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RECENT RESEARCH IN GROUND VIBRATION SECTION,
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To illustrate some of the research work in earthquake and soil dynamics being conducted at the Ground Vibration Section, Earthquake Disaster Prevention Division, Public Works Research Institute, recent papers prepared by members at the section have been compiled in this report.

July 1, 1977

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CONTENTS

	Page No.
1. "Effects of Stress-Strain Conditions on Dynamic Properties of Sands"	
Proc. of JSCE, No. 242, OCT. 1975	1
2. "Brief Review of Liquefaction during Earthquakes in Japan"	
Soils and Foundations, Vol. 15, No. 4, Dec. 1975	11
3. "Observation of Underground Seismic Motions at Four Sites around Tokyo Bay"	
Some Recent Earthquake Engineering Research and Practice in Japan, Dec. 1976	23
4. "A History of Soil Liquefaction in Japan"	
6th World Conference on Earthquake Engineering, New Dehli, India, Jan. 1977	33
5. "Dynamic Soil Properties with Emphasis on Comparison of Laboratory Tests and Field Measurements"	
6th World Conference on Earthquake Engineering, New Dehli, India, Jan. 1977	39
6. "Statistical Analysis of Strong-Motion Acceleration Records"	
9th Joint Meeting, U.S.-Japan Panel on Wind and Seismic Effects, U.J.N.R. May, 1977	45
7. "Studies on Soil Liquefaction Related to Earthquake Resistant Design of Structures"	
9th Joint Meeting, U.S.-Japan Panel on Wind and Seismic Effects, U.J.N.R. May, 1977	79

8. "Effects of Soil Liquefaction on Dynamic Behavior of Pile Foundations"	
9th I.C.S.M.F.E. July, 1977	129
9. "Effects of Grain Size and Grading on Dynamic Shear Moduli of Sands"	
Soils and Foundations (To appear)	139
10. "Shear Moduli of Sands under Cyclic Torsional Shear Loading"	
Soils and Foundations (To appear)	173

EFFECTS OF STRESS-STRAIN CONDITIONS ON DYNAMIC PROPERTIES OF SANDS

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1. INTRODUCTION

In analyzing seismic behavior of soil deposits and studying soil-structure interaction problems, it is essential to evaluate dynamic properties of soils. For this purpose, in situ measurements of elastic waves and various laboratory tests using simple shear apparatus, triaxial apparatus or resonant-column apparatus have been conducted by many investigators. It is already known that dynamic properties of soils such as shear moduli and damping characteristics might depend upon stress conditions, shear strain amplitudes, strain

histories, void ratios, saturation, particle characteristics, inner structures of soils, etc. To clarify the relationship between dynamic properties of soils and those parameters, the authors have been conducted extensive resonant-column tests on dry and saturated specimens of various sandy soils. This paper describes the instrumentation employed and some results obtained from the resonant column testing.

2. APPARATUS

A resonant column procedure for soil was initiated by Iida¹⁾ and the procedures are fully

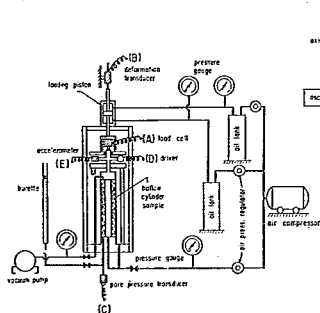


Fig. 1 Schematic diagram of the apparatus.

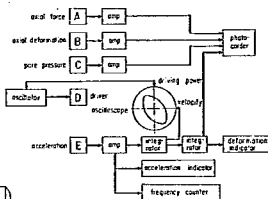


Fig. 2 Stress condition of Sample.

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described in the textbook authored by Richart, Hall and Woods²⁾. A precise equipment which is able to control the stress conditions and the magnitude of shear strain was developed by Hardin and Music³⁾. A more reliable apparatus was developed by Drnevich⁴⁾, where a hollow cylindrical specimen can be equipped. The samples of Drnevich-type apparatus are 30-cm long, 4-cm inside diameter, and either 1-cm or 0.5-cm in wall thickness depending on the grain size of the sandy material tested. The principal of ap-

paratus employed for the present study is similar to Drnevich-type apparatus. The schematic diagram is shown in Fig. 1. Fig. 2 shows the stress condition acting on a sample. The hollow cylindrical specimens tested are 25-cm in height, 10-cm in outside diameter and 6-cm in inside diameter. This arrangement permits a more uniform shear deformation in a section of samples. The specimens are fixed at the bottom and the oscillators which supply the system with the torsional vibratory force are fastened to the rigid mass on the top of the specimen. The confining pressure equally applied to the outside and inside of specimens is supplied by air pressure and the axial load can be applied independently of the confining pressure. Accordingly, the anisotropic stress condition similar to the K_0 -stress condition in the horizontal ground can be produced.

The quantities measured during a steady state vibration test are resonant frequency of the oscillator-specimen system, vibration amplitude at the top of the specimen, length change and volume change in the specimen, confining pressure and axial load. Furthermore, amplitude time decay curves are recorded after shutting off the driving power at the resonance. From these measurements, the shear strain amplitudes, the shear moduli and the logarithmic decrements can be evaluated for the testing materials.

3. PRINCIPLE OF TESTING

3.1 Shear strain amplitude

Shear strain γ in the sample is expressed as

$$\gamma = \frac{\partial u}{\partial x} = \frac{\partial \theta}{\partial x} \quad (\text{rad.}) \quad (1)$$

where $u(x, t)$ is the displacement, $\theta(x, t)$ is the angular displacement in radian, x is the axial coordinate and r is the radial coordinate. The configuration of the specimen with a large mass at the top produces linear deformation and accordingly permits uniform shear strain condition over the entire length of the specimen. Furthermore, an average shear strain in a horizontal section would be expressed by the value at 4-cm in radius. Accordingly, shear strain in the specimen is represented as

$$\gamma = \frac{4}{l} \theta_t \quad (\text{rad.}) \quad (2)$$

where l is the length of specimen (cm) and θ_t is the angular displacement at the top of the specimen.

3.2 Shear modulus

The shear stress-strain curves of sandy specimens subjected to cyclic loads have a property shown in Fig. 3. In case of the hysteretic damping such as that of sands, the linear approximation could be used to simply simulate its dissipative nature. Let the material constants of sands define as

- G : equivalent shear modulus,
- η : hysteretic damping coefficient⁽³⁾ = $\frac{\Delta W}{2\pi W}$,
- W : strain energy as shown in Fig. 3 and
- ΔW : damping energy as shown in Fig. 3.

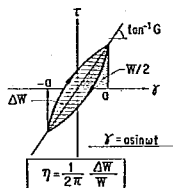


Fig. 3 Stress-strain curve of sands.

For a equivalent linear isotropic material having the same values as the above G and η , a more useful formation is that of a complex modulus. Denoting that τ and γ are shear stress and strain in complex notation, respectively and G and G' are complex shear moduli of the equivalent linear isotropic material, the stress-strain relation is expressed as

$$\tau = (G + iG')\gamma \quad (3)$$

Then, its hysteretic damping coefficient η could be related with complex shear moduli G and G' as

$$\eta = \frac{G'}{2G} \quad (4)$$

On the other hand, the wave propagation equation for the shear deformation of the continuous medium will be

$$\rho \frac{\partial^2 u}{\partial t^2} = \frac{\partial \tau}{\partial x} \quad (5)$$

where ρ is the density and u is the displacement in the orthogonal to propagation direction. Then, replacing equations (1), (3) and (4) into equation (5), the wave equation for the equivalent linear isotropic material in resonant-column tests is expressed as

Effects of Stress-Strain Conditions on Dynamic Properties of Sands

$$\rho \frac{\partial^2 \theta}{\partial t^2} = G(1+i2\eta) \frac{\partial^2 \theta}{\partial x^2} \quad (6)$$

where θ is the angular displacement in radian in complex notation. An analytical solution of this equation was obtained by the previous investigators^{2),3)}. In a steady vibration state, shear modulus G is a complicate function of the density and the dimensions of the sample, the apparatus constants and the resonant frequency. The value of G was evaluated by an iteration method by using an electronic computer.

3.3 Damping coefficient

By enforcing the system into the resonant state and then shutting off the driving power, logarithmic decrements d_t could be obtained. From the logarithmic decrements, hysteretic damping coefficients η could be obtained independently of the sample's dimensions and its boundary conditions. For free vibration, a solution could be assumed as

$$\theta = A(x) e^{(\lambda + i\omega_n t)} \quad (7)$$

where $A(x)$ is mode of deformation, ω_n is the natural circular frequency of the system and λ is an attenuation factor with respect to time. Replacing equation (7) into equation (6),

$$\lambda = -\frac{\omega_n}{2\eta} [-1 + \sqrt{1 + (2\eta)^2}] \approx \omega_n \eta \quad (8)$$

Then, the relation between logarithmic decrement d_t and hysteretic damping coefficient η will be

$$d_t = \lambda \frac{2\pi}{\omega_n} \approx 2\pi\eta \quad (9)$$

Using equation (9), hysteretic damping coefficient η can be obtained from measured logarithmic decrement d_t .

4. TESTING PROCEDURES

Physical properties of the materials tested are listed in Table 1 and their grain size distributions

Table 1 Physical properties of materials tested.

	G_s	D_{10} (mm)	D_{60} (mm)	U_c	e_{max}	e_{min}	Grain shape
Toyoura-sand	2.64	0.12	0.175	1.46	0.96	0.64	Sub-angular
Sengenyama-sand	2.70	0.16	0.38	2.37	0.91	0.59	"
Sengenyama-sand A	2.65	0.85~2.0 mm		—	0.82	0.61	"
Sengenyama-sand B	2.72	0.40~0.85 mm		—	0.92	0.67	"
Sengenyama-sand C	2.68	0.25~0.40 mm		—	0.98	0.65	"
Crushed sandstone	2.73	2.0~5.0 mm		—	1.09	0.80	"
Crushed limestone	2.74	2.0~5.0 mm		—	0.93	0.64	Angular
Glass beads A	2.42	2.0~2.6 mm		—	0.60	0.53	Spherical
Glass beads B	2.42	0.85~1.41 mm		—	0.62	0.54	"
Glass beads C	2.42	0.25~0.40 mm		—	0.65	0.52	"

are shown in Fig. 4. Toyoura-sand has uniform grading and sub-angular particles. Sengenyama-sand from Sengenyama area near Chiba, Japan has well-grading and also has sub-angular particles. This sand has diluvial origin and therefore includes some silty particles. Sengenyama-sands A, B and C shown in Fig. 4 were prepared by sieving sands obtained from Sengenyama area. Crushed sandstone have angular particles and crushed limestone have sub-angular particles. Glass beads A, B and C have almost spherical forms. As shown before, test results on granular materials are discussed in this paper.

Resonant-column tests were conducted on samples having various void ratios. For Toyoura-sand, both air-dry and saturated samples were

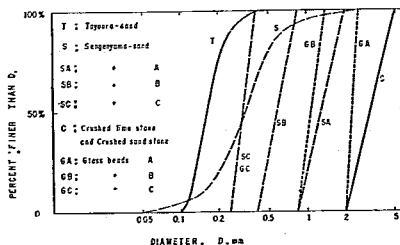


Fig. 4 Grain size distribution curves.

prepared and consolidated isotropically and anisotropically; namely, the stress ratios σ_1/σ_3 have

values of 1.0 and 2.0, respectively. For the other sands and glass beads, only air-dry specimens were tested and stress ratio σ_1/σ_3 was kept equal to 1.0. Saturated samples were prepared by spooning freshly boiled sands into a split mold filled with de-aired water. Air-dry samples were prepared by spooning air-dry sands into the mold. Densification of the samples was achieved by tapping the mold with a wooden hammer.

After the confining pressure and axial load were applied to the sample, the stress condition that is defined by σ_1 and σ_3 was kept constant and the vibratory shear strain amplitude was increased from about 5×10^{-6} to about 2×10^{-4} . Both resonant frequency and logarithmic decrement were measured at various values of shear strain amplitudes. Next, another series of tests was achieved for different stress condition and the above mentioned procedure was repeated. Confining pressure varied from 0.20 kg/cm² to 6.0 kg/cm². In Fig. 5 one of test results using a

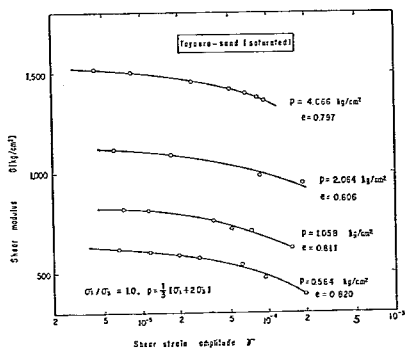


Fig. 5 Shear modulus versus shear strain amplitude of saturated Toyoura-sand.

Toyouura-sand sample is shown where p is the mean principal stress equal to $\frac{1}{3}(\sigma_1 + 2\sigma_3)$ and e is the void ratio which changed slightly during consolidation. In one sample, it would be impossible to control all values of shear strain amplitude, stress condition and void ratio. Then, it is necessary to convert the measured values of G into the values for predetermined values of p , γ and e . Meanwhile, Hardin et al.³⁾ showed the experimental equation for round Ottawa-sand for low strain amplitude as

$$G = 700 \frac{(2.17 - e)^2}{1 + e} p^{0.5} \quad (10)$$

where G is the shear modulus (kg/cm²), p is the mean principal stress (kg/cm²) and e is the void ratio. Then, the values of G for $\gamma = 10^{-3}$ and $\gamma = 10^{-4}$ were obtained from the curves in Fig. 5 and subsequently, using the postulate that G is proportional to $(2.17 - e)^2/(1 + e)$ as shown by equation (10), the values of G for the mean value of void ratios of tested sample, say $e = 0.81$ in Fig. 5, were obtained where the change in G due to the change of void ratio in this procedure was on the order of a few percent. In the next, the values of G and p were plotted on full-log graph as shown in Fig. 6, then values of G for $e = 0.81$ for $p = 0.5, 1, 2, 3, 4$ and 6 kg/cm² were obtained from the curves in Fig. 6.

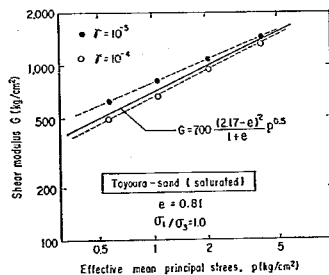


Fig. 6 Shear modulus versus mean principal stress of saturated Toyoura-sand.

5. TEST RESULTS

5.1 Effects of mean principal stress on shear modulus

As shown in Figs. 6 to 9, shear modulus varies

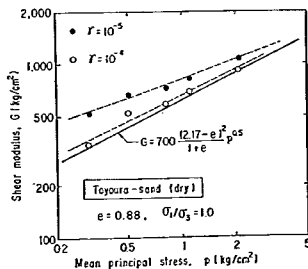


Fig. 7 Shear modulus versus mean principal stress of air-dry Toyoura-sand.

Effects of Stress-Strain Conditions on Dynamic Properties of Sands

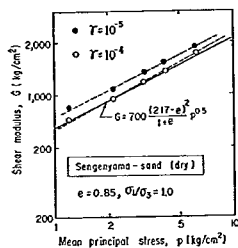


Fig. 8 Shear modulus versus mean principal stress of air-dry Sengenyama-sand.

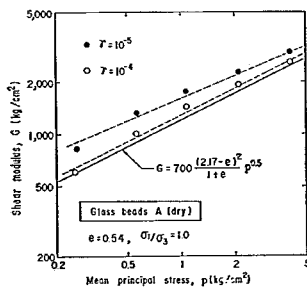


Fig. 9 Shear modulus versus mean principal stress of air-dry Glass-beads.

almost proportionally to the square root of mean principal stress provided that the other parameters keep constant. The relationship between shear modulus G and mean principal stress $p = \frac{1}{3}(\sigma_1 + 2\sigma_3)$ could be represented by

$$G = kp^m \quad (11)$$

where k is a constant. By observing these figures carefully, it is found that the values of m increase slightly with increase in shear strain amplitude as shown in Figs. 10 (a) and (b) where the legends are the same as those in Figs. 18 (a) and (b). By these figures, the value of m at $\gamma = 10^{-4}$ could be estimated to be about 0.5 for all materials tested and m at $\gamma = 10^{-5}$ is about 0.44. Furthermore, the value of m at $\gamma = 10^{-6}$ is obtained to be 0.38 by extrapolation.

5.2 Effects of void ratio on shear modulus

All data obtained are summarized in Fig. 11 where shear moduli and void ratios are plotted

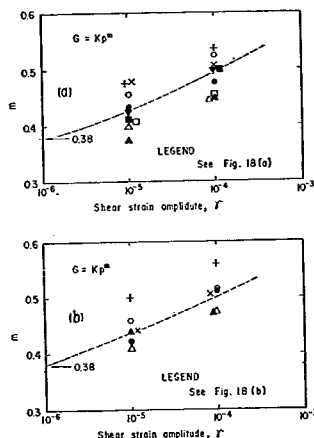


Fig. 10 m versus shear strain amplitude.

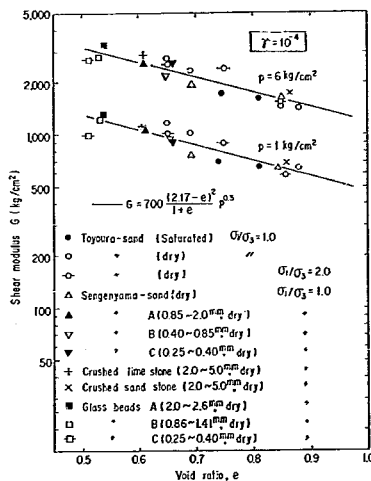


Fig. 11 Shear modulus versus void ratio of all materials tested.

on a semilog graph for two cases: $p = 1 \text{ kg/cm}^2$, $\gamma = 10^{-4}$ and $p = 6 \text{ kg/cm}^2$, $\gamma = 10^{-4}$. As denoted in Fig. 11, it is found that all data obtained almost agree with equation (10) for $\gamma = 10^{-4}$ in the range

of $\sigma_1/\sigma_3=1\sim 2$ irrespectively of roundness of particles, grading and kinds of materials used in these tests. It would be of interest that shear moduli of various granular materials are not dependent on relative density D_r , but on void ratio e .

5.3 Effects of stress ratio on shear modulus and damping coefficient

As shown in Fig. 11, shear moduli of anisotropically consolidated air-dry Toyoura-sand under $\sigma_1/\sigma_3=2.0$ agree with those of isotropically consolidated air-dry Toyoura-sand. To investigate the effects of stress ratio on shear modulus in a wider range of stress ratio, particular tests were conducted where stress ratio was increased and then decreased under constant value of p . Some of the test results are shown in Figs. 12 to 14, where stress ratio is denoted by

$$\frac{p}{q} = \frac{\sigma_a - \sigma_r}{\frac{1}{3}(\sigma_a + 2\sigma_r)} = \frac{\sigma_1 - \sigma_3}{\frac{1}{3}(\sigma_1 + 2\sigma_3)} \quad \dots\dots\dots (12)$$

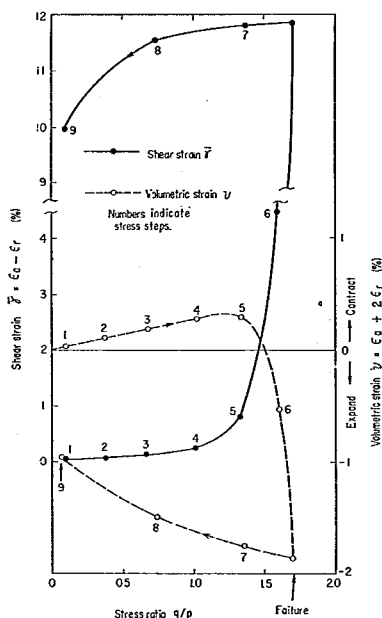


Fig. 12 Shear and volumetric strains versus stress ratio.

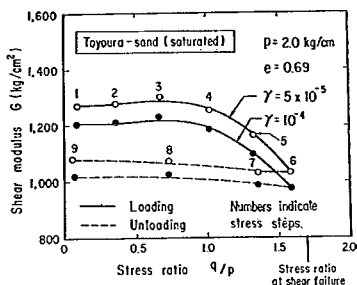


Fig. 13 Shear modulus versus stress ratio of saturated Toyoura-sand.

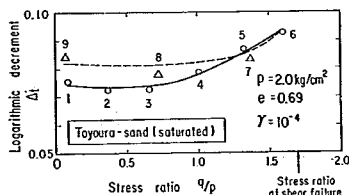


Fig. 14 Logarithmic decrement versus stress ratio of saturated Toyoura-sand.

where σ_a and σ_r mean the axial and radial stresses, respectively and q is $\sigma_a - \sigma_r$. As the value of p was not constant precisely and varied between 2.047 and 2.205 kg/cm² and the volume of the specimen varied due to dilatancy during shear deformation as shown in Fig. 12, shear moduli for $p=2.0$ kg/cm² and $e=0.69$ were obtained by modifying the measured values by using equation (10). On the other hand, logarithmic decrements in Fig. 14 are measured ones as its change with p and e is smaller than shear moduli. In Figs. 13 and 14, it is shown that shear moduli and logarithmic decrements take nearly constant values irrespectively of the variation of stress ratio q/p up to q/p of a certain value, say 1.0 (i.e., $\sigma_1/\sigma_3=2.5$). This has been indicated already by Hardin and Black¹⁷. It would be noted that shear strain $\bar{\epsilon} = \epsilon_a - \epsilon_r$ where ϵ_a and ϵ_r mean axial and radial strains, respectively, begins to increase abruptly and dilatancy of specimen starts at this stress ratio as shown in Fig. 12. However, beyond this value of stress ratio, shear moduli begin to decrease and logarithmic decrements begin to increase with increase in stress ratio. Furthermore, it is found that even after unloading shear

modulus is smaller and logarithmic decrement is larger than those before shearing. Similar results are obtained also for other p -constant test. These phenomena would be due to the anisotropic inner structure in specimen which was caused by shearing of the specimen.

5.4 Effects of shear strain amplitude on shear modulus

Because of the nonlinear characteristics of stress-strain relationship of soils, shear moduli would decrease with increasing shear strain amplitude. So far, some curves representing the G - γ relationship such as that by Seed and Idriss³⁾ have been proposed. Meanwhile, effects of shear strain amplitude on shear modulus increase with decrease in mean principal stress p . This could be supposed from Figs. 6 to 9 where the ratio of shear modulus at $\gamma=10^{-5}$ to that at $\gamma=10^{-4}$ gets larger with decrease in p . Fig. 15 shows the effects of mean principal stress on G - γ relations, where the abscissa is shear strain amplitude in log-scale and the ordinate is the ratio of shear modulus G to that at $\gamma=$

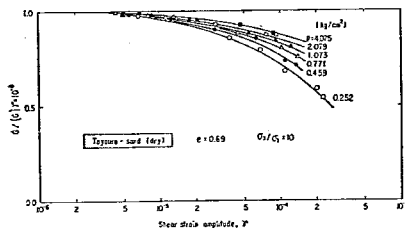


Fig. 15 Decreasing rate of shear modulus with increase in shear strain amplitude of air-dry Toyoura-sand.

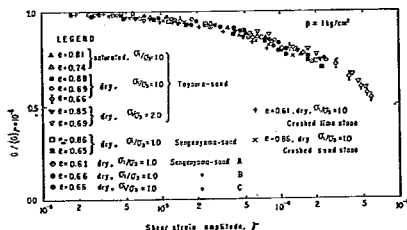


Fig. 16 Decreasing rate of shear modulus with increase in shear strain amplitude for $p=1$ kg/cm².

