building Center of Japan. With classification of soil into cohesive soil and sandy soil with a view to facilitating evaluation of the relation between G_0 and N-value, they have shown for cohesive soils that the following empirical formula existed between G_0 and N-value :

$$G_0=140N^{0.722}(\mathrm{kg/cm^2}) \tag{10}$$

where coefficient of correlation $\rho_{xy}=0.90$. Graphical expression of Eqs. (9) and (10) show that there is quite a similarity between them, as illustrated in Fig. 11.

SHEAR STRENGTH AND N-VALUE

Now that the relation between shear modulus G_0 and N-value of the standard penetration test has been evaluated in the preceding section, the relation between shear strength S_u and N-value has to be evaluated, too, in a similar way.

Shear strength S_u can be obtained from Eq. (8) by use of the results of triaxial compression test on undisturbed soil samples. When the relation between shear strength and N-value is to be evaluated, particular attention has to be paid to soil sampling practice and accuracy of soil test. With this taken into account, the authors have sampled data solely from such basic soil exploration reports as considered to be under stringent supervision for securing as high accuracy as possible of soil test and, especially, the results of such soil tests as were conducted with special attention paid to sampling method[12] or by use of block samples, since soil samples are easy to be disturbed, if N-value exceeds 10.

Criteria of Data Sampling

Criteria of data sampling was set up so that the following requirements could be satisfied :

(1) Soil has to fall under the category of cohesive soil.

(2) Soil sampling and standard penetration test have to be carried out in the same borehole.

(3) Any data with N-value of less than 1.0 shall be rejected, because there exists no coefficient of correlation between shear strength and N-value, if the latter is less than 1.0.

Relation between Shear Strength and N-Value

Just as in the preceding section, the data on shear strength and N-value thus sampled are plotted on a logarithmic graph shown in Fig. 12. On the assumption that the first order equation should be $S_u=aN^b$, the following empirical formula was obtained by calculating a and b the method of least squares :

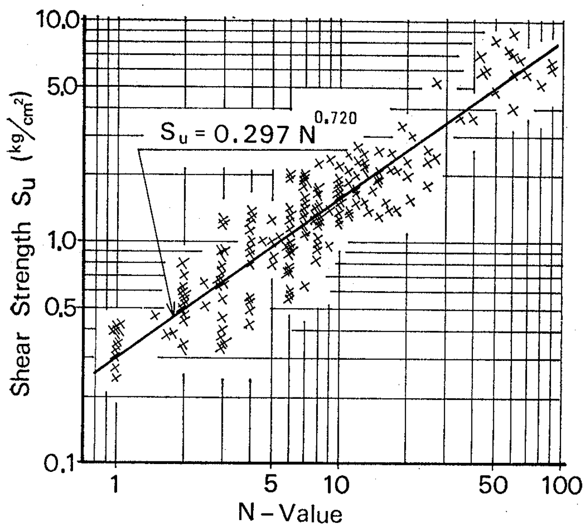

Fig. 12. Relationship between S_u and N-value

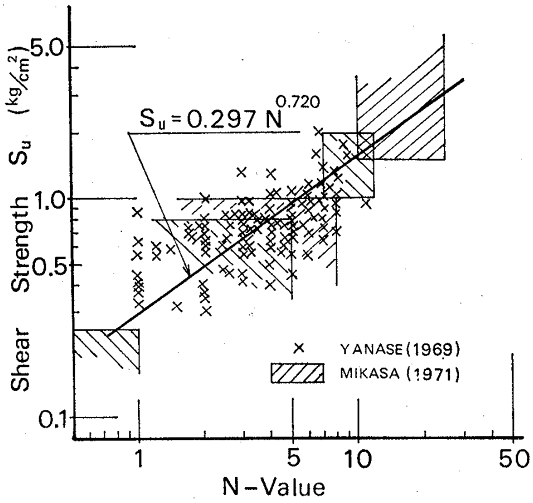

Fig. 13. Comparison between Eq. 11 and similar data by others

$$S_u = 0.297 N^{0.72} (\mathrm{kg/cm^2}) \tag{11}$$

where coefficient of correlation : $\rho_{xy} = 0.93$.