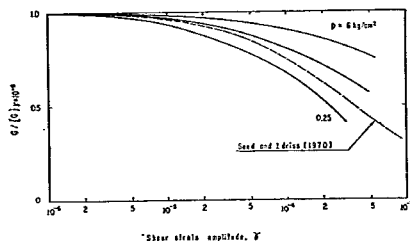


Fig. 17 Mean decreasing ratio of shear modulus with increasing in shear strain amplitude.

10^{-4} , G/G_0 at $\gamma=10^{-4}$. Furthermore, Fig. 16 shows that G/G_0 at $\gamma=10^{-4}$ - $\log \gamma$ curves are little affected by change in void ratios, stress ratios and kinds of sands for cases of materials tested. Fig. 17 shows average curves for various mean principal stress obtained by the procedure shown in Fig. 16. From this figure, it is found that the variation of shear moduli with shear strain amplitude would be smaller than that proposed by Seed and Idriss³⁾ when p has a value larger than about 0.5 kg/cm².

5.5 Shear modulus at $\gamma=10^{-4}$

The shearing strain amplitude generated in subgrounds during in-situ measurements of elastic waves is on the order of around 10^{-6} or less. To compare shear moduli obtained from those in-situ measurements with those from resonant-column tests, the relationship between shear modulus at $\gamma=10^{-6}$ and the mean principal stress and the void ratio in resonant-column tests would be necessary. However, shear modulus at $\gamma=10^{-6}$ could not be obtained directly from the resonant-column tests conducted by the authors. Therefore, those values at $\gamma=10^{-6}$ were estimated by extrapolating the G - γ curves as shown in Fig. 5. Shear moduli at $\gamma=10^{-6}$ obtained and the values of $\frac{(2.17-e)^2}{1+e} p^{0.33}$ are plotted in full-log scale in Figs. 18 (a) and (b). The power of p was obtained from Figs. 10 (a) and (b) where the value at $\gamma=10^{-6}$ would be estimated to be about 0.33. From Figs. 18 (a) and (b), it is found that the average curve for sandy soils could be represented by the following equation:

$$G = 900 \frac{(2.17-e)^2}{1+e} p^{0.33} \quad (13)$$

where G is shear modulus (kg/cm²) at $\gamma=10^{-6}$, e means the void ratio and p means the mean principal stress (kg/cm²).

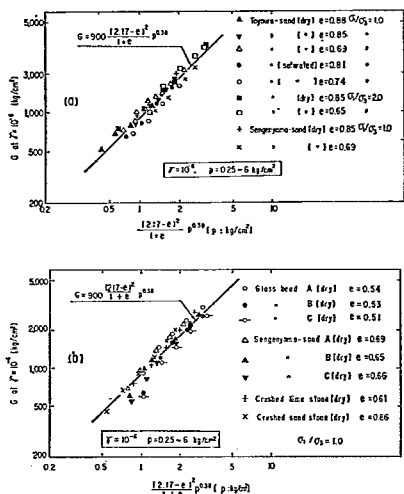


Fig. 18 Shear modulus at $\gamma=10^{-6}$ versus void ratio and mean principal stress.

5.6 Hysteretic damping coefficient

As stated before, damping characteristics of the materials tested would be expressed by hysteretic damping coefficients, which could be evaluated from logarithmic decrements if the mechanism of damping in soils is of hysteretic dissipation of energy. A typical test result for air-dry Toyoura-sand is shown in Fig. 19 where Δ_t means the logarithmic decrement and η denotes the hysteretic damping coefficient. This shows that η is

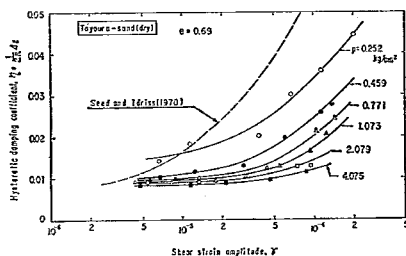


Fig. 19 Hysteretic damping coefficient versus shear strain amplitude of air-dry Toyoura-sand.

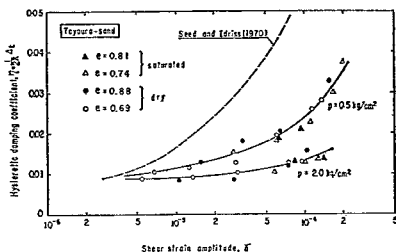


Fig. 20 Hysteretic damping coefficient versus shear strain amplitude of saturated and air-dry Toyoura-sands.

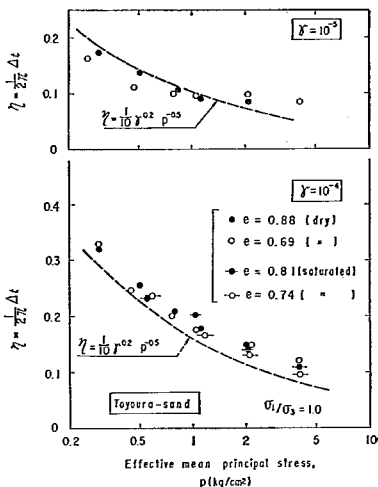


Fig. 21 Hysteretic damping coefficient versus mean principal stress of Toyoura-sands.

strongly affected by the mean principal stress and the shear strain amplitude. Saturated and air-dry specimens of Toyoura-sand had almost the same value of η as shown in Fig. 20. Furthermore, it is found that the change in void ratio does not affect damping capacities of Toyoura-sand and Sengen-yama-sand. Meanwhile, Figs. 21, 22 and 23 show that the scattering of the values of η for different materials is rather larger when comparing with that of G . Although it is

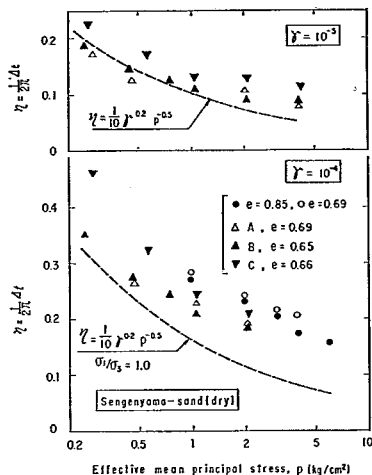


Fig. 22 Hysteretic damping coefficient versus mean principal stress of Sengenyama-sands.

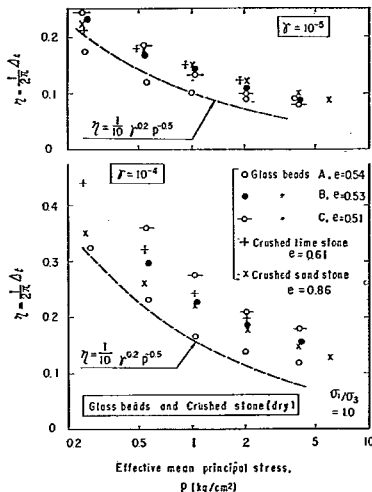


Fig. 23 Hysteretic damping coefficient versus mean principal stress of Glass-beads, Crushed sandstone and Crushed lime-stone.

hard to get a conclusion due to this scattering of data, some remarks could be obtained as follows: (1) In any case, η increases with increase in shear strain amplitude and with decrease in confining pressure. (2) η of Toyoura-sand have the values almost equal to those of Ottawa-sand, for which Hardin⁽⁶⁾ proposed an experimental equation as

$$\eta_0 = \frac{1}{10} \gamma^{0.2} p^{-0.5} \quad (14)$$

for the range of $\gamma = 10^{-6}$ to 10^{-4} and $p = 0.244$ to 1.46 kg/cm^2 where η_0 denotes a hysteretic damping coefficient for Ottawa-sand obtained by dividing logarithmic decrement by 2π . (3) While the values of η of different materials tested could not be represented by such a simple equation as equation (14), all of data could be represented by the following equation:

$$\eta = \eta_0 + \Delta\eta \quad (15)$$

where η means hysteretic damping coefficient of respective materials and η_0 is that of Ottawa-sand expressed in eq. (14) and $\Delta\eta$ is a constant which is not affected by p , void ratio and shear strain amplitude $\Delta\eta$ has a different values for different materials.

6. CONCLUSIONS

A procedure using a resonant-column apparatus developed in the present study would be one of the most effective ways to evaluate shear moduli and damping characteristics of soils. The following remarks would be concluded from the extensive resonant-column tests conducted.

- (1) By testing materials without fine particles it is found that the equation (10) expresses the experimental results of shear moduli at $\gamma = 10^{-4}$ and equation (13) fits with those at $\gamma = 10^{-6}$. For materials tested, water content, grading, particle shape and grain size have little effects on shear moduli of the value around $10^{-4} \sim 10^{-6}$ of the shear strain.
- (2) While the value of mean principal stress is kept constant, shear moduli and logarithmic decrements are nearly constant irrespectively of the value of shear stress up to the stress ratio q/p of a certain value, say 1.0. However, beyond this value of q/p , shear modulus starts to decrease and logarithmic decrement starts to increase with increase in q/p . The decreased shear moduli and the increased logarithmic decrements do not completely recover after unloading.
- (3) The decreasing rate of shear moduli with increase in shear strain amplitudes has larger

value for the smaller value of the mean principal stress.

(4) Damping capacities decrease with increase in p and increase with increase in shear strain amplitudes. However, damping capacities are not affected by the change in void ratios under constant values of the other parameters. For Toyoura-sand, water content has little effect on damping capacities. Damping values of various materials could be represented by eqs. (14) and (15).

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BRIEF REVIEW OF LIQUEFACTION DURING EARTHQUAKES IN JAPAN

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ABSTRACT

To correlate the actual liquefaction phenomena and site conditions, a literature survey about liquefaction phenomena brought by earthquakes during the last century in Japan was performed. Liquefaction distribution map of Japan and the regional maps of Kanto, Nobi and Hokuriku are presented and the factors in relation with liquefaction are discussed. In the last century liquefaction in subsoils has been observed at some hundred sites in at least 44 earthquakes and the sites were limited only to alluvial deposits and reclaimed lands. Furthermore, it was found that liquefaction occurred repeatedly in different earthquakes in several zones. Estimated JMA intensity scale at liquefied sites were more than the fifth grade which means maximum acceleration of 80 to 250 gals. The extent of liquefied zones are limited depending on the magnitude of the earthquakes.

Key words: alluvial deposit, case history, earthquake, earthquake damage, earthquake resistant, history, liquefaction

IGC: B4/E8

INTRODUCTION

Much of structural damage caused by settlement or tilting due to liquefaction of saturated sandy subsoil has inevitably occurred during major earthquakes. In accordance with prediction of liquefaction potential of subsoils, number of research efforts have been expended by means of laboratory tests and in-situ tests. Meanwhile, correlations between the actual liquefaction phenomena and site conditions have been rarely investigated upon limited spaces and periods. So far, reviews of the correlation of liquefaction have been performed at several sites in Japan (Miyabe, 1933, Kishida, 1969, Ishihara, 1974, and Iida, 1974). Because of many interesting facts in these reviews, wider reviews of this subject would be expected. This report contributes them through a literature survey of liquefaction phenomena brought by earthquakes during the last century in Japan.

Liquefaction distribution maps since 1872 is presented firstly and the factors in relation with liquefaction are discussed. In the literatures on earthquakes which brought damage during the past several centuries, there are many descriptions on liquefaction of subsoil. However, almost all of the descriptions in them before 1872 are rarely qualified on scientific and engineering viewpoint. Meanwhile, since 1872, liquefaction phenomena can be

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qualified in identification of liquefaction and characteristics depending on. In the descriptions, evidences of liquefaction in subsoil are given from i) water spouting with sand or mud from wells or cracks in the ground, ii) sand boils or sand volcanoes, iii) excessive settlements of heavy structures placed on sand layers and iv) floating out of wooden piles from rice paddy or floating out of caissons under construction from river beds. In this procedure as above, the descriptions of the increase of water spouting during earthquakes in the area under the artesian condition were not adopted as the evidences of liquefactions in sand layers. About 150 papers or reports give information on liquefaction in 44 earthquakes listed in Table 1.

LIQUEFACTION DISTRIBUTION MAPS

At least 44 earthquakes brought liquefaction of subsoil in Japan during the past century. Epicenters of the earthquakes and locations where liquefaction occurred are shown in Fig. 1. Characteristics of the earthquakes (year, date, magnitude, focal depth, epicenter location and the number of the victim) are listed in Table 1. Large scales of magnitude in the earthquake bring liquefaction in wider area except several cases as shown in Fig. 1. Among the earthquakes of large scale, the earthquakes No. 4 (Nobi), No. 12 (Gono), No. 15 (Kanto), No. 22 (Nishi-Saitama), No. 34 (Fukui) and No. 42 (Niigata) in Table 1 definitely brought widely spread and violent liquefaction in subsoil. Furthermore, the earthquakes No. 6 (Shonai), No. 8 (Rikuu), No. 16 (Tajima), No. 31 (Tonankai) and No. 44 (Tokachi-

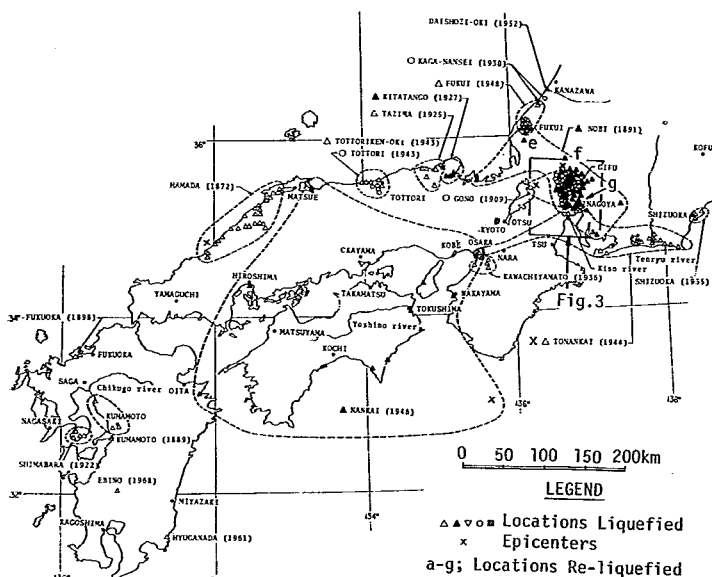
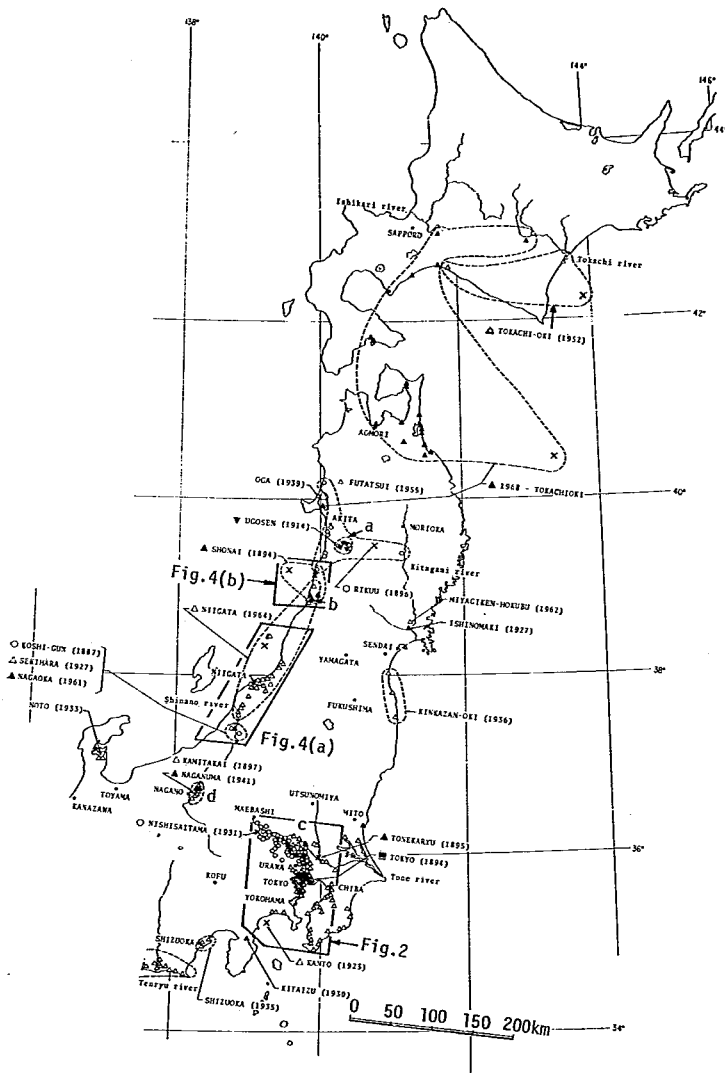


Fig. 1. Liquefaction

REVIEW OF LIQUEFACTION DURING EARTHQUAKES



distribution map (Japan)

KURIBAYASHI AND TATSUOKA

Table 1. Lists of earthquakes which induced liquefaction (1872-1968)

Earthquake	Year, Month, Date	Magnitude	Focal depth (km)	Epicenter	Persons killed
1. Hamada	1872. 3. 14	7.1		34.8°N, 132.0°E	552
2. Koshigun	1887. 7. 22	6.1		37.7°N, 139.0°E	0
3. Kumamoto	1889. 7. 28	6.3		32.8°N, 130.7°E	20
4. Nobi(Mino-Owari)	1891. 10. 28	8.4(7.9)		35.6°N, 136.6°E	7,273
5. Tokyo	1894. 6. 20	7.5		35.7°N, 139.9°E	31
6. Shonai	1894. 10. 22	7.3		39.2°N, 139.5°E	726*
7. Tone-karyu	1895. 1. 18	7.3		35.9°N, 140.4°E	9
8. Rikuu	1896. 8. 31	7.5		39.5°N, 140.7°E	209
9. Kamitakai	1897. 1. 17, 4. 30	6.3, 6.3		36.6°N, 138.2°E	0
10. Minami Uonumagun	1898. 5. 26	6.7		36.9°N, 138.9°E	0
11. Fukuoka	1898. 8. 10, 8. 12	6.5, 6.5		33.5°N, 130.2°E	0
12. Gono (Añegawa)	1909. 8. 14	6.9		35.4°N, 136.3°E	41
13. Ugosen	1914. 3. 15	6.4		39.5°N, 140.4°E	94
14. Shimabara	1922. 12. 8	6.5, 5.9		32.7°N, 130.1°E	50
15. Kanto	1923. 9. 1	7.9		35.2°N, 139.3°E	99,331
16. Tajima	1925. 5. 23	7.0		35.7°N, 134.8°E	128
17. Kitatango	1927. 3. 7	7.5	10	35.6°N, 135.1°E	2,925
18. Ishinomaki	1927. 8. 6				
19. Sekihara	1927. 10. 27	5.3	0-10	37.5°N, 138.8°E	0
20. Kaga-nanseibu	1930. 10. 17				
21. Kita-izu	1930. 11. 26	7.0	0-5	35.1°N, 139.0°E	272
22. Nishi-Saitama	1931. 9. 21	7.0	10-20	36.1°N, 139.2°E	16
23. Noto	1933. 9. 21	6.0	15	37.1°N, 137.0°E	3
24. Shizuoka	1935. 7. 11	6.3	10	35.0°N, 138.4°E	9
25. Kawachiyamato	1936. 2. 21	6.4	20	34.5°N, 135.7°E	9
26. Kinkazan-oki	1936. 11. 3	7.7		38.2°N, 142.2°E	0
27. Oga	1939. 5. 1	7.0	0	39.95°N, 139.8°E	27
28. Nagano	1941. 7. 15	6.2	5-20	36.7°N, 138.3°E	5
29. Tottoriken-oki	1943. 3. 4, 3. 5	6.1, 6.1	20, 20	35.6°N, 134.2°E	0
30. Tottori	1943. 9. 10	7.4	10	35.5°N, 134.2°E	1,083
31. Tonankai	1944. 12. 7	8.0	0	33.7°N, 136.2°E	998
32. Mikawa	1945. 1. 13	7.1	0	34.7°N, 137.0°E	1,961
33. Nankai	1946. 12. 21	8.1	30	33.0°N, 135.6°E	1,330
34. Fukui	1948. 6. 28	7.3	20	36.1°N, 136.2°E	3,895
35. Tokachi-oki	1952. 3. 4	8.1	4.5	42.15°N, 143.85°E	28
36. Daishoji-oki	1952. 3. 7	6.8	20	36.45°N, 136.20°E	7
37. Tokushima-nanbu	1955. 7. 27			33.80°N, 134.30°E	1
38. Futatsui	1955. 10. 19	5.7	0-10	40.3°N, 140.2°E	0
39. Nagaoka	1961. 2. 2	5.2	20	37°27'N, 138°50'E	5
40. Hyuganada	1961. 2. 27	7.0	40	31°36'N, 131°51'E	2
41. Miyagiken-hokubu	1962. 4. 30	6.5	0	38°44'N, 141°08'E	3
42. Niigata	1964. 6. 16	7.5	40	38°21'N, 139°11'E	26
43. Ebino	1968. 2. 21, 2. 22/ 3. 25	5.7, 6.1 5.6/5.7, 5.4	0 0/0, 10	38°01'N, 130°43'E	3
44. Tokachi-oki	1968. 5. 16	7.9	0	40°44'N, 143°35'E	49

* in Yamagata Prefecture only

REVIEW OF LIQUEFACTION DURING EARTHQUAKES

oki) brought locally violent spouting of water or a number of large sand volcanoes.

Microscopic features of regions of densely distributed locations liquefied are shown in Figs. 2, 3 and 4 which are maps of Kanto, Nobi and Hokuriku regions, respectively.

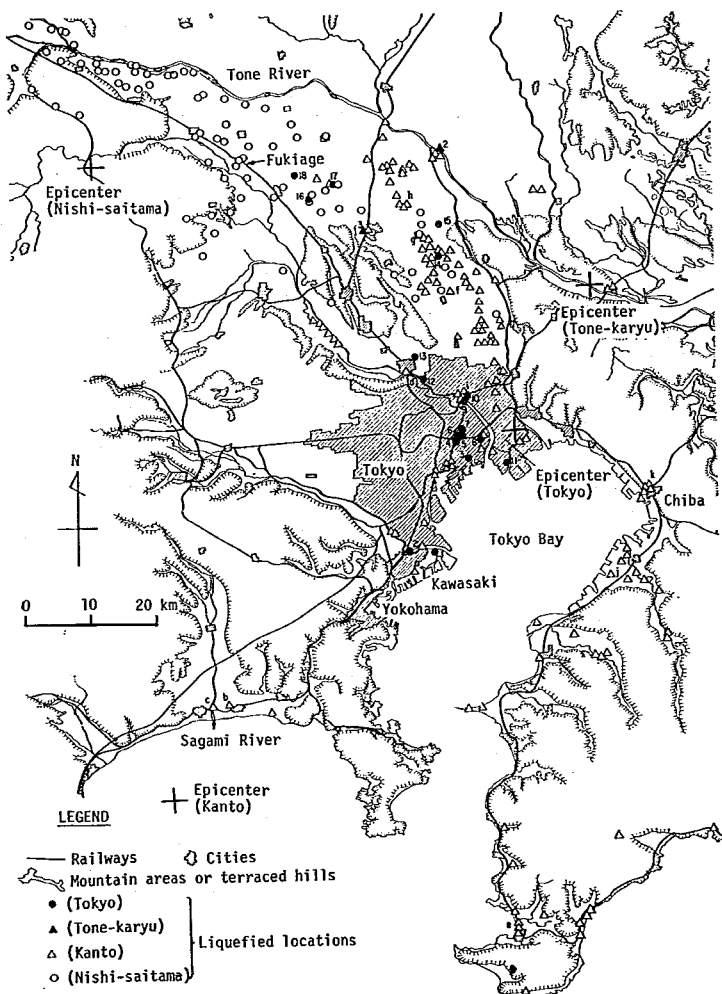


Fig. 2. Liquefaction distribution map (Kanto region)

Kanto Region

In Kanto region during the earthquake No. 5 (Tokyo), spouting of water with dark blue or black fine sand from cracks in the ground at many places were observed. In Fig. 2 the sites of liquefaction are denoted by solid circles with numbers 1 to 18 which are distributed only in alluvial areas and densely near the epicenter. About 30 years after this earthquake, the Kanto region was hit by the earthquake No. 15 (Kanto) which was one of the most disastrous earthquakes in Japanese history. Sites of liquefactions during this earthquake denoted by hollow triangles in Fig. 2 are very widely spread in alluvial plains in Kanto region. It is noteworthy that many liquefied sites are concentrated on the original winding river course of the existing Tone river located to the northeast of Tokyo. About three hundred and fifty years ago, the estuary of the Tone river was linked to the Pacific Ocean by human power. The original Tone river had flowed into Tokyo Bay before then and very deep and soft deposits with sand layers had been sedimented in the old river course in historical scale of long duration. Because of the revolution of the river course, higher potential of liquefaction has been retained in the original sites in comparison with ordinary site in Kanto plain, both of the earthquakes as above and also the earthquakes No. 7 and No. 22 (Nishi-Saitama) in this century brought liquefaction of subsoil in this area. Besides this area, sand boils, sand volcanoes, floating up of wooden piles and caissons during the earthquake No. 15 (Kanto) were reported as shown in Fig. 2 where representative liquefied sites are denoted by letters *a*, *b*, ..., *k*. For example, the site *a* is the reclaimed land in Kawasaki city where water with dark blue and grey fine sand ejected from dozens of cracks. At the site *b* near the epicenter, seven wooden piles floated up about 60 cm from rice paddy during the quake. These piles were used as bridge piles about 800 years ago over the original Sagami river. And at the site *c* many caissons for bridge foundations under construction floated up from the river bed of the Sagami river. More detailed description of liquefaction are reported in another paper (Kuribayashi, Tatsuoka and Yoshida, 1974).

The earthquake No. 22 of 1931 (Nishi-Saitama) also induced liquefaction at many sites along the original and existing courses of the Tone river. The liquefied sites during this earthquake are denoted by hollow circles in Fig. 2. One of the most violent water spouting was observed at the sites around Fukiage city about 20 km east of the epicenter where a large quantity of water with dark blue and grey fine sand ejected about 30 cm high and several hundred sand volcanoes were observed. Furthermore, the ground surface of liquefied sites settled about 15 cm and spouted water covered the ground to a depth of about 15 cm.

Nobi (Mino-Owari) Region

The Nobi region has been inflicted by four violent earthquakes during the past century which induced liquefaction at many sites as shown in Fig. 3. Among them, the most violent one was the earthquake No. 4 of 1891 (Nobi) which brought widely spread and violent liquefaction in Nobi plain and Fukui plain which are denoted by solid triangles in Fig. 1 and by hollow triangles in Fig. 3. Liquefied sites are limited to the alluvial soft deposits of the three rivers of Kiso, Nagara and Ibi in Nobi plain and to the sites along river courses in Fukui plain which is placed in the north of the epicenter. It would be noteworthy that the earthquake No. 34 of 1948 (Fukui) also induced liquefaction in the whole of Fukui plain as shown in Fig. 1. One of the most violent water spouting during the earthquake No. 4 was observed at the site denoted by the letter *a* in Fig. 3 near the Shonai river where water with sand ejected high above 2 m from wells and sands deposited over the roofs of nearby houses. And at the site *b* on the right bank of the Shonai river to the south of the site *a*, a thousand and several hundred sand boils were observed.

REVIEW OF LIQUEFACTION DURING EARTHQUAKES

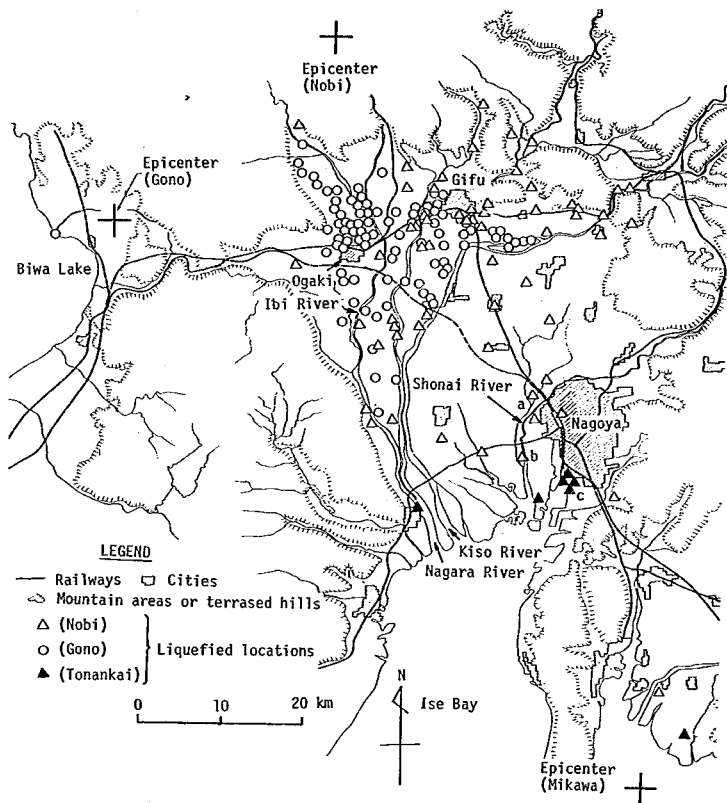


Fig. 3. Liquefaction distribution map (Nobi region)

The extent of liquefaction could be evaluated from that rice paddy and field of about 40 km² in the prefectures of Aichi and Gifu were made impossible to be cultivated due to sand boils, necessitating repairs and reconstruction.

Eighteen years after this earthquake, the earthquake No. 12 (Gono) brought liquefaction at many sites in the north-western part of the Nobi plain where liquefaction had also been induced during the earthquake No. 4. Many sites liquefied during this earthquake which are denoted by hollow circles in Fig. 3 densely distribute in the area surrounded by three rivers of Ibi, Nagara and Kiso.

The earthquake No. 31 (Tonankai) in the last stage of the World War II brought liquefaction in the coastal region with soft alluvial deposits between the Cape Omaezaki and

Lake Hamana and in the south coastal part of Nagoya city which had been reclaimed during the past hundred and fifty years before this earthquake. Such liquefied sites as above are denoted by hollow triangles in Fig. 1 and solid triangles in Fig. 3. It was found that these liquefied sites were mostly distributed in severely damaged zones.

In the south coastal region of Nagoya city the damage to wooden houses such as settlement and tilting was mainly due to a very large quantity of ejected sand and water. Furthermore, many sand boils and water spoutings were observed at the sites of very severely damaged airplane factories at Nishi-inaei town which are denoted by the letter *c* in Fig. 3. It would be of interest that the earthquake No. 32 (Mikawa) of the following year violent

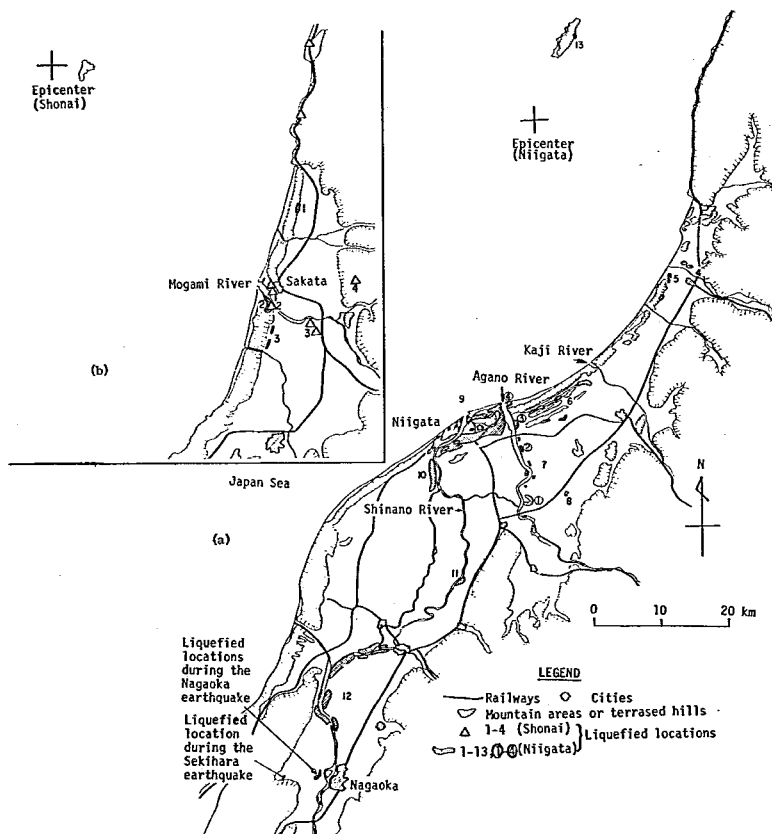


Fig. 4. Liquefaction distribution map (Hokuriku region)

REVIEW OF LIQUEFACTION DURING EARTHQUAKES

boiling of sand and water was observed again at the sites of these factories as above.

It is characteristic of the earthquakes with large magnitude but with offshore epicenters such as the earthquake No. 31 (Tonankai), No. 33 (Nankai) and No. 44 (Tokachi-oki) that most sites liquefied are located at the soft deposits in the coastal regions as shown in Fig. 1.

Hokuriku Region

Fig. 4 shows the liquefaction history of prefectures of Yamagata and Niigata during the past century. The earthquake No. 6 (Shonai) brought liquefaction at many sites which are denoted by hollow triangles in Fig. 4(b). In Sakata city located at the river mouth of the Mogami river numerous sand boils were observed and the largest sand volcano reported was about 60cm in height and about 3m in diameter. The area where sand boiled most violently were covered by ejected sand as if this area were seashore. Similar violent sand boils were observed at the site 2 in Fig. 4(b) where several tens of sand volcanoes were observed and the largest one was about 9m in diameter. It is of interest that liquefaction occurred again in this site during the earthquake No. 42 (Niigata) as shown in Fig. 4(b).

The earthquake No. 42 (Niigata) is well known for its widely spread liquefaction and the damage to modern buildings and civil engineering structures due to the liquefaction of deep saturated sand deposits. As this is reported in detail in many other reports, only a summary is presented in this paper. Liquefied zones during this earthquake are denoted by hatched zones with numbers 1 to 13 in Fig. 4 and it may be seen that these zones are limited to the original riverbeds of the rivers Shinano, Agano, etc. For example, the zone denoted by the letter *a* in Fig. 4(a) is the original riverbed of Agano river which connected with Shinano river at the river mouth about three hundred years before. This zone was reclaimed by sand about fifty years ago. The national highway No. 7 passes through this zone and its low bank made with sand of 1 m in height in this zone settled down about 1 m and moved laterally 4 m in the maximum during the earthquake. This would be due to liquefaction of sandy bank and sandy ground because boiled sand covered the road surface and one truck on the bank sank into the ground.

DISCUSSION AND REMARKS

From the whole liquefaction history surveyed as above the following was deduced.

(i) Re-liquefaction was observed in 7 zones at least as shown by the letters *a* to *g* in Fig. 1 and as listed in Table 2. This suggests that re-liquefaction occurs if conditions are satisfied.

(ii) One of the causes of liquefaction in subsoils is the intensity of quake. Earthquake intensity at liquefied site were estimated by comparing the both maps of liquefaction distribution and intensity scale distribution. From this procedure, it was found that estimated earthquake intensity in JMA (Japan Meteorological Agency) scale at liquefied sites were more than the fifth grade which means the maximum accelerations of 80 gals to 250 gals.

(iii) As well known, the liquefaction zone gets wider, the larger the earthquakes. In Fig. 5 the maximum epicentral distance of liquefied sites *R* (km) and the magnitude of the earthquake *M* are plotted. In relation with this arrangement Fukuoka (1971) suggested that this relationship could be expressed by

$$\log_{10} R = M - 5.7 \quad (1)$$

Eq. (1) is based on data of the earthquakes No. 33 (Nankai) and No. 42 (Niigata). Meanwhile it is found from Fig. 5 that the following equations express the linear relationship more appropriately than Eq. (1). The mean line of the whole data in Fig. 5 is expressed by

Table 2. Zones where re-liquefaction was observed
a, b, ... , g correspond to those in Fig. 1.

Zones	Earthquakes
<i>a</i> : the narrow zone along the Omono River in the northwest of Oomagari city in Akita Prefecture	No. 8 (Rikuu) and No. 13 (Ugosen)
<i>b</i> : on the left side at the river mouth of Mogami river	No. 6 (Shonai) and No. 42 (Niigata)
<i>c</i> : in the original river course of Tone river in the northeast of Tokyo	No. 5 (Tokyo), No. 7 (Tonekaryu), No. 15 (Kanto) and No. 22 (Nishi-Saitama)
<i>d</i> : on the both sides of Chikuma river near Nagano city	No. 9 (Kamitakai) and No. 28 (Nagano)
<i>e</i> : in the southern part of Nobi plain	No. 4 (Nobi) and No. 24 (Fukui)
<i>f</i> : in the northwestern part of Nobi plain	No. 4 (Nobi) and No. 12 (Gono)
<i>g</i> : in the southern part of Nobi plain	No. 31 (Tonankai) and No. 32 (Mikawa)

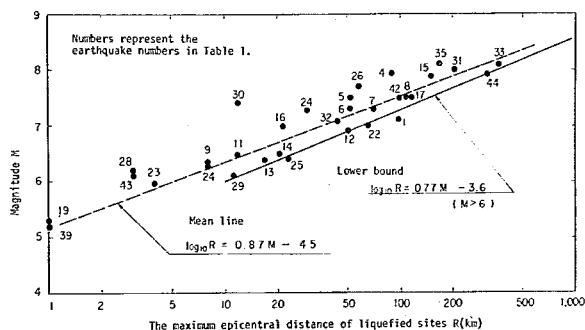


Fig. 5. Relationship between the maximum epicentral distance of liquefied sites R and magnitude M

$$\log_{10} R = 0.87M - 4.5 \quad (2)$$

and the lower bound for $M > 6.0$ is expressed by

$$\log_{10} R = 0.77M - 3.6 \quad (3)$$

Limiting discussion on the data of 44 earthquakes, a lower bound of liquefaction in R - M relation can be conceivably anticipated by Eq. (3) and in Fig. 5. On the basis of this relation liquefaction may not occur at any site more distant than expressed by Eq. (3). For $M=7.0$, for example, liquefaction may not occur at more distant site than 60 km

REVIEW OF LIQUEFACTION DURING EARTHQUAKES

from epicenter.

CONCLUSIONS

By surveying many reports of earthquakes during the last century, it was found that liquefaction in subsoils was induced at many sites in Japan during at least 44 earthquakes and liquefied sites were limited only to alluvial deposits and reclaimed lands. Especially, liquefaction occurred selectedly in old river courses which had been reclaimed during the last few hundred years and in reclaimed lands along seas or lakes. Such a tendency as above becomes clearer in case of the earthquakes with large magnitude and with offshore epicenter. Furthermore, it could be inferred that liquefaction occurred repeatedly during different earthquakes in several zones while rigorous identification of liquefied sites is difficult. Estimated JMA intensity scale at liquefied sites were more than the fifth grade. A lower bound of liquefaction potential can be defined as a relation between epicentral distance and magnitude of earthquake on the basis of this discussion.

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APPENDIX

Japan Meteorological Agency (JMA) Intensity Scale

0	No sensation: recorded by seismographs but no perception by the human body. [under 0.8]
I	Slight: felt by persons at rest or persons especially sensitive to earthquakes. [0.8~2.5]
II	Weak: felt by most persons; slight rattling of doors and Japanese latticed paper sliding doors (shōji) [2.5~8.0]
III	Rather strong: shaking of houses and buildings; heavy rattling of doors and shōji, swinging of chandeliers and other hanging object; movement of liquids in vessels. [8.0~25.0]

KURIBAYASHI AND TATSUOKA

IV	Strong: strong shaking of houses and buildings; overturning of unstable objects; spilling of liquids out of vessels four-fifths full. [25.0~80.0]
V	Very strong: cracking of plaster walls; overturning of tombstones and stone lanterns; damage to masonry chimneys and mud-plastered warehouses. [80.0~250.0]
VI	Disastrous: demolition of up to 30% of Japanese wooden houses; numerous landslides and embankment failures; fissures in flat ground. [250.0~400.0]
VII	Ruinous: demolition of more than 30% of Japanese wooden houses. [over 400.0]

note) []: Maximum acceleration, in gal

Magnitudes of Earthquakes

1. For earthquake before 1922 in Table 1, the magnitude, the Kawazumi scale, is defined as follows,

$$M_k = I + 0.00166(A - 100) - 2 \log_e(100/A) \quad (B-1)$$

where M_k is the magnitude on the Kawazumi scale, A is the epicentral distance in km and I is the intensities in JMA scale.

2. For earthquake after 1923 in Table 1, the magnitude, the JMA scale, is defined as follows,

$$M = \log_{10} A + 1.73 \log_{10} d - 0.83 \quad (B-2)$$

where M is the magnitude, A is the maximum displacement in cm and d is the epicentral distance in km.

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OBSERVATION OF UNDERGROUND SEISMIC MOTIONS

AT FOUR SITES AROUND TOKYO BAY

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ABSTRACT

This article describes dynamic behavior of subsurface soil and rock layers on the basis of acceleration records triggered during actual moderate earthquakes. Borehole accelerometers are installed at four sites around the Bay of Tokyo. These were equipped in 1970-74, in connection with the Tokyo Bay Loop Highway Project proposed by the Ministry of Construction.

Important acceleration records were obtained during sixteen moderate earthquakes (Magnitudes = 4.8 - 7.2) which occurred near that area in September, 1970 through February, 1975. From vertical distributions of maximum accelerations at the four stations, it seems that the surface magnification factors (ratios of the surface acceleration to the base acceleration) are large (2.5 to 3.5) at a soft clayey soil site, small (about 1.5) at a rocky site, and medium (1.5 to 3) at sandy soil sites.

Response spectrum curves from typical acceleration records are shown. Comparison to the spectral curves from records obtained at three of four levels at any one of four stations during an earthquake suggests that frequency characteristics at the several depths are comparatively similar. Also it seems that frequency characteristics of earthquake ground motions are influenced by seismic conditions such as magnitudes, epicentral distances, as well as soil conditions at the sites.

1. INTRODUCTION

At the present there are more than 200 strong motion accelerographs (mostly SMAC type) installed at the ground surface of various sites throughout Japan, which provide us with ground motion records during strong earthquakes. The number of stations, however, where borehole seismometers are installed in the underground is comparatively less. To analyze dynamic behavior of soil layers during earthquakes, the measurements of earthquake motions at the deeper levels below the ground surface seem very important. This article intends to present the results of seismic observation with use of borehole accelerometers installed at four stations around Tokyo Bay.

2. OUTLINE OF OBSERVATION STATIONS

The Public Works Research Institute and the Kanto Regional Construction Bureau have been measuring underground seismic motions at four stations around the Bay of Tokyo¹⁾. As shown in Fig. 1, the stations are located in Futtsu Cape, Chiba Prefecture; Ukishima Park, Kawasaki City; Kannonzaki, Yokosuka City; and Ohgishima, Yokohama City. These were equipped in 1970 through 1974, in connection with the Tokyo Bay Loop Highway Project proposed by the Ministry of Construction.

Depths of borehole accelerometers and soil profiles at the four stations are shown in Figs. 2 and 3, respectively. At the site of Futtsu Cape, where a comparatively uniform sand layer exists, 3-component borehole accelerometers are set up at three levels, the surface, -70 m, and -110 m. At Ukishima Park,

consisting of reclaimed soils, silts, clays and sands, four pickups are installed at the surface, -27 m, -67 m, and -127 m. At Kannonzaki, indicating a siltstone rocky layer with a shallow topsoil, three accelerometers are set up at the surface, -80 m, and -120 m. Furthermore, at Ohgishima, a newly reclaimed island consisting of deep reclaimed sands and stiff clays, four acceleration transducers are installed at the surface, -15 m, -38 m, and -150 m. Each accelerometer has three components, one vertical and two horizontal. Time dependent acceleration waves are triggered directly on papers of oscillographs.

3. ACCELERATION RECORDS OBTAINED

Table 1 indicates a list of earthquake records obtained during sixteen moderate earthquakes (Magnitudes = 4.8 to 7.2) which broke out near that area between 1970 and 1975. In the table the maximum accelerations in the three directions are listed, together with the dates and the magnitudes of the earthquakes. In the column of remarks of the table H denotes the hypocentral depths and D means the distance from the epicenter to the stations. An example of the records triggered is shown in Fig. 4, which is a set of acceleration records obtained at Ukishima Park during the earthquake (M = 4.8) of September 30, 1970. Most of earthquake records including those in Table 1 are digitized, and their spectral characteristics are obtained²⁾.

4. DISTRIBUTION OF MAXIMUM ACCELERATIONS TO DEPTHS

The variation of maximum accelerations with respect to depths in the three components is illustrated in Fig. 5 for six typical earthquakes. Normalized are distribution of maximum accelerations which are obtained by averaging all data in Table 1. In the upper of the figure the maximum acceleration at the surface is taken as a unit, and in the lower of that at the deepest is taken as a unit. From the figures the following matters can be derived, as for the distribution of underground accelerations during moderate earthquakes.

- (1) Vertical distribution of underground accelerations with respect to depths varies with stations, directions, earthquake properties, etc.
- (2) Depths where the accelerations become one half of the surface accelerations at Futtsu Cape, Ukishima Park, Kannonzaki, and Ohgishima are about 70^m, 50^m, more than 120^m, and 150^m, respectively.
- (3) Accelerations at the deeper layers are rather close among the values at the four stations. Accelerations at the surface, however, differ extremely among the four stations. Furthermore, accelerations at the deeper layer of each station do not change largely with respect to depths.
- (4) Ratios of the surface acceleration to the deepest acceleration are large (2.5 to 3.5) at the soft clayey soil site (Ukishima Park), small (about 1.5) at the rocky site (Kannonzaki), and medium (1.5 to 3) at the sandy soil sites (Futtsu Cape and Ohgishima).
- (5) Vertical accelerations are generally smaller (1/3 to 1/2) than horizontal accelerations.

5. CHARACTERISTICS OF RESPONSE SPECTRA FOR UNDERGROUND MOTIONS

To show frequency characteristics of underground motions, response spectral curves for horizontal components of typical records are illustrated in Fig. 7³⁾. The abscissa denotes natural periods of a single-of-freedom system. The ordinate indicates the maximum absolute response accelerations of the system with damping ratio of 2%, when subjected to time dependent accelerations recorded at respective levels of each station. From these spectral curves the followings can be indicated.

(1) It is apparent that frequency characteristics of acceleration records obtained at one station during an earthquake are very similar among three or four layers of the station.

(2) By comparing (A) with (B) of Fig. 7, it is seen that frequency characteristics at one station considerably differ from records obtained during two different earthquakes. Fig. 7(A) shows spectra at Ukishima Park for a small and near earthquake ($M = 4.8$, $\Delta = 10$ km), and Fig. 7(B) shows spectra at the same station for a moderate and distant earthquake ($M = 7.2$, $\Delta = 320$ km). The characteristics of these two spectra are very different.

(3) By comparing spectra at two stations whose soil conditions are considerably different (for example, Fig. 7(B) and (D)) during an earthquake, it is seen that general characteristics are very close, and that responses in the range of longer periods are predominant at the soft soil site (Ukishima Park).