Comparison with Some Other Investigations

The shear strength of clay can be approximately obtained from the unconfined compression strength q_u by the following equation.

$$S_u \fallingdotseq \frac{1}{2} q_u (\mathrm{kg/cm^2}) \tag{12}$$

Mikasa[13] has shown the range of N-value and unconfined compression strength of cohesive soil on the basis of his investigation into typical cohesive soil layers in Osaka district. Similarly, Yanase[14] has shown many data on N-value and unconfined compression strength of cohesive soil by way of his test conducted on such soil samples as considered to have been least disturbed during sampling. These data are plotted in Fig. 13 together with graphical expression of Eq. (11), which is expressed as a straight line passing near the center of those data shown by Mikasa and Yanase.

INITIAL SHEAR MODULUS AND SHEAR STRENGTH

So far, the authors' effort has been directed toward evaluating relationship between N-value of the standard penetration test and initial shear modulus or shear strength of cohesive soil.

In this section, however, their effort is to be directed solely to evaluating direct relation between initial shear modulus G_0 and shear strength S_u. It is to be noted here that criteria of data sampling can be more moderate in this section, because of reduction in number of data on initial shear modulus and N-value, compared with the previous sections.

The data for use in this section are from one of the investigations made in the past by other researchers[5] and such data sources as Kajima Institute of Construction Technology by which the authors are employed, involving 20 sites and 194 data in total.

Criteria of Data Sampling

Criteria of data sampling was set up so that the following requirements could be met :

(1) Soil has to fall under the category of cohesive soil.

(2) Shear wave velocity has to be measured by way of the well-shooting test.

(3) Values of G_0 and S_u have to be those obtained by experiment at the same site and in the same depth. If there exist several values for S_u, corresponding number of data should be made availably by combining each value of S_u with the corresponding value of G_0.

Relation between Initial Shear Modulus and Shear Strength

Data on initial shear modulus G_0 and shear strength S_u are plotted on a logarithmic graph shown in Fig. 14. On the assumption that as a first approximation equation should be $G_0=aS_u^b$ and taking the data plotted in Fig. 14 into account, the following empirical formula was obtained by calculating a and b by the method of least squares:

$$G_0=516\,S_u^{1.012}(\mathrm{kg/cm^2}) \tag{13}$$

where coefficient of correlation: $\rho_{xy}=0.95$.

Exponent of S_u in the Eq. (13) is 1.012 or approximately equal to 1.0. G_0 can therefore be considered to be in proportion to S_u.