6. CONCLUSIVE REMARKS

From the analyses of underground seismic acceleration records obtained during moderate earthquakes with magnitudes of 4.8 to 7.2, the following conclusions can be drawn.

(1) The distribution of the maximum accelerations with respect to depths changes considerably with the change of soil conditions near the ground surface. Ratios of the surface acceleration to that at the deeper layer (110 to 150^m) are about 1.5 at a rocky ground, 1.5 to 3 at sandy grounds, and 2.5 to 3.5 at a very soft clayey ground.

(2) Although the acceleration values are smaller at deeper layers, frequency characteristics of underground seismic motions are very close to those of the surface motions.

(3) It seems provable that characteristics of earthquake ground motions are influenced by seismic conditions such as magnitudes of earthquakes, epicentral distances, as well as soil conditions at the site concerned.

ACKNOWLEDGEMENTS

The observation of underground seismic motions described in this article was realized with the strong promotion of Dr. Tadayoshi Okubo and Mr. Eiichi Kuribayashi at the Public Works Research Institute. The authors appreciate their achievements to attain this project. Acquisition of acceleration records was made by the staff members at the Kanto Regional Construction Bureau. The authors wish to thank them for their strenuous efforts.

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Table 1 Maximum Accelerations during 16 Moderate Earthquakes Since Sept., 1970 (in gals)

Eq. #	Date Magnitude Epicenter	Compo- nent	Futtsu Cape				Ukiashima Park				Kannonsaki			Ohgishima				Remarks
			0m	-70m	-110m		0m	-27m	-67m	-127m	0m	-80m	-120m	0m	-15m	-38m	-150m	
1	9.30.1970 M=4.8 Eastern Kanagawa-ken	NS EW UD	282 294 84	160 98 42	78 112 64	118 128 78	107 112 38	55 60 18	43 34 20									H=40km △=10~30km Mouth of Tama River
2	7.23.1971 M=5.3 Eastern Yamanashi-ken	NS EW UD	56 294 22	25 98 12	27 112 13	168 123 57	133 118 33	56 50 30	64 35 26	124 67 54	53 55 41	49 85 34						H=40 km △=50~80 km
3	2.29.1972 M=7.2 E Off Hachijo	NS EW UD	461 516 172	323 236 109	186 216 133					103 86 60	131 149 113	122 146 104						H=70 km △=300 km
4	10.18.1972 M=5.1 Northern Chiba-ken	NS EW UD	64 80 28	35 33 13	21 25 12	65 120 42	58 96 28	37 21 16	31 36 11	61 172 40	46 30 21	31 35 18						H=80 km △=50~70 km
5	11. 6.1972 M=5.1 SW Ibaraki-ken	NS EW UD	28 32 18	16 14 09	08 14 09	33 81 22	32 63 14	20 25 09	18 16 09									H=40 km △=50~90 km
6	12. 4.1972 M=7.2 E Off Hachijo	NS EW UD				200 156 41	146 113 36	74 71 30	64 47 30	65 73 —	101 106 —	104 107 —						H=50 km △=300~320 km
7	12. 8.1972 M=4.8 Northern Tokyo Bay	NS EW UD	123 70 40	72 36 18	32 85 21	101 134 37	93 104 24	53 42 16	44 33 15	83 168 61	50 59 25	42 53 25						H=90 km △=20~40 km
8	3.27.1973 M=4.9 Tokyo Bay	NS EW UD	475 47 71	307 123 65	74 137 58					186 315 85	259 267 93	168 — 54						H=60 km △=20~40 km
9	12.22.1973 M=5.0 Southern Chiba-ken	NS EW UD	105 76 34	85 43 18	26 78 23					130 184 63	64 73 30	60 74 39						H=70 km △=50~55 km
10	5. 9.1974 M=6.9 S Coast of Izu Pen.	NS EW UD	380 406 139	223 264 68	287 137 71					300 377 125	147 177 69	115 224 55						H=10 km △=10~120 km Off Izu Pen. Eq.
11	7. 8.1974 M=6.3 Off Ibaraki- ken	NS EW UD	41 33 23	24 19 17	09 20 13	42 33 15	20 07 10	29 13 10	10 17 10	35 36 20	19 19 19	15 20 12						H=40 km △=160~180 km
12	8. 4.1974 M=5.8 SW Ibaraki-ken	NS EW UD				136 129 58	106 77 31	65 55 20	51 44 19	125 150 45	65 71 45	49 65 32	121 82 107	71 60 47	85 108 —	62 48 34		H=50 km △=60~90 km
13	9.27.1974 M=6.5 SE Off Boso pen	NS EW UD	118 102 69	70 71 40	50 59 30	70 55 31	59 30 20	37 33 13	31 19 13				47 59 45	50 39 25	50 55 23	29 32 16		H=40 km △=240 km
14	11.16.1974 M=6.1 Near Choshi Chiba-ken	NS EW UD				48 48 17	40 26 12	30 22 16	28 17 12				54 36 31	34 42 28	29 33 29	18 19 14		H=40 km △=140 km
15	11.30.1974 M=7.5-7.75 Near Tori- shima	NS EW UD	116 85 47	85 58 31	40 45 28	62 53 23	70 50 17	25 29 13	28 22 11				62 62 37	32 34 22	56 68 24	42 36 15		H=350 km △=550 km
16	2. 8.1975 M=5.4 Northern Chiba-ken	NS EW UD				142 189 56	93 166 31	48 75 26	42 55 20									H=50 km △=50 km

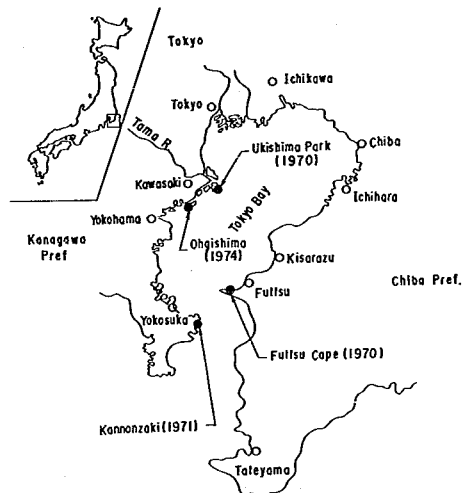


Fig. 1 Stations of Borehole Accelerometer Installation

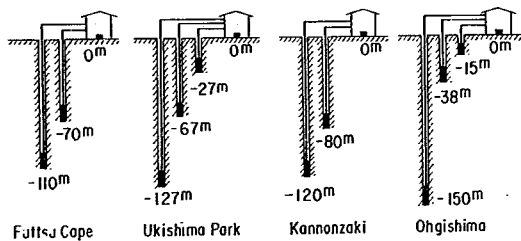


Fig. 2 Depths of Acceleration Pickups

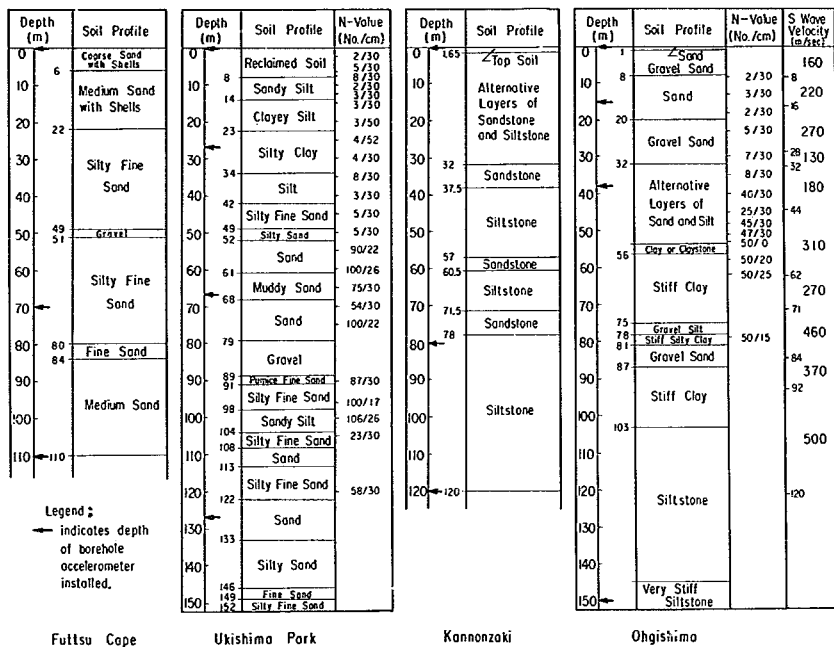


Fig. 3 Soil Profiles at Four Observation Stations

At Ukishima Park

(Sept. 30, 1970
35° 29' N, 139° 38' E
M = 4.8, H = 40 km

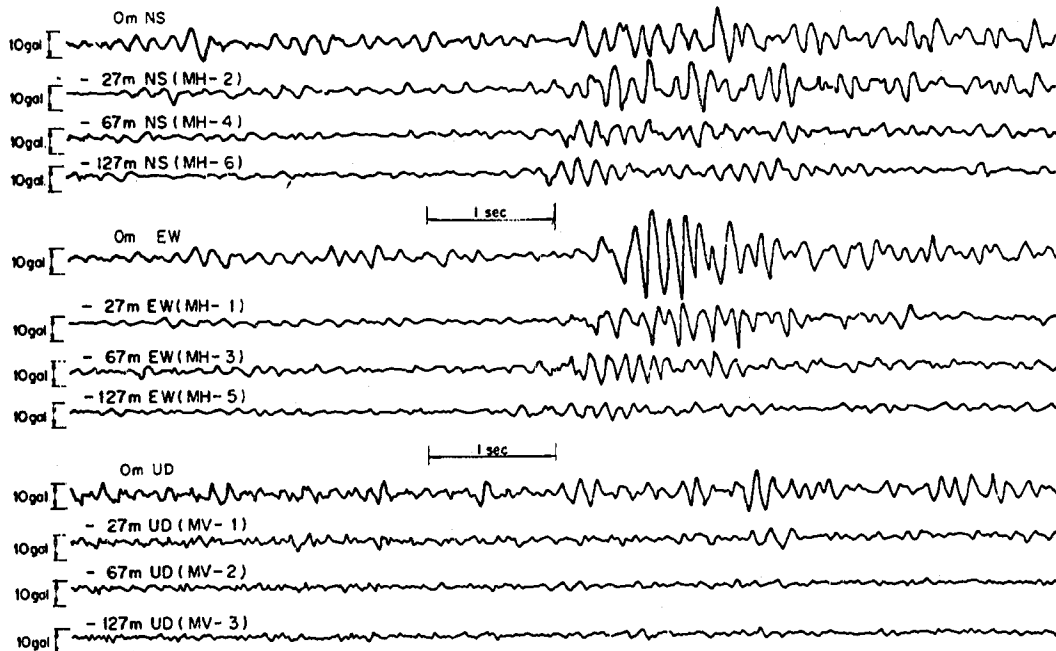


Fig. 4 Example of Records from Borehole Accelerometers

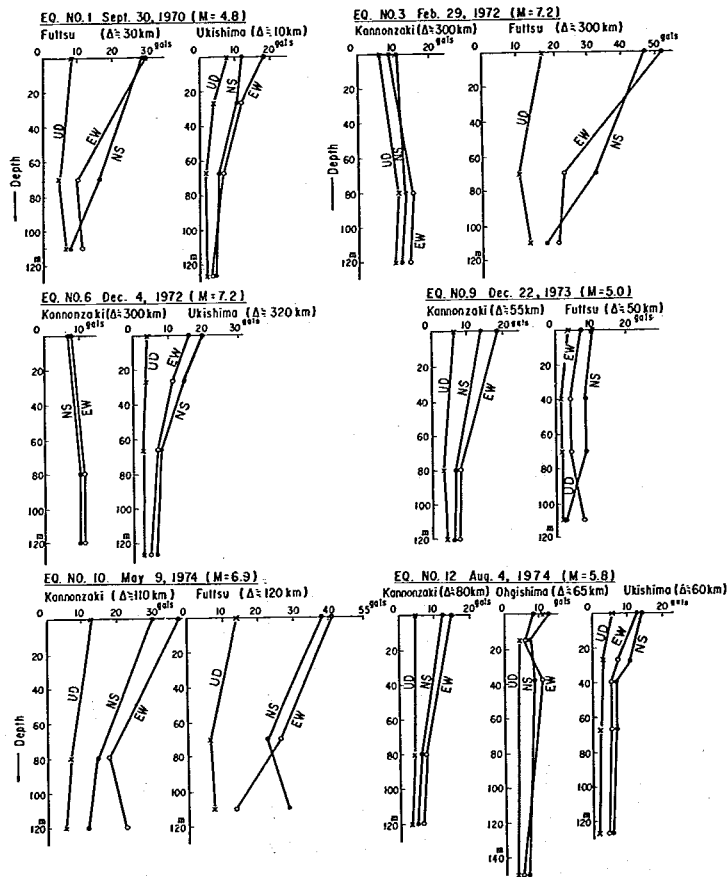


Fig. 5 Vertical Distribution of Maximum Accelerations during Six Earthquakes

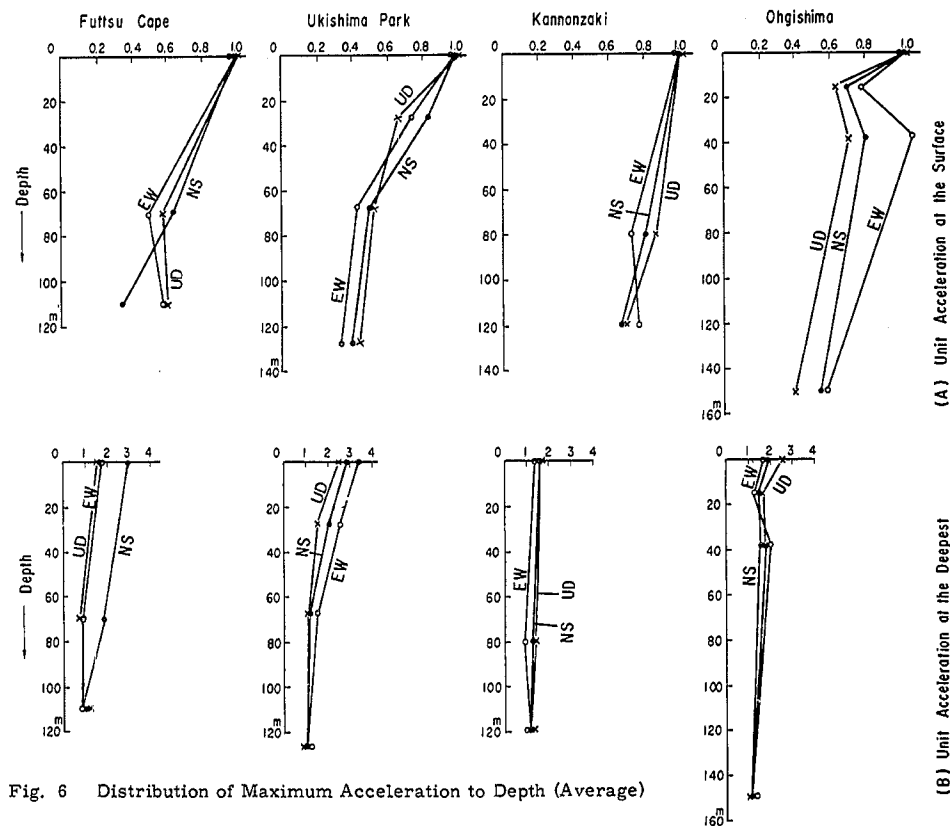
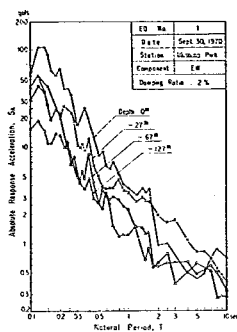
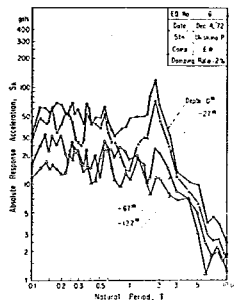


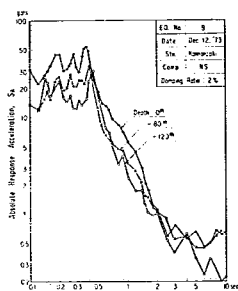
Fig. 6 Distribution of Maximum Acceleration to Depth (Average)



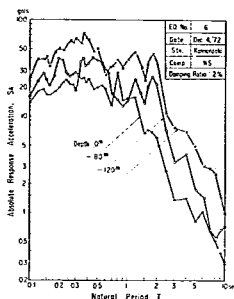
(A) Ukishima Park,
EQ. NO.1



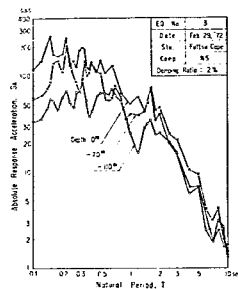
(B) Ukishima Park,
EQ. NO.6



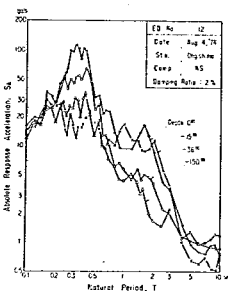
(C) Kannonzaki,
EQ. NO.9



(D) Kannonzaki,
EQ. NO.6



(E) Futtsu Cape,
EQ. NO.3



(F) Ohgishima
EQ. NO.12

Fig.7 Response Spectral Curves

A HISTORY OF SOIL LIQUEFACTION IN JAPAN

by

Eiichi Kuribayashi^I, Toshio Iwasaki^{II}, and Fumio Tatsuoka^{III}

SYNOPSIS

In order to obtain the relationship between occurrence of liquefaction phenomena and site conditions involved, a literature survey on liquefaction occurred during earthquakes in the last century in Japan was made, and site conditions at liquefied site were investigated. It is found that liquefaction of subsoils was induced during at least 44 earthquakes since 1872. Liquefied sites were limited only to alluvial deposits and reclaimed lands along seas or lakes. Seismic intensity at the liquefied sites was estimated to be 5 or higher on the JMA scale.

INTRODUCTION

During the past major earthquakes many engineering structures were damaged due to liquefaction phenomena of saturated sandy deposits. Correlations between liquefaction and site conditions were investigated at several regions in Japan^{1), 2), 3), 4)}. Kuribayashi and Tatsuoka^{5), 6)} performed a literature survey on liquefaction during 44 earthquakes in the last century throughout Japan. They obtained a distribution map for the whole Japan and regional precise maps for three areas, Kanto, Nobi, and Hokuriku. Furthermore detailed descriptions of liquefaction phenomena are provided. In these studies, liquefaction of subsoils is defined from the evidences such as 1) water spouting with sands and muds from wells or cracks on the ground, 2) sand boils or sand volcanoes, 3) excessive settlement of heavy structures constructed in sand layers, and 4) floating of wooden piles from rice paddy and floating of caissons under construction from river beds. This paper will discuss factors affecting the occurrence of liquefaction of soils, on the basis of the studies of the experienced history in Japan.

FACTORS AFFECTING LIQUEFACTION OCCURRENCE

Fig. 1 shows a relationship between magnitudes (M) of earthquakes and distances (R) from the epicenters to the most distant liquefied points. Numerals in the figure represent the earthquakes listed in Table 1 where earthquake names, dates, magnitudes, focal depths, epicenters, and number of victims are presented. All of these earthquakes induced liquefaction in certain areas. It is noted from Fig. 1 that the liquefaction area becomes wider as the magnitude of earthquake gets higher. From Fig. 1 an average relation between M and R can be expressed by

$$\log_{10} R = 0.87 M - 4.5 \quad (R ; \text{km}) \quad (1)$$

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and the lower bound is approximately expressed by

$$\log_{10} R = 0.77 M - 3.6 \quad (R; \text{km}) \quad (2)$$

for earthquakes with magnitudes larger than 6.0. It is suggested that liquefaction may occur only inside an area with radius of R given by eq. (2). For example, for $M = 6, 7$, and 8 the values of R are 10, 60 and 360 km, respectively.

One of factors affecting liquefaction may be the intensity of ground shaking. An area of highly intense ground motion will become wider for a larger earthquake. Therefore, the area where liquefaction occurred gets wider for a larger earthquake. It was found that seismic intensity on the JMA (Japan Meteorological Agency) scale was 5 or higher at liquefied sites, which corresponds to the maximum acceleration at the ground surface of 80 to 250 gals or higher.

In Fig. 2 shown are the equi-average maximum ground acceleration lines presented by the Public Works Research Institute⁷⁾ through the results of strong-motion observation for earthquakes which occurred in or near Japan. The average maximum acceleration (equal to 100 gals at base rock) obtained through the observation in the western part of the United States⁸⁾ is also indicated by a dotted line. Also shown in this figure is the lower bound line with points which represent the relationship between the magnitude (M) and the maximum epicentral distance of liquefied sites (R). It is noted that the slope of the lower bound line for liquefaction does not coincide with these of the equi-average maximum acceleration lines, and that along the lower bound line the average maximum acceleration becomes smaller for the larger earthquakes. On the other hand, the duration of the strong ground motion becomes longer for larger earthquakes and the predominant period of the ground motion becomes longer at distant sites from epicenters. It is inferred that the duration and the predominant period of ground motion shall be considered in assessing liquefaction potential as well as the maximum acceleration.

Another factor affecting liquefaction might be ground conditions.

Fig. 3 shows the relationship between the locations of liquefied sites, old and present riverbeds and soil conditions in the eastern part of Saitama Prefecture located north of Tokyo. This shows clearly that liquefaction occurred only in the alluvial deposit and densely in or along the old and present riverbeds. Boring logs and soil profiles at the representative liquefied zones are shown in Fig. 4 and Fig. 5, respectively. Hollow circles, hollow triangles and a solid circle in Fig. 5 represent liquefied sites and correspond to the marks in Fig. 3. From these figures, the followings can be found. First, the ground water levels are high at all of these liquefied sites. Furthermore, in the alluvial fan of Tone river, the representative section A-A' of which is shown in Fig. 5, liquefaction occurred in the alluvial sand layer (A_s) and in the alluvial sandy gravel layer (A_g). N -values (number of blows by the standard penetration test) of the sandy gravel layer are large as shown in Fig. 4. However, high N -values would be due to gravels and would not indicate the high density of the layer. In the middle reach of rivers where sandy banks are naturally formed liquefaction occurred in the upper alluvial sand layer

(As-1). The representative section B-B' is presented in Fig. 5. As shown in Fig. 4, N-values of this layer are about 10 to 20 which correspond to the medium density. In the delta of rivers, the representative profile C-C' of which is shown in Fig. 5, liquefaction occurred also in the upper alluvial sand layer (As-1). The density of soils in the delta is relatively loose with the N-values of about 2 to 10.

CONCLUSIONS

By surveying the relation between liquefaction phenomena and site conditions in Japan in this century, the followings are found.

- 1) A lower bound for liquefaction potential can be defined as a relation between epicentral distances and magnitudes of earthquakes.
- 2) To evaluate liquefaction potential, duration and predominant period of ground motion as well as maximum acceleration should be taken into account.
- 3) Liquefaction occurred exclusively at alluvial sandy deposits and reclaimed lands, especially severely at reclaimed lands where rivers, seas or lakes existed in the near past.

ACKNOWLEDGEMENTS

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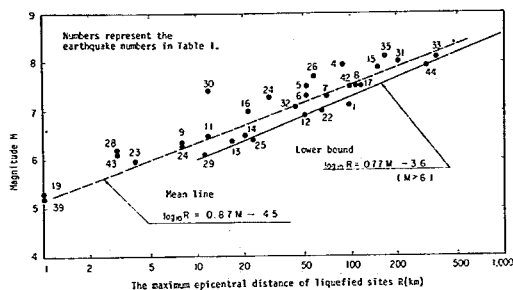


Fig. 1 Relationship between the maximum epicentral distance of liquefied sites R and magnitude M⁵⁾

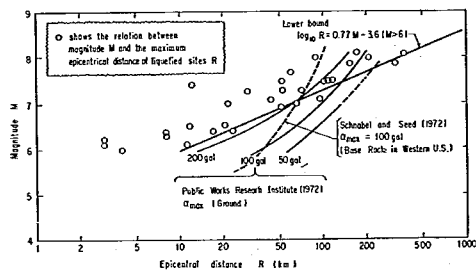


Fig. 2 Average maximum acceleration at the liquefied sites

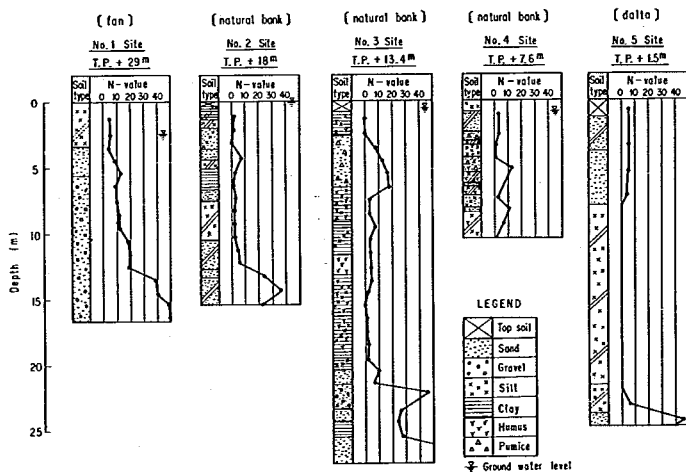


Fig. 4 Boring logs at the representative liquefied sites (See Fig. 3)

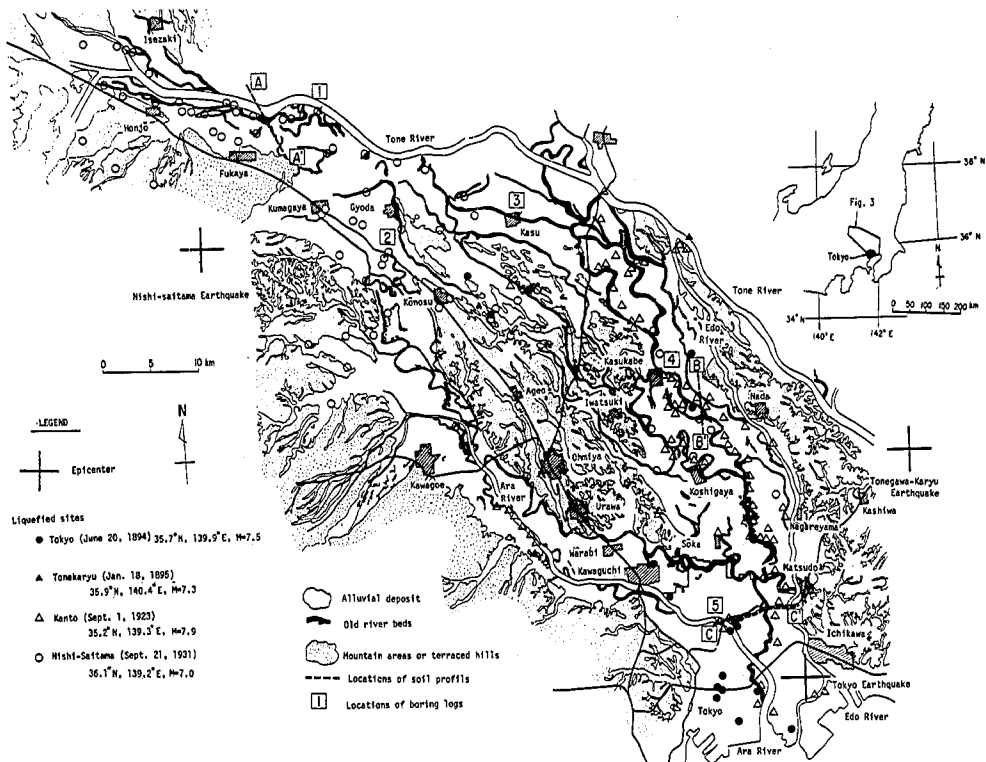


Fig. 3 Liquefied sites, old river beds and soil condition in the eastern Saitama Prefecture

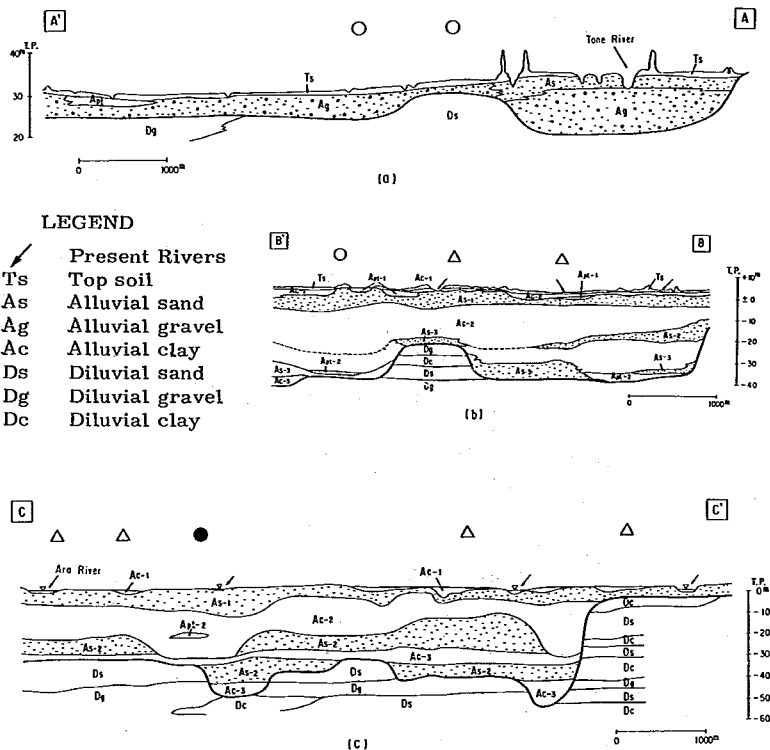


Fig. 5 Soil profiles of liquefied zones (See Fig. 3)

Table 1. Lists of earthquakes which induced liquefaction (1872 - 1968)

Earthquake	Year	Magnitude	Focal depth (km)	Epicenter	Persons killed	Year	Latitude	Longitude	Persons killed	Year	Latitude	Longitude	Persons killed
1. Kanto	1872.8.14	7.1		34.7°N, 132.9°E	102	23. Noto	36.1°N, 136.7°E	10	10	1951.9.21	36.1°N, 136.7°E	10	10
2. Kishinoue	1897.7.22	6.1		37.7°N, 139.9°E	0	24. Shinshu	35.5°N, 137.0°E	3	3	1933.8.21	35.5°N, 137.0°E	3	3
3. Kanto	1906.7.28	6.3		35.7°N, 139.7°E	30	25. Kawachi	35.7°N, 139.7°E	9	9	1935.7.11	35.7°N, 139.7°E	9	9
4. Noto (Mino-Owari)	1901.10.28	8.4(7.8)		35.7°N, 139.7°E	1,073	26. Kanto	35.7°N, 139.7°E	9	9	1936.7.21	35.7°N, 139.7°E	9	9
5. Tokyo	1904.6.20	7.5		35.7°N, 139.7°E	31	27. Oga	35.7°N, 139.7°E	0	0	1906.11.9	35.7°N, 139.7°E	0	0
6. Shonai	1904.10.22	7.5		35.7°N, 139.7°E	720	28. Nagano	35.7°N, 139.7°E	27	27	1908.5.1	35.7°N, 139.7°E	27	27
7. Tsuchiura	1905.1.18	7.3		35.7°N, 140.4°E	9	29. Fukuoka	35.7°N, 139.7°E	5	5	1913.7.15	35.7°N, 139.7°E	5	5
8. Kanto	1906.8.10	7.5		35.7°N, 140.4°E	259	30. Fukuoka	35.7°N, 139.7°E	0	0	1913.8.4, 8.5	35.7°N, 139.7°E	0	0
9. Kanto	1907.1.17, 4.30	6.3, 6.9		35.7°N, 139.7°E	0	31. Tokyo	35.7°N, 139.7°E	1,003	1,003	1914.8.10	35.7°N, 139.7°E	1,003	1,003
10. Mino Utsunomiya	1908.5.26	6.7		35.7°N, 139.7°E	0	32. Tokyo	35.7°N, 139.7°E	999	999	1914.12.7	35.7°N, 139.7°E	999	999
11. Tokyo	1908.8.13, 8.12	6.5, 6.5		35.7°N, 139.7°E	0	33. Tokyo	35.7°N, 139.7°E	1,901	1,901	1915.1.13	35.7°N, 139.7°E	1,901	1,901
12. Goto (Aomori)	1908.8.14	6.9		35.7°N, 139.7°E	41	34. Tokyo	35.7°N, 139.7°E	1,300	1,300	1915.3.4	35.7°N, 139.7°E	1,300	1,300
13. Utsunomiya	1911.3.15	6.4		35.7°N, 140.4°E	54	35. Tokyo	35.7°N, 139.7°E	7	7	1915.3.7	35.7°N, 139.7°E	7	7
14. Shinshu	1922.11.6	6.5, 6.9		35.7°N, 139.7°E	20	36. Tokyo	35.7°N, 139.7°E	1	1	1915.7.27	35.7°N, 139.7°E	1	1
15. Kanto	1923.8.1	7.9		35.7°N, 139.7°E	91,321	37. Tokyo	35.7°N, 139.7°E	0	0	1915.10.19	35.7°N, 139.7°E	0	0
16. Tokyo	1925.5.23	7.0		35.7°N, 139.7°E	428	38. Tokyo	35.7°N, 139.7°E	5	5	1916.7.2	35.7°N, 139.7°E	5	5
17. Kanto	1927.7.7	7.5		35.7°N, 139.7°E	2,925	39. Tokyo	35.7°N, 139.7°E	2	2	1916.7.2	35.7°N, 139.7°E	2	2
18. Tokyo	1927.8.6	7.5		35.7°N, 139.7°E	0	40. Tokyo	35.7°N, 139.7°E	3	3	1916.7.2	35.7°N, 139.7°E	3	3
19. Tokyo	1927.10.27	5.3	0-10	35.7°N, 139.7°E	0	41. Tokyo	35.7°N, 139.7°E	3	3	1916.7.2	35.7°N, 139.7°E	3	3
20. Kanto	1928.10.17	7.0		35.7°N, 139.7°E	277	42. Tokyo	35.7°N, 139.7°E	48	48	1916.7.2	35.7°N, 139.7°E	48	48
21. Kanto	1930.11.30	7.0	0-5	35.7°N, 139.7°E	277								

DYNAMIC SOIL PROPERTIES WITH EMPHASIS ON COMPARISON OF LABORATORY TESTS AND FIELD MEASUREMENTS

by

Toshio Iwasaki^I, Fumio Tatsuoka^{II}

SYNOPSIS

To analyze ground motions during earthquakes, it becomes necessary to obtain strain amplitude-dependent shear moduli and damping coefficients of soils. At two sites, insitu seismic surveys were carried out. Sands sampled from these sites were tested with resonant-column apparatus to obtain shear moduli at small strain. Laboratory test results showed that these two natural sands have smaller shear modulus than uniform clean sands. The comparison of shear moduli from shear wave velocities and those from resonant-column tests was performed and an excellent coincidence was obtained for the both sites.

SHEAR MODULUS G BY RESONANT-COLUMN METHOD

Shear moduli G of sands were obtained with a resonant-column apparatus developed at the Public Works Research Institute(2), (3). This is of Drnevich-type⁽¹⁾ with a hollow cylindrical sample of 25 cm in height, 10 cm in outside diameter and 6 cm in inside diameter. The confining pressure equally applied to the outside and to the hollow of the specimen is supplied by air pressure and the axial load can be applied independently of the confining pressure. With this apparatus shear moduli in the range of shear strain of 10^{-6} to 5×10^{-4} were obtained.

Listed in Table 1 are sands tested in this study. These sands can be divided into two groups: (i) Clean sands which do not include fine particles smaller than 0.074 mm in diameter and are poor graded. They are made by sieving several natural sands, and (ii) Natural sands, Iruma-sand and Ohgi-shima-sand, which are well graded and include finer particles. Shown in Fig. 1 are gradings of sands tested where TO, OS, ... are the abbreviations of the names of sands listed in Table 1. Shear moduli of air dry clean sands at single shear strain amplitude of 10^{-6} , 10^{-5} and 10^{-4} are shown in Figs. 2 through 4 where p is the mean principal stress denoted by $(\sigma_a + 2\sigma_r)/3$ (σ_a is axial stress and σ_r is radial stress). These figures show that shear moduli G of clean sands tested can be represented approximately by the following empirical equations irrespectively of the kinds of sands.

$$(\gamma = 10^{-6}) \quad G = 900 \frac{(2.17 - e)^2}{1 + e} p^{0.38} \quad (1)$$

$$(\gamma = 10^{-5}) \quad G = 850 \frac{(2.17 - e)^2}{1 + e} p^{0.44} \quad (2)$$

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$$(\gamma = 10^{-4}) \quad G = 700 \frac{(2.17 - e)^2}{1 + e} p^{0.50} \quad (3)$$

where G is shear modulus in kg/cm^2 , p is mean principal stress in kg/cm^2 and e is void ratio. Eq. (3) is identical to the empirical equation for round Ottawa-sand proposed by Hardin, et al⁽⁴⁾.

COMPARISON OF LABORATORY TESTS AND FIELD MEASUREMENTS

In situ borehole seismic surveys were carried out at two sites: Iruma, Minami-Izu-cho and Ohgi-shima, Kawasaki-shi. At the former site, a sandy embankment was damaged by the Off-Izu-peninsula earthquake of 1974. The latter site is a reclaimed land where the borehole accelerometers are installed down to the depth of about 120 m. At Iruma, borehole seismic surveys were carried out at three points. One of the soil properties are shown in Fig. 5. Shear wave velocity measurements were performed at each 2 m depth. Resonant-column tests were performed on sands taken from the deposit just below the ground surface. As the sand deposit is almost uniform to the depth of about 10~20 m, the sand near the ground surface could represent properties of the whole sand deposit. Tests were carried out on dense air-dry and saturated samples which were completely disturbed. Samples were made by pouring air-dry sand through air into a split mold or by pouring saturated sand into a mold filled with de-aired water. Densification was performed by tamping the split mold with a wooden hammer. Consolidation time was about 1 hr. Considering that the sand deposit is young and plasticity index of the sand is almost zero, the effect of cementation and delayed consolidation could be negligible⁽⁵⁾. Therefore shear modulus of the sand deposit could be estimated from the test results on completely disturbed specimens. Illustrated in Fig. 6 are shear moduli at $\gamma = 10^{-6}$, 10^{-5} and 10^{-4} of Iruma sand, which are smaller than those represented by eqs. (1) through (3). It is to be noted that shear moduli of Iruma sand A, C and Z3 which were made by sieving original Iruma Sand are represented by eqs. (1) through (3) as shown in Figs. 2 through 4. This means that shear moduli of sands are affected by grading or content of fine particles. To obtain shear moduli of sand deposits from empirical equations of Iruma sand, it is necessary to estimate effective mean principal stress p' and void ratio e in the ground. First, relative density of every 1 m thick layer was estimated from the relationship of effective overburden pressure σ'_v and N -value proposed by Gibbs and Holtz⁽⁶⁾. In the estimation extremely high N -value, provably due to the existence of gravels, were omitted. Secondly, to estimate e , both values of maximum and minimum void ratios were obtained by the method proposed by Yoshimi and Tohno⁽¹⁰⁾. Thirdly, the unit weight of sand γ_t was estimated. Then, effective mean principal stress p' was calculated supposing the earthpressure coefficient at rest to be 0.5. Lastly, shear moduli at $\gamma = 10^{-6}$ and 10^{-4} were obtained by substituting the estimated p' and e into the empirical equations for shear moduli of Iruma sand shown in Fig. 6. Shown in Fig. 7 are the shear moduli from shear wave velocities V_s which is denoted by solid vertical lines and values of shear moduli at $\gamma = 10^{-6}$ and $\gamma = 10^{-4}$ obtained by laboratory tests which are represented by the black circles (G at

$\gamma = 10^{-6}$) and black triangles (G at $\gamma = 10^{-4}$). The solid curve in Fig. 7 indicates shear moduli estimated from these two laboratory-test values of G at $\gamma = 10^{-6}$ and 10^{-4} considering that shear strain amplitudes occurred in the ground during borehole seismic survey decrease with depth on the order of about 10^{-7} to 10^{-5} . It is apparent from this figure that the agreement of the values of shear moduli predicted by laboratory tests with the values by shear wave velocities is satisfactory. Almost same result was obtained for other points at Iruma as shown in Fig. 8. On the other hand, shear moduli obtained from empirical equations for "clean" sands, eqs. (1) through (3), using estimated values of p' and e are also plotted in Figs. 7 and 8 with marks white circles (G at $\gamma = 10^{-6}$) and white triangles (G at $\gamma = 10^{-4}$). Predicted values by eqs. (1) through (3) are much larger than the values from shear wave velocities.

A similar survey was also carried out at Ohgi-shima (Fig. 9). Resonant-column tests were performed for samples taken from just below the ground surface (Fig. 10). Ohgi-shima was reclaimed recently with sands from Sengenyama located in Chiba-prefecture. It is also to be noted that Sengenyama sand A, B and C which have the shear moduli expressed by eqs. (1) through (3) as illustrated in Figs. 2 through 4. From Fig. 11 it is also seen that shear moduli predicted by laboratory tests are almost identical to those obtained by field measurements and that shear moduli estimated by empirical equations of clean sands are overestimated.

CONCLUSIONS

The principal conclusions drawn from this study are as follows.

- (1) Shear moduli of each shear strain level of clean sands can be expressed by identical empirical equations irrespectively of sands tested.
- (2) Shear moduli of natural sands which are well-graded and contain silty and clayey particles are smaller than those of clean sands.
- (3) Shear moduli obtained from laboratory tests is almost identical to those obtained by borehole seismic surveys.
- (4) Shear moduli in grounds estimated by laboratory tests vary more naturally with respect to depth when comparing with the variation of shear moduli obtained by seismic surveys.

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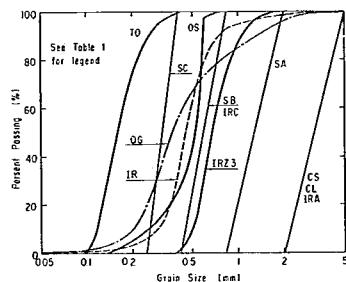


Fig. 1 Gradings of Sands Tested

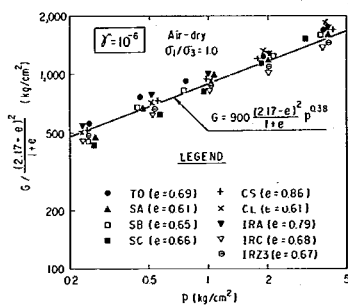


Fig. 2 Shear Moduli at $\gamma = 10^{-6}$ of Clean Sands

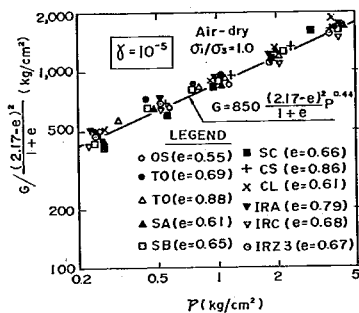


Fig. 3 Shear Moduli at $\gamma = 10^{-5}$ of Clean Sands

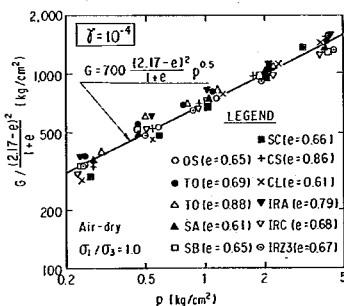


Fig. 4 Shear Moduli at $\gamma = 10^{-4}$ of Clean Sands

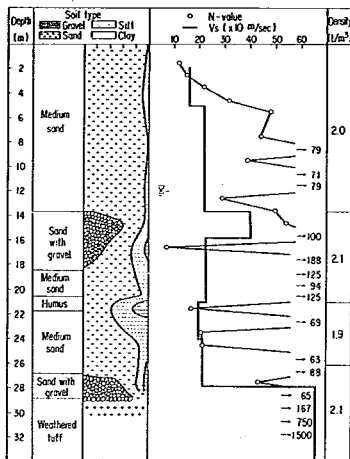


Fig. 5 Soil Profile of Point No. 1 at Iruma

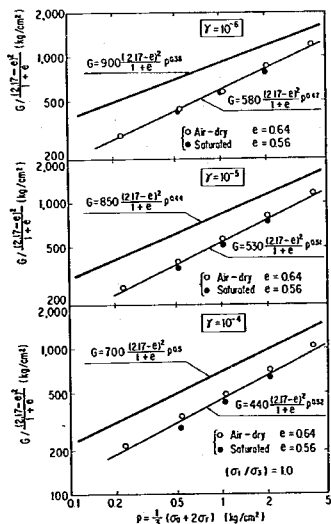


Fig. 6 Shear Moduli of Iruma Sand

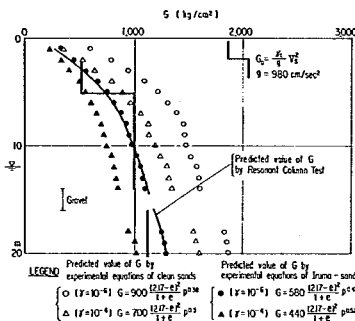


Fig. 7 Comparison of Shear Modulus by Laboratory Tests and Field Measurements at Point No. 1 at Iruma

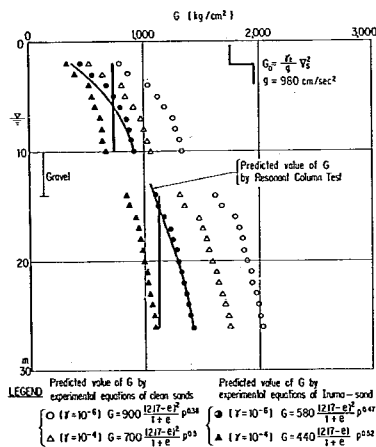


Fig. 8 Comparison of Shear Moduli by Laboratory Tests and Field Measurements at Point No. 3 at Iruma

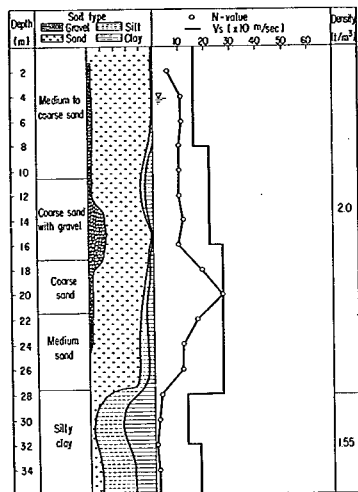


Fig. 9 Soil Profile of Ohgi-shima

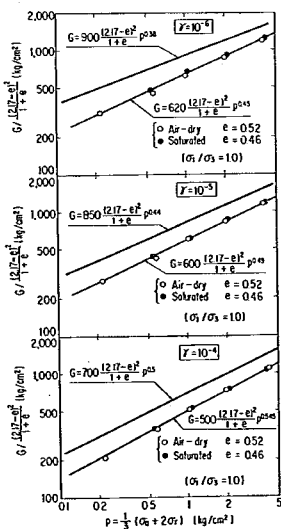
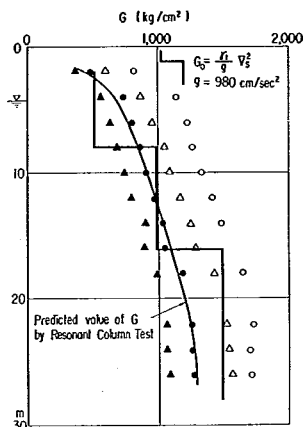