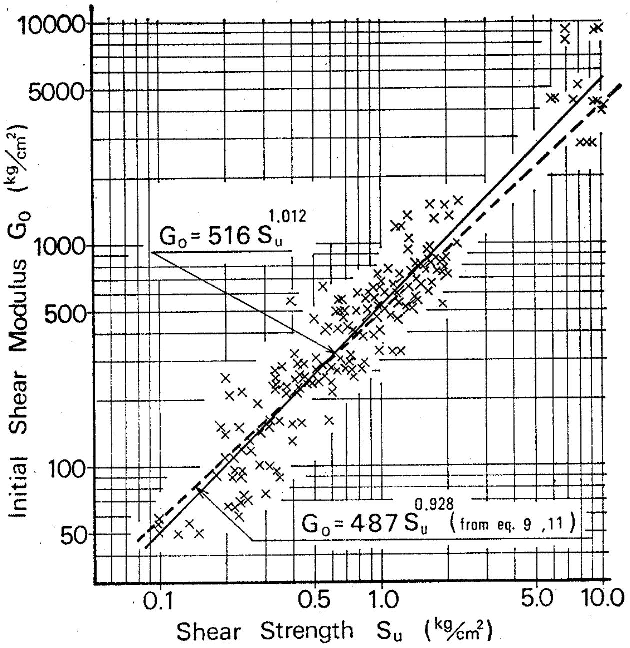

Fig. 14. Relationship between G_0 and S_u

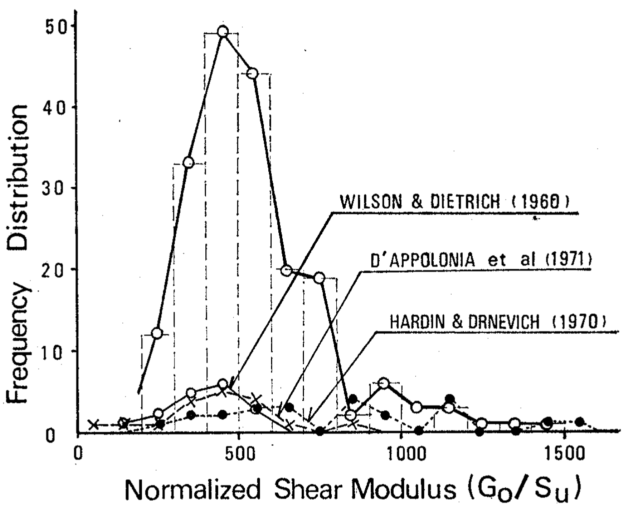

Fig. 15. Histogram of normalized shear modulus (G_0/S_u)

Investigation of Normalized Shear Modulus

Now that G_0 is confirmed to be in proportion to S_u, it is convenient to define the normalized shear modulus G_n as follows:

$$G_n=\frac{G_0}{S_u} \tag{14}$$

Fig. 15 shows a histogram of the normalized shear modulus drawn by calculating values of G_n from the data plotted in Fig. 14. The value of G_n are in the region ranging from 250 to 1430, with the mean value of 548 and the standard deviation of 211.

For comparison, the following values of G_n can be quoted independently from the data obtained by Hardin et al., Wilson et al., and D'Appolonia et al.

(1) Hardin et al.[9] have obtained the initial shear modulus and the shear strength of the same cohesive soil by means of a reasonant-column apparatus in the laboratory. The normalized shear modulus G_n calculated from these values is ranging from 380 to 1500 with its mean value of about 760.

(2) Wilson and Dietrich[1] have obtained the values of initial Young's modulus, initial shear modulus and compressible strength by measuring a longitudinal or torsional natural

frequency of clay samples. The value of G_n can be calculated from these measured data by assumming the Poisson's ratio as 0.5, and it is in the region ranging from 178 to 550 with its mean value of about 390.

(3) D'Appolonia et al.[2] have shown eleven combinations of data on the initial Young's modulus and the shear strength by in-situ and laboratory tests.

The value of G_n calculated from these values by assumming the Poisson's ratio as 0.5 is in the region ranging from 53 to 833 with its mean value about 420.

The results of these investigations are shown in Fig. 15 for comparison with the authors' result. The histogram in Fig. 15 shows that the peaks of frequency distribution curves are in the region ranging from 400 to 500 except Hardin's data.

DISCUSSION AND CONCLUSIONS

The standard penetration test is easy and less expensive to conduct mainly because of its standardization, though it has a drawback in rather low accuracy. Investigation into soil deposits by way of the standard penetration test allows a three-dimensional grasp of their structure, if the test is conducted as many times as possible, so that as many test data as possible can be obtained.

With due attention paid to such advantageous feature of the standard penetration test, the authors have finally succeeded in introducing Eqs. (9) and (11) by arranging in a statistical way the data on initial shear modulus G_0, shear strength S_u and N-value of the standard penetration test.

On the other hand, the authors have also evaluated the relation between G_0 and S_u, so that it turns out to be expressed in Eq. (13). Elimination of N from the Eqs. (9) and (11) allows the relation between G_0 and S_u to be expressed in the following equation:

$$G_0=487\,S_u^{0.928}\,(\mathrm{kg/cm^2}) \tag{15}$$

Eq. (15) is represented in Fig. 14, so that it can be compared with Eq. (13). A close resemblance existing between the two equations is self-explanatory of mutual relationship among initial shear modulus, shear strength and N-value.

The investigations covered in this paper are summarized as follows.

(1) Shear strain amplitude generated in a very soft soil deposit by the well-shooting test was in general found less than $10^{-3}\%$. It follows from this that shear modulus obtained by in-situ tests can be regarded as the initial shear modulus.

(2) As a result of investigating the mutual relationship among initial shear modulus, shear strength and N-value, it became clear that the initial shear modulus of cohesive soil is proportional to its shear strength obtained under the K_0-condition.