Fig. 10 Shear Moduli at $\gamma = 10^{-6}$, 10^{-5} and 10^{-4} of Ohgi-shima Sand



LEGEND Predicted value of G by experimental equations of clean sands
 $\circ (\gamma = 10^{-6}) G = 900 \frac{(12/17 - e)^2}{1 + e} p^{0.58}$
 $\triangle (\gamma = 10^{-4}) G = 700 \frac{(12/17 - e)^2}{1 + e} p^{0.5}$
 Predicted value of G by experimental equations of Ohgi-shima-sand
 $\bullet (\gamma = 10^{-6}) G = 620 \frac{(12/17 - e)^2}{1 + e} p^{0.45}$
 $\blacktriangle (\gamma = 10^{-4}) G = 500 \frac{(12/17 - e)^2}{1 + e} p^{0.58}$

Fig. 11 Comparison of Shear Modulus by Laboratory Tests and Field Measurements at Ohgi-shima

Table 1. List of Materials^{a)}

Material	C_u	D_{50} (mm)	D_{60} (mm)	U_c	$e_{max}^b)$	$e_{min}^b)$
Toyoura-sand (70%) sub-angular, uniform	2.84	0.12	0.175	1.45	0.96	0.64
Sengosayama-sand A (54) uniform	2.65	$0.85 \sim 2.9^{c)}$	-	-	0.82	0.61
" B (54) "	2.72	$0.40 \sim 0.85^{c)}$	-	-	0.87	0.67
" C (56) "	2.68	$0.25 \sim 0.40^{c)}$	-	-	0.88	0.65
Crushed sand-stone (CS) angular, uniform	2.71	$2.0 \sim 5.0^{d)}$	-	-	1.09	0.80
Crushed lime-stone (CL) sub-angular, uniform	2.74	$2.0 \sim 5.0^{d)}$	-	-	0.83	0.64
50-50 Ottawa-sand (OS) round, *) uniform	2.66	0.5	0.54	1.4	0.80	0.48
Ohgi-shima-sand (OS) sub-angular, well-graded	2.69	0.2	0.45	2.25	0.72	0.49
Iruma-sand (IR) angular, well-graded	2.76	0.28	0.56	2.0	0.84	0.57
Iruma-sand A (IRa) angular, uniform	2.67	$2.0 \sim 5.0^{d)}$	-	-	1.06	0.79
Iruma-sand C (IRC) angular, uniform	2.83	$0.4 \sim 0.85^{d)}$	-	-	0.96	0.66
Iruma-sand B (IRB) angular, uniform	2.79	0.5	0.74	1.48	0.94	0.68

a) All tests were conducted under $e_0/e_1 = 10$ condition.

b) e_{max} and e_{min} were obtained on air-dry condition.

c) U_c , S_u , C_u , ... are abbreviation for materials tested.

d) Range of diameter.

e) After Drnevich and Richart (1970).^{4b)}

STATISTICAL ANALYSIS OF STRONG-MOTION ACCELERATION RECORDS

BY

Masamitsu Ohashi^I, Toshio Iwasaki^{II}, Susumu Wakabayashi^{III}, and Ken-ichi Tokida^{III}

SYNOPSIS

This paper briefly introduces the present status of strong-motion observation for engineering structures in Japan. Next, it presents the results of multiple regression analysis of 301 strong-motion acceleration records, to evaluate the effects of earthquake magnitude, epicentral distance and subsoil condition on characteristic variables of ground accelerations such as maximum horizontal acceleration, duration of major motion, ratio of vertical to horizontal accelerations, etc. The paper also shows the results of a quantification analysis of average response accelerations obtained from 277-component horizontal acceleration records, to clarify the effects of seismic and subsoil conditions on average response spectra.

From the analysis done, empirical formulas which can statistically estimate maximum horizontal acceleration, duration of major motion, and number of zero-crossing in terms of earthquake magnitude, epicentral distance, and subsoil condition, are proposed. Frequency characteristics of horizontal motions and ratios of vertical to horizontal accelerations are averagedly evaluated depending on subsoil conditions. Furthermore, various average response spectrum curves for a linear single-degree-of-freedom system are proposed in terms of earthquake magnitude, epicentral distance, and subsoil condition.

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51. INTRODUCTION

In establishing reasonable procedures for earthquake resistant design of structures, it seems essential to properly evaluate characteristics of ground motions and structural behavior during strong earthquakes. A number of strong motion accelerographs are installed at various engineering structures and on neighboring grounds in Japan to grasp dynamic properties of ground motions and structural responses. The strong-motion observation network for engineering structures in this country has progressed after experiencing the Niigata Earthquake of 1964 and the Tokachi-oki Earthquake of 1968. At the present more than 1,000 sets of strong-motion accelerographs are equipped at structures and on grounds near the structures, and many of meaningful acceleration records are already available. Although the records triggered include those of structural responses, only records on ground surfaces are employed in the present analysis.

Characteristic variables of acceleration records and response spectral accelerations of a linear single-degree-of-freedom system computed from ground accelerations are statistically investigated. Records used are classified into four groups, depending on the subsoil conditions at the observation stations, from rocky ground to soft alluvial ground.

Although there will be a number of factors which affect characteristics of ground motions, it seems probable that three principal factors might be earthquake magnitude, geographical situation of the focal area of an concerned earthquake related to the structural site, and subsoil condition at the site. Accordingly, in this study characteristic variables of ground accelerations are statistically analyzed in terms of these three factors. It is expected that the results of the analysis will give the designer invaluable information when conducting seismic design of structures.

The study shown in this paper was performed as one of comprehensive research works done in the Aseismic Technology Development Committee established at the Technology Center for National Land Development with a commission from the Public Works Research Institute. The results are utilized as basic materials in standard design seismic forces and seismic loads of "New Criteria of Earthquake Resistant Design (Draft)," 1), 2) proposed by the Ministry of Construction in March, 1977.

§2. OBSERVATION OF STRONG-MOTION EARTHQUAKES AT ENGINEERING STRUCTURES IN JAPAN

In Japan the observation of strong-motion earthquakes for engineering structures was initiated when SMAC-type accelerographs were developed in 1953. The observation network has gradually advanced with experiences of severe structural damages due to the Niigata Earthquake of 1964 and the Tokachi-oki Earthquake of 1968. As of March, 1976, the number of SMAC accelerographs installed on various structures and neighboring grounds is 1048 totally, as shown in Fig. 1 and Table 1. It is believed, however, that the same number of accelerographs are necessitated in order to obtain reliable records for major earthquakes which might take place at any areas of this country.

At the present about one third of installed accelerographs are set up on the ground surfaces, and the rest on structures. Fig. 1 illustrates the present network of strong motion observation for various structures. Installation of the instruments is generally conducted by individual organizations such as governmental organs and private companies which are in charge of construction and maintenance of the structures.

In addition to the stations with strong-motion accelerographs, there are a number of stations where electromagnetic-type seismographs are equipped. Moreover, dynamic behavior of subsoils during earthquakes is also being measured using downhole seismometers at several points (some at depths greater than 100 m) as well as standard seismometer and SMAC accelerograph measurements on the surface. These measurements will provide pertinent information for establishing seismic design methodology of engineering structures.

Records obtained from these field measurement programs have been published periodically by institutions concerned³⁾. Records obtained at port and harbor structures are digitized, analyzed and published at the Port and Harbour Research Institute. Records triggered at highway bridges, tunnels, dams, and embankments are processed mainly at the Public Works Research Institute. Furthermore, records at building structures are treated mostly at the Building Research Institute.

Incidentally, in order to promote the strong-motion observation program

in Japan, the Strong-Motion Observation Council is established at the National Research Center for Disaster Prevention, Science and Technology Agency, with a cooperation of universities, governmental organs, public corporations, and private companies. The council has been preparing a nation-wide strong-motion observation program, and also has been publishing important acceleration records collected from overall engineering fields covering the above-mentioned structures and others.

§3. ANALYSIS OF ACCELERATION RECORDS

A statistical analysis of measured acceleration records was attempted to quantitatively express characteristic variables of ground motions as functions of seismic conditions and subsoil properties at the observation point. Records analyzed were extracted from the publications issued at the Earthquake Research Institute of the University of Tokyo, the National Research Center for Disaster Prevention of Science and Technology Agency, Port and Harbour Research Institute of the Ministry of Transport, the Public Works Research Institute of the Ministry of Construction, and the Railway Technical Research Institute of the Japanese National Railways.

In the present analysis were employed those records which were obtained during earthquakes with the Richter magnitude of 5.0 or higher and the hypocentral depth of 60 km or shallower, and which include at least one record with the maximum acceleration of 50 gals or higher for an earthquake. Records with the maximum acceleration less than 10 gals were excluded. The total number of the records used is 301 (the number of components treated is $3 \times 301 = 903$) for 51 earthquakes.

Factors considered in the analysis are as follows:

- (1) Seismic properties: Richter magnitude M , epicentral distance Δ (km)
- (2) Subsoil conditions: Observation stations are classified into the following four groups depending on subsoil conditions, with reference of the stipulation in "Specifications for Earthquake Resistant Design of Highway Bridges," Japan Road Association^{5),6)}.

- 1st Group: Rock
- 2nd Group: Diluvium
- 3rd Group: Alluvium (Except the 4th Group)
- 4th Group: Soft Alluvium

The definition of this classification is described in Table 2.

(3) Characteristic variables of acceleration records:

- (a) Absolute maximum horizontal acceleration (larger value of two perpendicular horizontal motions) : $H_{\max}(\text{gals})$
- (b) Period of vibration about the time when acceleration becomes $H_{\max}$: $T_1(\text{sec})$
- (c) Duration of major motion : $T_d(\text{sec})$
- (d) Ratio of vertical to horizontal accelerations : $v = V_{\max}/H_{\max}$
- (e) Number of zero-crossing during the time interval of T_d : N_z
- (f) Mean period of vibration during the time interval of T_d : $T_m = 2T_d/N_z$

The procedure for determining the above characteristic variables are illustrated in Fig. 2.

Each of the characteristics values (expressed as X below) was assumed to be represented by the following two types of expressions:

$$\text{Type-1 : } X = a_1 \cdot 10^{b_1 M} \cdot \Delta^{c_1} \quad (1)$$

$$\text{Type-2 : } X = a_2 + b_2 M + C_2 \Delta \quad (2)$$

where X: characteristic value (namely, $H_{\max}$, T_1 , T_d , N_z , and T_m)

M : Earthquake magnitude on the Richter scale

Δ : Epicentral distance (km)

a_1 , b_1 , c_1 ; a_2 , b_2 , c_2 : constants for each characteristic value

The analysis was conducted for five different cases, namely four groups of subsoil conditions plus the average subsoils including the entire data.

In determining the expressions for $H_{\max}$, only eq. (1) was investigated and the term of Δ in eq. (1) was substituted by $(\Delta + \Delta_0)$, where Δ_0 was taken as 0, 10, 20, 30 and 40 km.

Table 3 indicates the distribution of 301 records used in the analysis. Table 4 shows formulated regression equations and correlation coefficients, which are obtained from the analysis based on the multiple regression analysis. In the table are shown the results of three characteristic values $H_{\max}$, T_d and N_z in which multiple correlation coefficients are greater than 0.5. Since correlation coefficients for expressions of Type-1 were generally greater than those for Type-2, the results for only Type-1 are shown in the table. Although the correlation coefficient was the highest in the cases of $\Delta_0=10$ km among the five cases analyzed, in the table $H_{\max}$ in the cases of $\Delta_0=0$ is expressed for simplicity. Fig. 3 (A) ~ (E) illustrates the expressions for $H_{\max}$ in the cases of $\Delta_0=0$ as functions of M and Δ , for the four groups of subsoil conditions and the average subsoils.

As for the other three characteristic values T_1 , v and T_m , the multiple correlation coefficients are found to be less than 0.5. This means that there is no distinct relation among those values and M and Δ . For these variables the mean values and the standard deviations are evaluated, and are listed in Table 5.

The results of the analysis described in the above may be summarized as follows:

- (1) The maximum horizontal acceleration ($H_{\max}$), the duration of major motion (T_d), and the number of zero-crossing (N_z) can be approximately expressed as functions of earthquake magnitude and epicentral distance, as shown in Table 4.
- (2) The maximum horizontal acceleration ($H_{\max}$) gets larger as the earthquake magnitude (M) becomes higher and the epicentral distance (Δ) becomes shorter. The average values of $H_{\max}$ for $M=8$ and $\Delta=50$ km are estimated 130, 270, 200 and 400 gals for rock, diluvium, alluvium, and soft alluvium, respectively. The average values of $H_{\max}$ for $M=8$ and $\Delta=100$ km are estimated 85, 150, 110 and 200 gals for the four subsoil conditions, respectively.

The decreases in $H_{\max}$ with the epicentral distance are not considerable for rock comparing with the other three subsoils.

- (3) The duration of major motions (T_d) defined in Fig. 2 gets longer as the earthquake magnitude (M) becomes greater and as the epicentral distance (Δ) becomes longer. Fig. 4 (A) and (B) show the relation between Δ and T_d for $M=8$, and the relation between M and T_d for $\Delta=100$ km, respectively. It is seen from those results that the duration of major motions is between 10 and 30 seconds for ground motions of earthquakes with the Richter magnitude between 7 and 8 at the epicentral distance of 100 km.
- (4) The relation between the number of zero-crossing (N_z) and the epicentral distance (Δ) is considerably affected by subsoil conditions. The value of N_z increases with the increase in Δ for harder grounds such as rocky and diluvial grounds. It seems that N_z is not affected by Δ but M , for softer grounds. The number of repetitions of major motions (N) which can be approximately $N_z/2$, is 30 to 100 for the various subsoil conditions.
- (5) The period of vibration about the time of occurring of $H_{\max}$ (T_1), the ratio of vertical to horizontal accelerations (v), and the mean period of vibration during the major motion (T_m) are not clearly related with M and Δ . The following two points, however, can be seen from Table 5.
 - (a) The mean values of T_1 and T_b get longer as subsoils become softer. This tendency is obvious for soft alluvial grounds. The ratio of T_1 to T_m is 1.20 to 1.26, and the ratio is almost constant, irrespectively of subsoil conditions.
 - (b) The mean value of v being 0.32 to 0.34 is not strongly affected by subsoil conditions. Since the standard deviation is 0.15 to 0.21, it may be reasonable to assume that the vertical acceleration is equal to the half of the horizontal acceleration for the designing purpose. Fig. 5 illustrates an example of the relation between Δ and v . In the figure the mean value of v is equal to 0.32.

54. ANALYSIS OF EARTHQUAKE RESPONSE SPECTRA

The Port and Harbour Research Institute, the Ministry of Transport and the Public Works Research Institute, the Ministry of Construction have been conducting response spectrum analysis for major ground motions obtained at various grounds during moderate to strong earthquakes^{7),8)}. In the present analysis the results of the response spectra done at the two Institutes for 277 horizontal components of ground motions, which were triggered during 68 earthquakes with the Richter magnitude between 4.5 and 7.9 and the hypocentral depth of 60 km or shallower.

Factors considered in the analysis are as follows:

- (1) Seismic properties: Earthquake magnitude on the Richter scale (M), and epicentral distance (Δ in km)
- (2) Subsoil conditions: Observations are classified into four, in the same manner as shown in §3 (See Table 2 for the definition of the classification of subsoils)
- (3) Characteristic variables of acceleration records and their response spectral values:
 - (i) Maximum horizontal acceleration (two perpendicular components for one record): A_{max} (gals)
 - (ii) Absolute response acceleration: S_A (gals), which is computed from digitized values of triggered records by assuming a linear viscously damped single-degree-of-freedom system with the natural periods of T (=0.1, 0.15, 0.2, 0.25, 0.3, 0.35, 0.4, 0.5, 0.6, 0.7, 0.8, 0.9, 1, 1.5, 2, 2.5, 3, and 4 seconds) and the damping ratio of h (=5% of critical).

In the analysis for estimating S_A and A_{max} are assumed the followings.

- (a) A quantification analysis can be applied to the estimation of S_A in terms of earthquake magnitude (M), epicentral distance (Δ), and subsoil conditions (S), by assuming an expression as

$$\log_{10} \bar{S}_A = C(M) + C(\Delta) + C(S) \quad (3)$$

where

$\bar{S}_A$: Estimated value of S_A (gals)

$C(M)$, $C(\Delta)$, and $C(S)$: Weighting functions depending on M , Δ , and S , respectively.

- (b) A quantification analysis can be also applied to the estimation of $A_{\max}$ in terms of the three factors in the above, by assuming an expression of

$$\log_{10} \bar{A}_{\max} = C'(M) + C'(\Delta) + C'(S)$$

where

$\bar{A}_{\max}$: Estimated value of $A_{\max}$ (gals)

$C'(M)$, $C'(\Delta)$, and $C'(S)$: Weighting functions depending on M , Δ , and S , respectively.

Classification of 277-component records employed in the analysis is shown in Tables 6 and 7. Magnitudes in the tables are irregularly divided into five groups with consideration of scattering of the data used and also convenience of practical application of the results.

Table 8 shows the results of the quantification analysis for S_A , and lists the weighting functions $C(M)$, $C(\Delta)$, and $C(S)$ in Eq. (3), ranges of the three functions, and the multiple correlation coefficients. Furthermore, Figs. 6 to 9 illustrate the response spectral curves (T - S_A relation) which are obtained from Table 8. In these figures, the maximum values of ground acceleration ($A_{\max}$) which are also estimated from the quantification analysis are tabulated. From the two types of information, the acceleration magnification factor β ($= \bar{S}_A / \bar{A}_{\max}$) can be evaluated. The values of $A_{\max}$ shown here, however, may not be used for the estimation of the absolute maximum horizontal accelerations ($H_{\max}$), but for the estimation of β . For estimating $H_{\max}$, the results described in §3 may be employed.

Degrees of the effects of the three factors (M , Δ , and S) on $\bar{S}_A$ are shown in Fig. 10, by indicating the variation of the ranges of $C(M)$, $C(\Delta)$, and $C(S)$ with T . On the other hand, Fig. 11 shows an example of frequency

distribution for 277 values of $S_A/\bar{S}_A$ (ratio of response spectral accelerations computed directly from the measured records to those estimated by the results of the quantification analysis) for the natural period of $T=0.6$ seconds. Shapes of frequency distribution of $S_A/\bar{S}_A$ for the other natural periods are found to be similar to that of Fig. 11.

From the analysis shown in the above, the following matters can be pointed out.

- (1) Values of $\bar{S}_A$ at the same epicentral distance increase as the earthquake magnitude gets greater.
- (2) Values of $\bar{S}_A$ for the same earthquake magnitude decrease as the epicentral distance gets smaller.
- (3) As for the effects of subsoil conditions on $\bar{S}_A$, the effect on rock ground is considerably different from those on the other three subsoils. Values of $\bar{S}_A$ for rock are smaller than those for the other subsoils, except for natural periods shorter than 0.3 seconds. For the range of comparatively longer periods, the values of $\bar{S}_A$ tend to become larger for softer subsoils.
- (4) The shapes of $T-S_A$ curves do not change much with the variations of M and Δ . It is supposed from Figs. 6 to 9 that for the range of longer natural periods the values of β will increase as M gets greater, and as Δ gets longer.
- (5) Among the effects of the three factors, earthquake magnitude ($M=4.5$ to 7.9), epicentral distance ($\Delta=6 \sim 405$ km), and subsoil condition (rock to soft alluvium), the effects of M are found to be the greatest for the entire range of natural periods. The effects of Δ are close to those of M for the shorter period range, and become small for the longer period range. The effects of subsoil condition are small comparing with those of M and Δ , for the entire period range. This may be reasonable by the fact that the ranges of M and Δ are both considerably large. It is noted, however, that for the range of the periods between 0.5 and 2 seconds the effects of subsoils become rather great.
- (6) Multiple correlation coefficients in the quantification analysis of

$\bar{S}_A$ are between 0.53 and 0.72. The coefficients are generally larger as the natural periods become longer. Multiple correlation coefficient in the analysis of $\bar{A}_{\max}$ is approximately 0.6.

- (7) Frequency distribution of $S_A/\bar{S}_A$ do not vary largely with the change in natural periods. With a view of this frequency distribution, it may become possible to stochastically assess design seismic forces in consideration of importance of structures.

The results described so far are for the case of the damping ratio of $h=0.05$ only. Values of $\bar{S}_A$ for damping ratios other than 0.05 can be approximately estimated by multiplying the values of $\bar{S}_A$ for $h=0.05$ for C_h shown in Fig. 12, which are derived from the previous investigation^{9),10)}.

As an example of use of the results shown in the above, let estimate the average response of a structure with the natural period of $T = 0.35$ seconds and the damping ratio of $h = 0.1$, when subjected to an earthquake with the Richter magnitude of $M=7.5$ to 7.9 and the epicentral distance of $\Delta=60$ to 120 km. From Fig. 9(c) and Fig. 12.

$$\begin{aligned} S_A &= S_A(T=0.35 \text{ sec.}, h=0.05) \times C_h(h=0.1) \\ &= (200 \sim 308) \times 0.78 = (156 \sim 240) \text{ gals} \end{aligned}$$

The value of S_A varies depending on the subsoils at the structural site, being 156 gals for rock, 211 gals for dilluvium, and 240 gals for alluvium, and 234 gals for soft alluvium. Response accelerations of structures with different dynamic properties can be estimated in the similar way, dependent on the earthquake magnitude, the epicentral distance, and the subsoil condition.

It should be noted that in the above analysis structures are assumed as linear systems with viscous dampings. A similar analysis for nonlinear structures may be necessitated in the succeeding step.

5. CONCLUSIONS

The following remarks may be pointed out as the conclusions of the

statistical analyses of strong motion records measured in Japan.

- (1) The absolute maximum horizontal acceleration, the duration of major motions, and the number of zero-crossings can be approximately evaluated as functions of the earthquake magnitude and the epicentral distance, depending on subsoil conditions.
- (2) The mean value of the ratio of vertical to horizontal accelerations is found to be around $1/3$. The ratio may be conservatively taken as $1/2$, for the design purposes.
- (3) Response spectral curves by which structural response accelerations can be averagedly estimated in terms of earthquake magnitude, epicentral distance, and subsoil conditions, are proposed.
- (4) Among the three factors of the earthquake magnitude (M), the epicentral distance (Δ), and the subsoil condition (S), which affect the response acceleration of a structure during an earthquake, the effects of M are the greatest. The effects of Δ are close to those of M for the range of shorter natural periods, and get smaller for the range of longer natural periods. The effects of subsoil conditions are comparatively small, but become comparatively large for the range of natural periods between 0.5 and 2 seconds. It is remarkable that response accelerations for the range of the natural periods between 0.5 and 2 seconds get larger as subsoils become softer.
- (5) The results presented in this paper would be useful in evaluating the characteristics of seismic forces and seismic loads for the earthquake resistant design of structures.

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Table 1 Number of Strong-Motion Accelerographs (SMAC-Type)

(as of March, 1976)

Number of Accelerographs Installed at	At Structures	On Grounds	Total
Buildings	424	48	472
Highway Bridges	89	79	168
Railways	38	57	95
Ports	5	51	56
Telephone Offices	94	0	94
Power Plants	27	11	38
Nuclear Power Plants	21	20	41
Dams	9	10	19
River & Coastal Dykes	15	28	43
Storage Tanks	3	4	7
Others (Tunnels, Subways, Pipelines, etc.)	5	10	15
Total	730	318	1,048

Table 2 Definitions of Classification of Subsoil Conditions

Category	Definitions	Abbreviation
I	(1) Ground of the Tertiary era or older defined as bedrock hereafter in this table (2) Diluvial layer with depth less than 10 meters above bedrock	Rock
II	(1) Diluvial layer with depth greater than 10 meters above bedrock (2) Alluvial layer with depth less than 10 meters above bedrock	Diluvium (Dil.)
III	Alluvial layer with depth less than 25 meters, which has soft layer with depth less than 5 meters	Alluvium (All.)
IV	Softer than the above	Soft Alluvium (Soft All.)

Table 3. Classification of 301 Records Analyzed

Magnitude M	Subsoil Condition	Epicentral Distance Δ (km)					Total	
		$20 < \Delta$	$20 \leq \Delta < 60$	$60 \leq \Delta < 120$	$120 \leq \Delta < 200$	$200 \leq \Delta$		
$5.0 \leq M < 6.0$	Rock	2	3				5	65
	Dil.	3	12	3			18	
	All.	4	16	5	2	2	29	
	Soft All.	2	5	7	1		15	
$6.0 \leq M < 6.5$	Rock		3	6	3	1	13	81
	Dil.		7	8	2	1	18	
	All.		12	13	8	1	34	
	Soft All.		3	6	6	1	16	
$6.5 \leq M < 7.0$	Rock		1	2	1		4	62
	Dil.		1	6	6		13	
	All.	1	5	10	10	1	27	
	Soft All.		1	4	9	4	18	
$7.0 \leq M < 7.5$	Rock			1		3	4	40
	Dil.				2	12	14	
	All.				2	14	16	
	Soft All.			1	1	5	7	
$7.5 \leq M \leq 7.9$	Rock				1	2	3	50
	Dil.				3	8	11	
	All.			2	6	16	24	
	Soft All.				1	11	12	
Total		12	69	74	64	82	301	

Note. 1) Numbers of records for four subsoil conditions are 29 for rock, 74 for diluvium, 130 for alluvium, and 68 for soft alluvium.

Table 4 Regression Equations for H_{max} , T_d and N_z

Characteristic Value	Subsoil Condition	Empirical Equations	Partial Correlation Coefficient		Multiple Correlation Coefficient r
			Magnitude M	Epicentral Distance Δ	
H_{max}	Rock	$H_{max} = 28.5 \times 10^{0.207M_x} \Delta^{-0.598}$	0.33	-0.47	0.48
	Dil.	$H_{max} = 13.2 \times 10^{0.330M_x} \Delta^{-0.806}$	0.50	-0.59	0.59
	All.	$H_{max} = 32.1 \times 10^{0.254M_x} \Delta^{-0.757}$	0.47	-0.61	0.61
	Soft All.	$H_{max} = 6.5 \times 10^{0.423M_x} \Delta^{-0.977}$	0.65	-0.72	0.72
	Total	$H_{max} = 18.4 \times 10^{0.302M_x} \Delta^{-0.800}$	0.51	-0.62	0.62
T_d	Rock	$T_d = 3.89 \times 10^{-4} \times 10^{0.466M_x} \Delta^{0.589}$	0.59	0.40	0.89
	Dil.	$T_d = 1.37 \times 10^{-2} \times 10^{0.262M_x} \Delta^{0.485}$	0.29	0.26	0.71
	All.	$T_d = 2.75 \times 10^{-2} \times 10^{0.291M_x} \Delta^{0.265}$	0.37	0.17	0.61
	Soft All.	$T_d = 2.28 \times 10^{-1} \times 10^{0.199M_x} \Delta^{0.233}$	0.25	0.16	0.52
	Total	$T_d = 2.08 \times 10^{-2} \times 10^{0.274M_x} \Delta^{0.394}$	0.32	0.23	0.65
N_z	Rock	$N_z = 1.43 \times 10^{-2} \times 10^{0.410M_x} \Delta^{0.444}$	0.41	0.22	0.75
	Dil.	$N_z = 4.23 \times 10^{-1} \times 10^{0.0974M_x} \Delta^{0.681}$	0.11	0.35	0.66
	All.	$N_z = 6.28 \times 10^{-1} \times 10^{0.272M_x} \Delta^{0.0313}$	0.39	0.02	0.54
	Soft All.	$N_z = 4.05 \times 10^{0.190M_x} \Delta^{-0.0646}$	0.22	-0.04	0.29
	Total	$N_z = 4.68 \times 10^{-1} \times 10^{0.243M_x} \Delta^{0.195}$	0.30	0.12	0.54

Table 5. Averages and Standard Deviations

of T_1 , v , and Subsoil T_m

		Subsoil Condition			
		Rock	Diluvium	Alluvium	Soft Alluvium
T_1 (sec)	Average	0.36	0.43	0.49	0.75
	S.D.	0.52	0.39	0.30	0.42
v	Average	0.34	0.33	0.34	0.32
	S.D.	0.15	0.18	0.18	0.21
T_m (sec)	Average	0.30	0.34	0.39	0.61
	S.D.	0.36	0.22	0.22	0.35
T_1/T_m		1.20	1.26	1.26	1.23

Note: 1) S.D. = Standard Deviation

Table 6. Classification of 277-Component Records for the Response Spectrum Analysis

Magnitude M	Subsoil Condition	Epicentral Distance Δ (km)					Total	
		$\Delta < 20$	$20 \leq \Delta < 60$	$60 \leq \Delta < 120$	$120 \leq \Delta < 200$	$200 \leq \Delta$		
$4.5 \leq M < 5.4$	Rock	6	4				10	60
	Dil.	4	10				14	
	All.	12	8	8	2		30	
	Soft All.	6					6	
$5.4 \leq M < 6.1$	Rock		4	2			6	48
	Dil.	4	4	4			12	
	All.	2	12	6			20	
	Soft All.	4	2	4			10	
$6.1 \leq M < 6.8$	Rock		4	6			10	102
	Dil.		4	4	2		10	
	All.	4	32	22	8	2	68	
	Soft All.		6	4	2	2	14	
$6.8 \leq M < 7.5$	Rock			4	3	2	9	29
	Dil.			2	4	2	8	
	All.				4	4	8	
	Soft All.					4	4	
$7.5 \leq M \leq 7.9$	Rock				2	2	4	38
	Dil.				6	2	8	
	All.		2	6	4	2	14	
	Soft All.				2	10	12	
Total		42	92	72	39	32	277	

Note: 1) Numbers of components for four subsoil conditions are 39 for rock, 52 for diluvium, 140 for alluvium, and 46 for soft alluvium.

Table 7 Classification of 277-Component Records and
Mean Values in Each Category

Factor	Category		Number of Components	Mean Value		
				Peak Ground Acc. A _G (gals)	Magnitude M	Epicentral Distance Δ (km)
Magnitude (M)	I	4.5 ≤ M ≤ 5.4	60	66.0	5.0	31
	II	5.4 ≤ M ≤ 6.1	48	67.5	5.8	43
	III	6.1 ≤ M ≤ 6.8	102	63.6	6.3	76
	IV	6.8 ≤ M ≤ 7.5	29	58.1	7.1	194
	V	7.5 ≤ M ≤ 7.9	38	112.4	7.7	197
Epicentral Distance (Δ)	I	6 ≤ Δ < 20km	42	110.6	5.3	12
	II	20 ≤ Δ < 60km	92	68.2	5.8	38
	III	60 ≤ Δ < 120km	72	59.1	6.2	83
	IV	120 ≤ Δ < 200km	39	56.2	7.0	159
	V	200 ≤ Δ ≤ 405km	32	71.3	7.3	271
Subsoil Conditions (S)	I	Rock	39	59.9	6.2	87
	II	Diluvium	52	69.4	6.2	86
	III	Alluvium	140	73.2	6.1	74
	IV	Soft Alluvium	46	75.1	6.4	143
Total	-		277	70.9	6.2	90

Table 8 Estimation of Average Response Acceleration S_A ($\log_{10} S_A = C(M) + (C(\delta) + C(S))$) for Systems with Natural Period of T and Damping Ratio of 0.05, by Quantification Analysis of 277 Components of Strong-Motion Records

(in gals)																		
Natural Period	C (M)					C (δ)					C (S)				Multi-Correlation Coefficient, r	Range of C (M)	Range of C (δ)	Range of C (S)
	Category of Earthquake Magnitude					Category of Epicentral Distance (km)					Category of Subsoil Condition							
	I	II	III	IV	V	I	II	III	IV	V	I	II	III	IV				
T ₁	4.52M<5.4	5.42M<6.1	6.12M<6.8	6.82M<7.5	7.52M<7.9	650<20	2050<60	6050<120	12050<200	20050<405	Rock	Diluvium	Alluvium	Soft Alluvium				
0.1	-0.66	-0.56	-0.53	-0.40	0	0.71	0.43	0.31	0.00	0	2.10	2.03	2.08	2.03	0.56	0.66	0.71	0.08
0.15	-0.65	-0.56	-0.53	-0.35	0	0.69	0.48	0.33	0.00	0	2.19	2.11	2.15	2.10	0.53	0.65	0.69	0.10
0.2	-0.73	-0.55	-0.54	-0.30	0	0.74	0.51	0.32	0.02	0	2.23	2.17	2.21	2.11	0.54	0.73	0.74	0.12
0.25	-0.77	-0.60	-0.55	-0.27	0	0.84	0.56	0.37	0.08	0	2.13	2.11	2.16	2.11	0.55	0.77	0.84	0.05
0.3	-0.79	-0.57	-0.55	-0.26	0	0.82	0.55	0.35	0.11	0	2.04	2.11	2.17	2.12	0.56	0.79	0.82	0.13
0.35	-0.79	-0.56	-0.52	-0.23	0	0.76	0.48	0.33	0.10	0	1.97	2.10	2.17	2.15	0.55	0.79	0.76	0.20
0.4	-0.82	-0.57	-0.51	-0.25	0	0.74	0.48	0.28	0.12	0	1.92	2.09	2.16	2.16	0.57	0.82	0.74	0.24
0.5	-0.97	-0.63	-0.51	-0.23	0	0.80	0.46	0.20	0.13	0	1.88	2.05	2.15	2.19	0.63	0.97	0.80	0.31
0.6	-1.05	-0.61	-0.49	-0.21	0	0.77	0.45	0.16	0.12	0	1.79	2.00	2.13	2.20	0.67	1.05	0.77	0.41
0.7	-1.14	-0.65	-0.50	-0.19	0	0.83	0.47	0.19	0.14	0	1.70	1.95	2.07	2.17	0.70	1.14	0.83	0.47
0.8	-1.17	-0.67	-0.53	-0.23	0	0.77	0.44	0.19	0.11	0	1.68	1.96	2.06	2.16	0.68	1.17	0.77	0.48
0.9	-1.17	-0.67	-0.55	-0.24	0	0.71	0.38	0.17	0.08	0	1.67	1.96	2.05	2.13	0.67	1.17	0.71	0.47
1	-1.19	-0.69	-0.55	-0.20	0	0.66	0.33	0.15	0.06	0	1.64	1.95	2.03	2.10	0.67	1.19	0.66	0.46
1.5	-1.30	-0.86	-0.69	-0.27	0	0.64	0.34	0.16	0.00	0	1.52	1.75	1.84	1.93	0.72	1.30	0.64	0.41
2	-1.22	-0.83	-0.67	-0.23	0	0.56	0.30	0.11	-0.03	0	1.39	1.57	1.64	1.66	0.71	1.22	0.60	0.27
2.5	-1.23	-0.87	-0.74	-0.39	0	0.54	0.29	0.13	-0.02	0	1.34	1.51	1.55	1.52	0.70	1.23	0.57	0.21
3	-1.18	-0.86	-0.71	-0.41	0	0.51	0.25	0.13	-0.06	0	1.27	1.42	1.45	1.42	0.69	1.18	0.58	0.18
4	-1.15	-0.84	-0.73	-0.40	0	0.45	0.21	0.10	-0.10	0	1.19	1.31	1.38	1.28	0.68	1.15	0.55	0.19

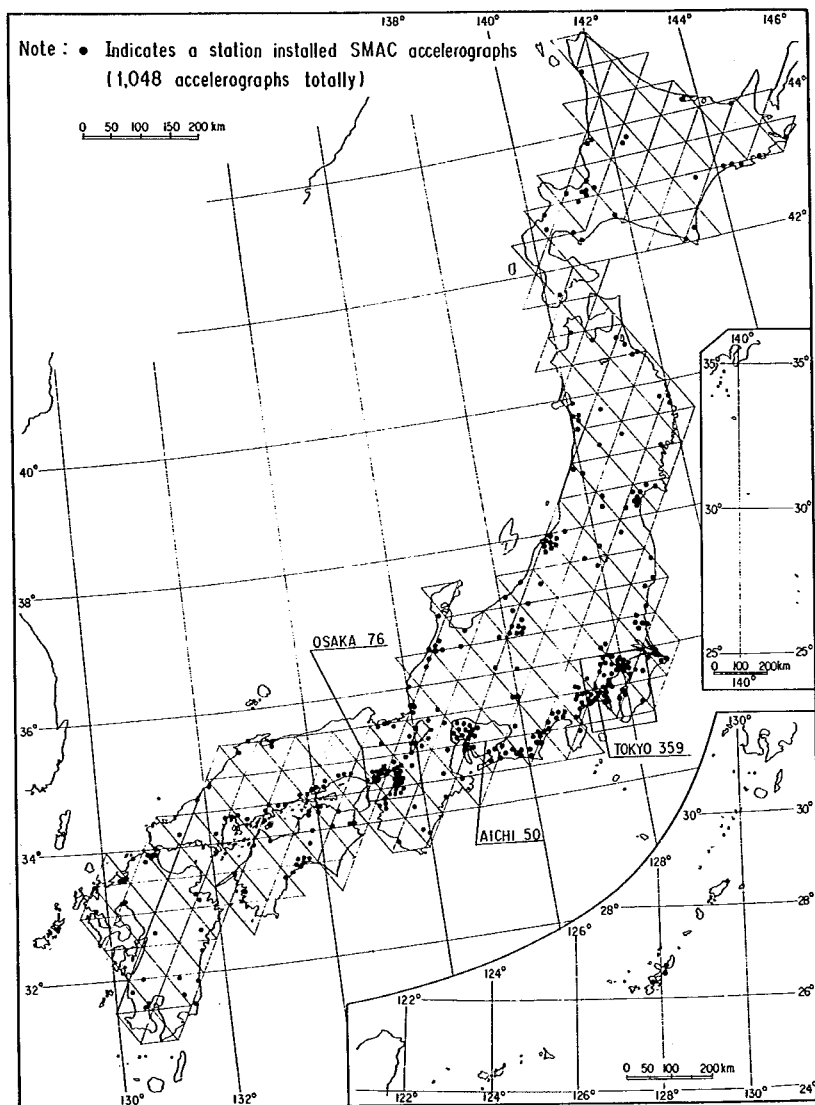


Fig. 1 Strong-Motion Observation Network in Japan as of March, 1976

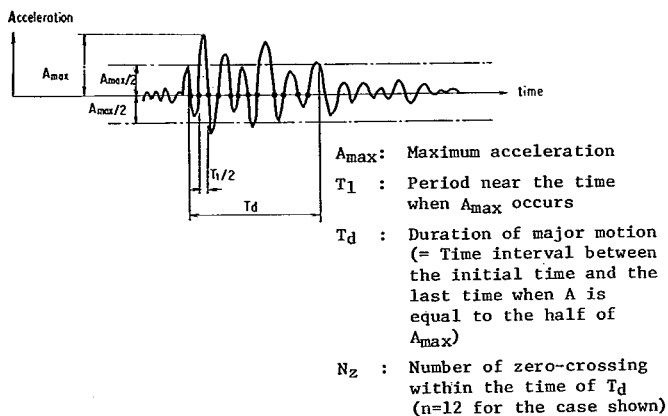


Fig. 2 Definitions of T_1 , T_d and N_z

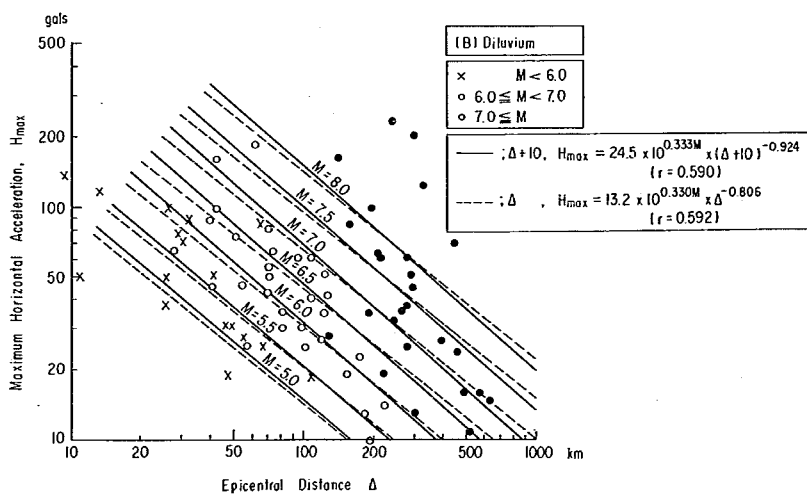
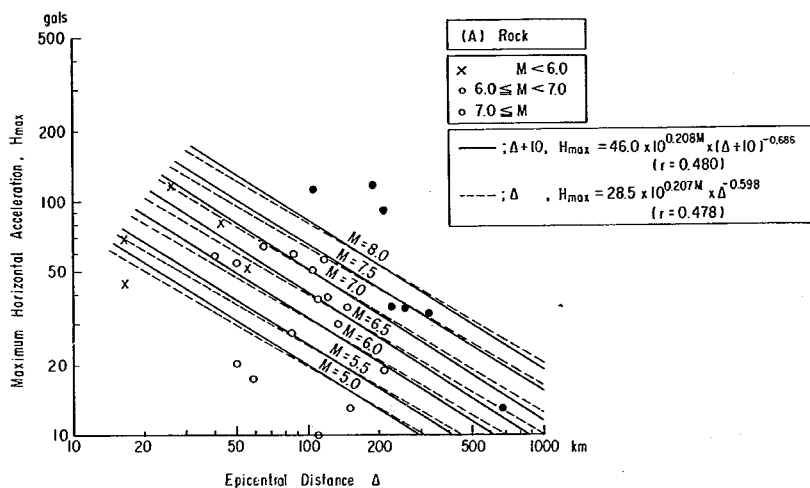


Fig.3(A) ~ (B) Epicentral Distance versus Maximum Horizontal Acceleration for Four Subsoil Conditions

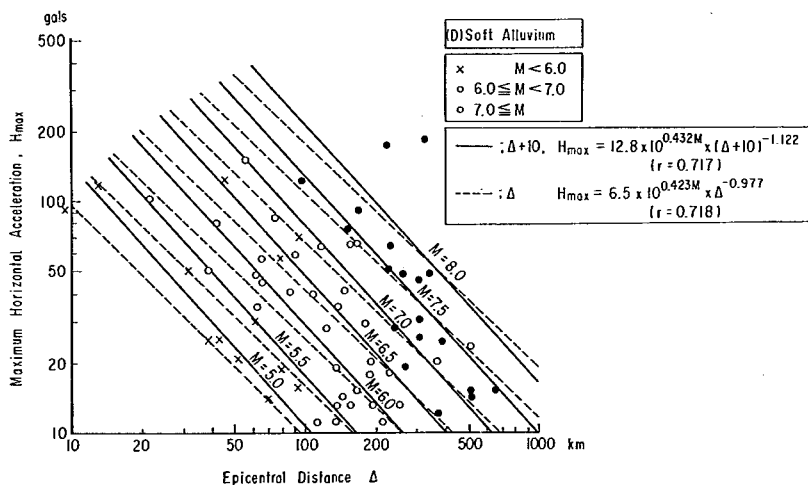
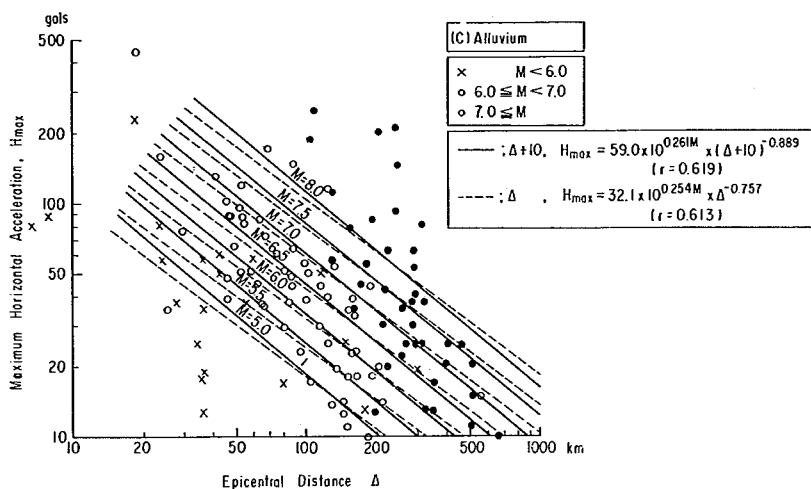


Fig.3(C)~(D) Epicentral Distance versus Maximum Horizontal Acceleration for Four Subsoil Conditions

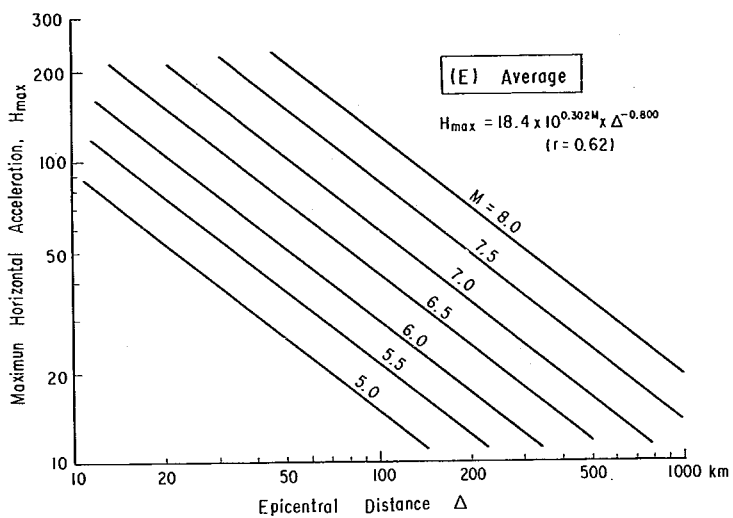
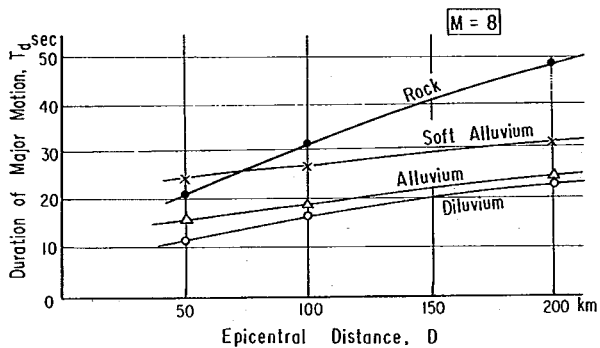
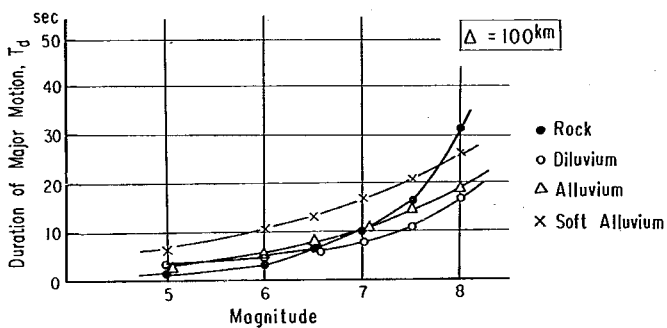