(3) The ratio between the initial shear modulus and the shear strength can be estimated to be about 500 from Eq. (13) and Fig. 15.

(4) It is simple and convenient if the initial shear modulus is estimated from N-value. However, we should not resort to N-value only, because errors of the standard penetration test become large, especially when N-value is less than 2.

ACKNOWLEDGEMENT

The authors wish to express their gratitude to Dr. Y. Ohsaki, Professor of University of Tokyo, and Dr. H. Kishida, Associate Professor of Tokyo Institute of Technology for their kind guidance and encouragement.

NOTATION

S_u=undrained shear strength in kg/cm^2
σ_v'=vertical effective stress in kg/cm^2
K_0=coefficient of lateral pressure at rest
OCR=overconsolidation ratio
C=cohesion obtained from unconsolidated-undrained triaxial compression test in kg/cm^2
ϕ=angle of internal friction obtained from unconsolidated-undrained test in degrees
N=number of blows per foot in the standard penetration test
G_0=initial shear modulus in kg/cm^2
V_s=shear wave velocity in m/sec
ρ_{xy}=coefficient of correlation
G_n=normalized shear modulus (G_0/S_u)

REFERENCES

1) Wilson, S.D. and Dietrich, R.J. (1960) : "Effect of consolidation pressure on elastic and strength properties of clay," ASCE, Research Conference on Shear Strength of Cohesive Soils, Boulder, Colorado, pp. 419-435.
2) D'Appolonia, D.J. and Poulos, H.G. (1971) : "Initial settlement of structure on clay," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 97, No. SM 10, pp. 1359-1376.
3) Miller, R.P. and Brown, (1972) : "Shear modulus determination of soils by in-situ methods for earthquake engineering," Proceedings of the International Conference on Microzonnation, pp. 545-558.
4) Kitsunezaki, C. (1967) : "Observation of shear wave by the special borehole geophone (1), field test on foundamental characteristics of the geophone specially designed," Geophysical Exploration, Vol. 20, No. 1, pp. 1-11 (in Japanese).
5) Shima, E. et al. (1968, 1969) : "S wave velocity measurement in subsoil layers at various parts of Tokyo," Bulletin of the Earthquake Research Institute, University of Tokyo Vol. 46, 47, pp. 1301-1312, pp. 819-829 (in Japanese).
6) Imai, T. and Yoshimura, M. (1970) : "Elastic wave velocity and soil mechanics in soft soil deposits," Tsuchi-to-Kiso, JSSMFE, Vol. 18, No. 1, pp. 17-21 (in Japanese).
7) Duncan, J.M. and Dunlop, P. (1969) : "The behavior of soils in simple shear tests," Proceedings of Seventh International Conference on Soil Mechanics and Foundation Engineering, Mexico, Vol. 1, pp. 101-109.
8) Seed, H.B. and Peacock, W.H. (1971) : "Test procedures for measuring soil liquefaction characteristics," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 97, No. SM 8, pp. 1099-1122.
9) Hardin, B.O. and Drnevich, V.P. (1972) : "Shear modulus and damping in soils : Design equations and curves," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 98, No. SM 7, pp. 667-692.
10) Brooker, E.W. and Ireland, H.O. (1965) : "Earth pressure at rest related to stress history," Canadian Geotechnical Journal, Vol. II, No. 1, pp. 1-15.
11) Ohsaki, Y. and Iwasaki, R. (1973) : "On dynamic shear moduli and Poisson's ratios of soil deposits," Soils and Foundations, Vol. 13, No. 4.
12) Koizumi, Y. and Ito, K. (1968) : "New method of soil sampling," Proceedings of Sampling Symposium, JSSMFE (in Japanese).
13) Mikasa, M. (1971) : "Characteristics of soft deposits in Osaka district," Architecture and Society, Archtectural Association of Japan, Vol. 52, No. 9, pp. 38-42 (in Japanese).
14) Yanase, S. (1969) ; "Soundings in the alluvial clay stratums (On the results of several methods)," Report of the Port and Hobour Research Institute, Vol. 8, No. 1, pp. 37-58 (in Japanese).

(Received June 15, 1973)