Fig. 3(E) Epicentral Distance versus Maximum Horizontal Acceleration for Average Subsoils



(A) Epicentral Distance versus Duration of Major Motion for Magnitude of 8



(B) Earthquake Magnitude versus Duration of Major Motion at Epicentral Distance of 100 Km

Fig. 4 Duration of Major Motion Defined by Fig. 2

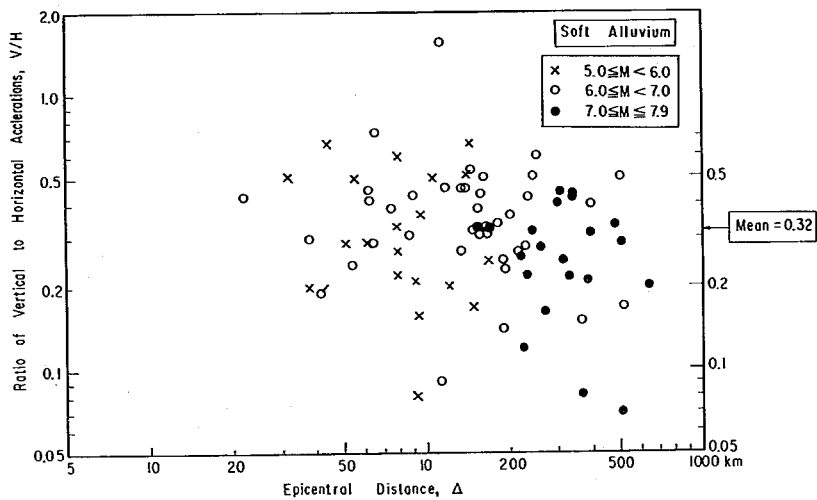
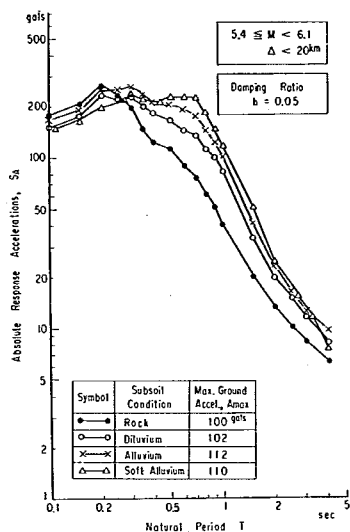
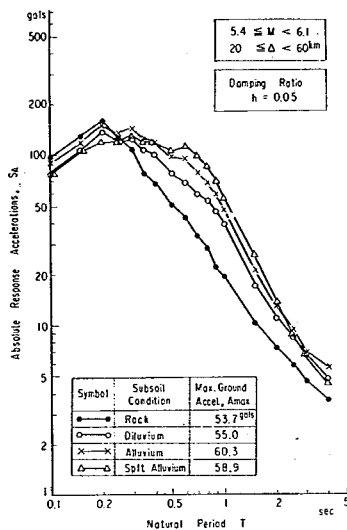


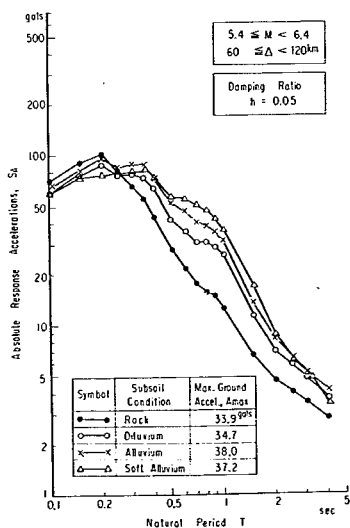
Fig. 5 Example of Ratios of Vertical to Horizontal Accelerations



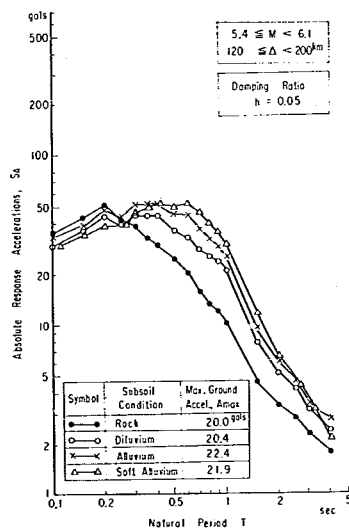
(A)



(B)

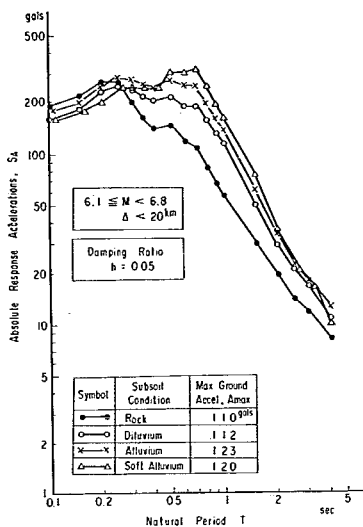


(C)

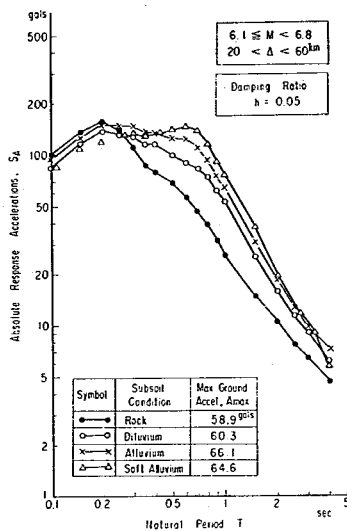


(D)

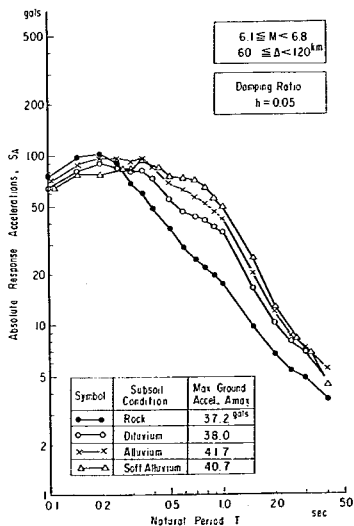
Fig.6 (A)~(D) Response Spectrum Curves for Maximum Response Accelerations in Case of $5.4 \leq M < 6.1$



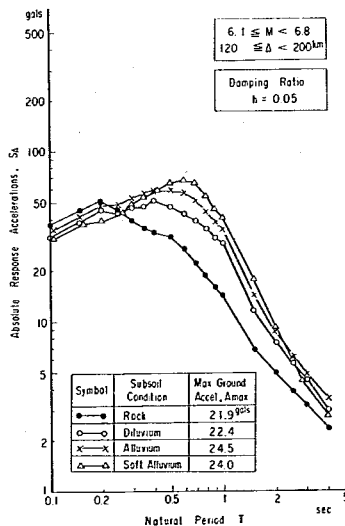
(A)



(B)

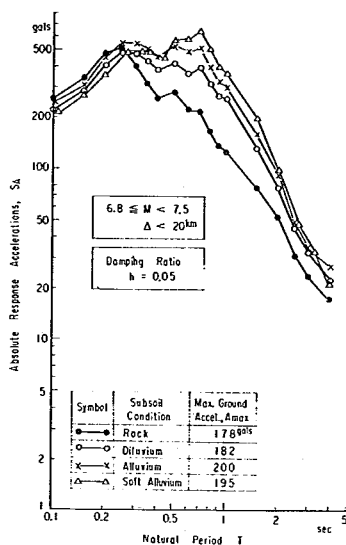


(C)

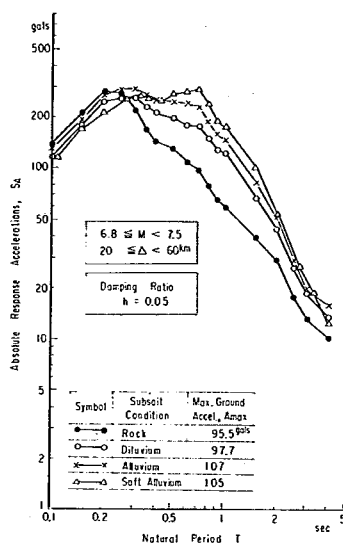


(D)

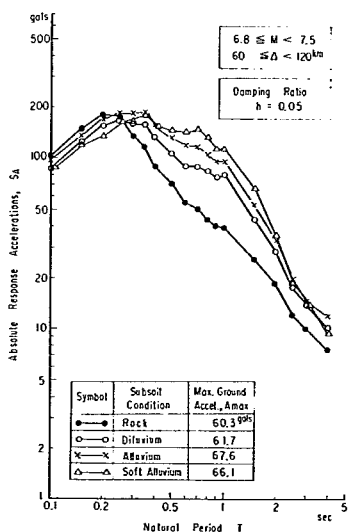
Fig.7(A)~(D) Response Spectrum Curves for Maximum Response Accelerations in Case of $6.1 \leq M < 6.8$



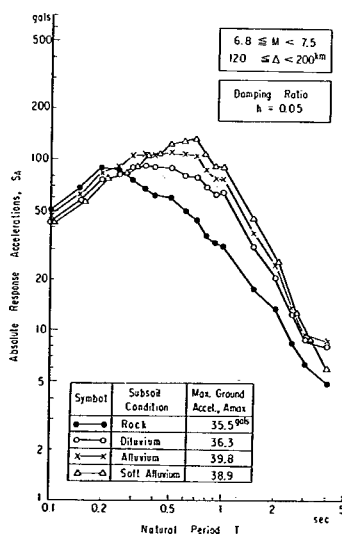
(A)



(B)

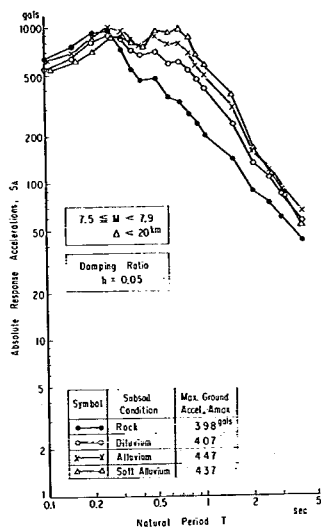


(C)

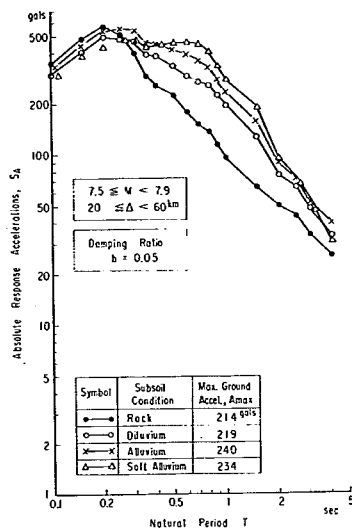


(D)

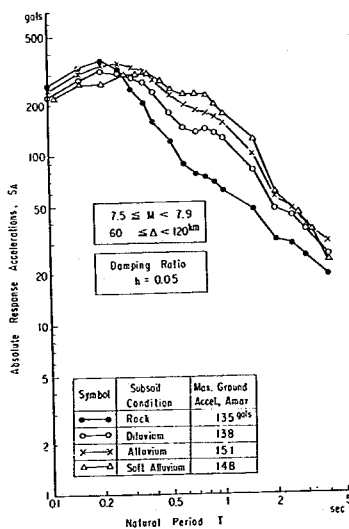
Fig. 8(A)~(D) Response Spectrum Curves for Maximum Response Accelerations in Case of $6.8 \leq M < 7.5$



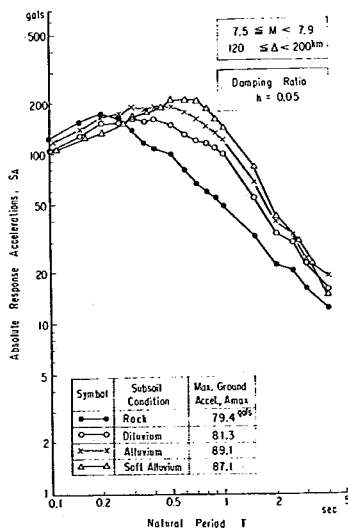
(A)



(B)



(C)



(D)

Fig.9(A)~(D) Response Spectrum Curves for Maximum Response Accelerations in Case of $7.5 \leq M \leq 7.9$

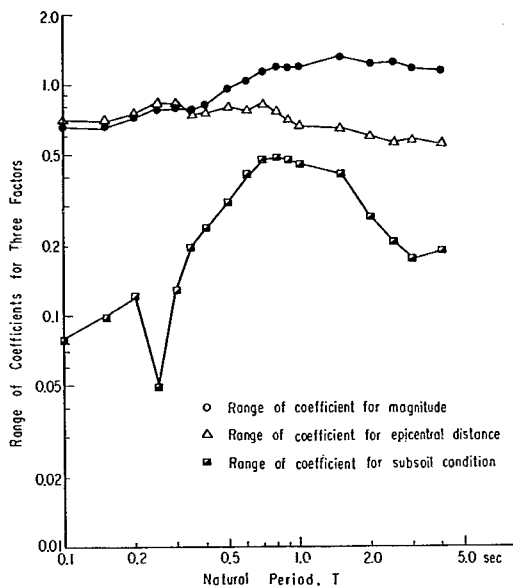


Fig. 10 Natural Period versus Ranges of Coefficients for the Three Factors (Magnitude, Epicentral Distance, and Subsoil Condition)

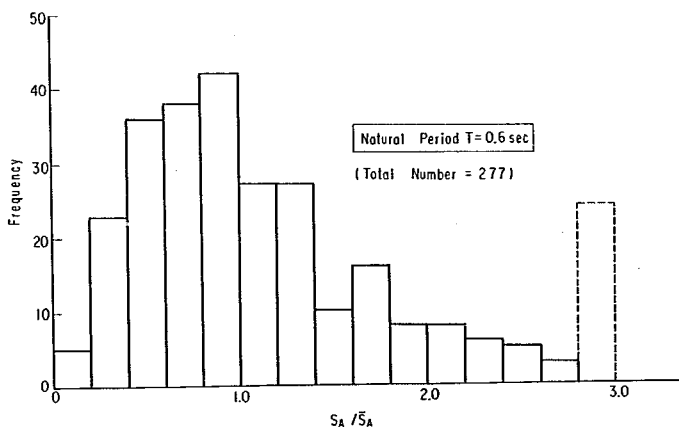


Fig. 11 Distribution of $S_A / \bar{S}_A$

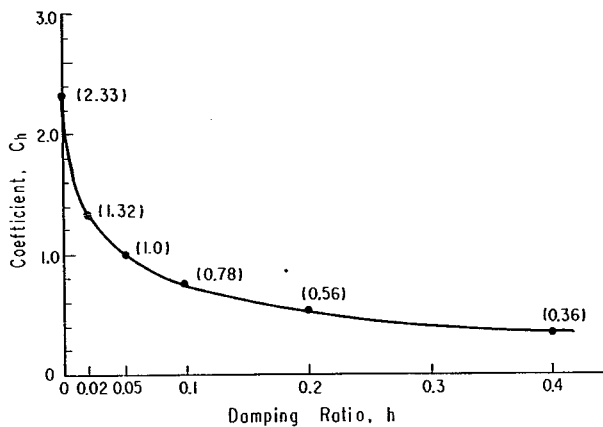


Fig. 12 Effect of Damping Ratio (C_h) on Response Spectrum Values ($C_h = 1.0$ when $h = 0.05$)

STUDIES ON SOIL LIQUEFACTION RELATING TO THE EARTHQUAKE RESISTENT DESIGN OF STRUCTURES

by
* Masamitsu Ohashi, ** Toshio Iwasaki and *** Fumio Tatsuoka

SYNOPSIS

In order to evaluate dynamic behavior of structural foundations and embedded structures during earthquakes, it is essential to estimate the effects of the surrounding soils on those structures. The authors have conducted a literature survey on the effects of liquefied soils on bridge foundations and also laboratory experiments using models of pile foundations including surrounding soils.

Furthermore, a simplified method to evaluate liquefaction potential of sand deposits was investigated on the basis of numbers of blows (so called N-values) by the standard penetration test. Proposed herein are the critical N-values which can be used in determining liquefaction potential. In this method, it is estimated that the sand deposit may or may not be likely to liquefy during future severe earthquakes by comparing N-values measured at the site of interest with the critical values proposed. These critical values were determined on the basis of dynamic triaxial tests on undisturbed sandy samples and N-values which were measured at the points where the undisturbed samples were obtained.

These studies are to clarify the effects of liquefied soils on pile foundations and to establish design methodology of pile foundations considering the effects of soil liquefaction.

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§ 1. INTRODUCTION

In order to appropriately evaluate dynamic behavior of pile foundations or embedded structures during earthquakes, it is essential to estimate the effects of the behaviors of the surrounding soils on those structures. It can be supposed that the surrounding soils do not move severely in normal cases and normally resist movements of the structures, but sometimes the surrounding soils exert large movements of the structures when the soils vibrate severely as a result of a resonant state of the soils and furthermore in special cases the surrounding soils do not support these structures when soil liquefaction takes place.

To provide a proper solution to this problem, it seems necessary to precisely analyze the behavior of the structures considering the effects of soil structure interactions. When the surrounding soils liquefy during earthquakes, the problem becomes more complicated. In this case, the effects of liquefied soils on the structures will be much severer.

The authors have conducted a literature survey on the effects of liquefied soils on bridge foundations and also laboratory experiments using models of structures including surrounding soils. This attempt is to clarify the effects of liquefied soils on pile foundations, and to establish design methodology of pile foundations considering the effects of soil liquefaction.

Another problem to be considered in the asismic design of bridges with pile foundations which are constructed in sand deposits with high liquefaction potential is the evaluation liquefaction potential of sand deposits. In simplified procedures of the evaluation of liquefaction potential generally used in Japan, the N-value obtained from standard penetration tests are used in evaluating the liquefaction potential of the sand deposit.

If the N-value of the sand deposit of interest is lower than a critical value, say $N=10$, it is judged that this sand deposit will liquefy during severe earthquake motions. In this study dynamic triaxial tests on undisturbed sand samples were conducted and at the same time N-value were measured at the points from where undisturbed samples were secured. Then the appropriate critical N-values were determined on the basis of the relationship among dynamic shear strength, N-value and insitu effective overburden pressure. And it was found that even in case dynamic shear strength is

identical N-values may vary according to the variation of soil properties which can be represented by the content of fine soils or the mean diameter of soil particles.

The results of this study can be summarized as follows:

1) The supporting capacities of surrounding soils may become considerably small when liquefaction takes place.

In some cases, dynamic forces of soils acting on piles may become considerably large in the course of liquefaction. When the complete liquefaction takes place, both of the supporting capacities and the dynamic forces can be approximately estimated by assuming that a liquefied sand layer is heavy water.

2) On the course of liquefaction, soil-pile foundation systems may resonate to the input motions, and may vibrate severely.

3) Critical N-values in evaluating liquefaction potential should be determined taking into account the fact that N-values decrease with the increase in the content of fine soils in sand deposits even in case the dynamic shear strength does not decrease so much compared with the decrease in N-values.

§ 2. DAMAGE TO BRIDGE FOUNDATIONS DUE TO LIQUEFACTION

It has been recognized that one of the main causes of the damages to bridges which were constructed in alluvial deposits or reclaimed lands is unstable performances of ground soils. It is also well known that liquefaction of sandy soils induce catastrophic movements of grounds and loss of bearing capacities of grounds. A number of damages to bridge structures with pile foundations were observed during the Niigata Earthquakes of 1964 in Japan (Iwasaki (1973)) and during the Alaska Earthquake of 1964 in United States (Kachadoorian (1968)). Some of similar damages to the above are reported for other earthquakes in Japan (Iwasaki (1973)) and for the San Francisco Earthquake of 1906 (Youd and Hoose (1976)). On the other hand, liquefaction has been observed in alluvial deposits and reclaimed lands during major earthquakes including the above earthquakes. Only in Japan, 44 earthquakes induced liquefaction phenomena during the period from 1872 to 1968. (Kuribayashi and Tatsuoka (1974)).

In viewing damages to engineering structures caused by soil liquafac-

tion, numerous studies on evaluation of liquefaction potential of sandy deposits have been conducted by using dynamic soil testing equipments such as a dynamic triaxial apparatus and a dynamic simple shear apparatus. Furthermore, conducted are some studies using shaking table on the effects of the loss of bearing capacities caused by liquefaction on the behavior of heavy structures (Yoshimi et al. (1975), Ishihara et al. (1975)). However, a little efforts have been made on studies of effects of soil liquefaction on the dynamic behavior of pile foundations (Shikamori, Sato and Hakuno (1973), Yoshida and Uematsu (1975), Iwasaki, Tatsuoka and Sakaba (1976)). Accordingly methodology of aseismic design of pile foundations considering the effects of liquefaction is still in the process of formation.

Intending to establish a rational aseismic design criterion of pile foundations which are constructed in soil deposits with high liquefaction potential, a series of model tests of pile foundations placed in liquefying sand deposits was performed using shaking tables.

§ 3. EXPERIMENT ON DYNAMIC BEHAVIOR OF PILE FOUNDATION MODELS

Test Arrangements In this experiment small models of pile foundations were prefabricated on a shaking table. Five different tests listed in Table 1 were conducted. Typical test arrangements in the cases of Test 1 and Test 2 are illustrated schematically in Fig. 1 and in Fig. 2, respectively. A saturated loose sand layer including a small model of pile foundation was made in a container on a shaking table. To actuate the shaking table a electro-magnetic actuator was used for Test 1. A displacement-controlled hydraulic actuator was used for Tests 2 to 5. Sand layers were prepared by pouring air dry sand into water. Initial relative densities of sand layers ranged from 24 to 40 percentages. The water level coincided with the sand surface at the beginning of shaking. As shown in Figs. 1 and 2, accelerations and strain gauges were installed to measure movements of the shaking table, sand layers and pile foundation models. Earthpressure cells and pore pressure cells were also equipped to measure earth pressures acting on piles and excessive pore water pressures. Sinusoidal table motions were applied in the whole tests (Tests 1 to 5). The actual acceleration form

measured at the table was of triangular form in the case of Test 1 as shown in Fig. 3 where the table acceleration is denoted as A_1 . However, in the other tests (Test 2 to Test 5) where a hydraulic actuator was used, the measured table acceleration was close to the sinusoidal motion as shown in Figs. 4 and 5. Several motions with different frequencies were adopted as input motions. Frequencies were selected on the basis of the natural frequencies of pile foundation models submerged in water. The base acceleration was determined in view of outbreak of gradual increase in excessive pore water pressure in sand layers. Actuators used in the experiments were not enough to keep an accurately constant base acceleration during a test run. This is due to a great change in rigidity of sand layers which are caused by soil liquefaction. Actually, the base acceleration changed somewhat in the course of liquefaction of sand layers as shown in Figs. 3 through 5.

Four different pile foundation models were examined. Their natural frequencies when submerged in water are 3.8, 4.0, 10 and 14 Hz, respectively. Their natural frequencies in air are slightly higher than those in water. Each of these models consists of several solid aluminium bars with the diameter of 2 cm and a top mass. The values of the natural frequencies of models were adjusted by choosing the number of bars and the weight of the top mass. These models were fixed at the bottom as shown in Figs. 1 and 2 to facilitate the boundary condition. The degree of the fixation would be considered to be larger than those in actual cases.

Principles of Similarity Frequencies of input motion and natural frequencies of pile foundation models submerged in water were determined in the following manner. Objectives of model tests in the field of soil mechanics can be classified into two. The first objective is to find fundamental parameters required to be taken into account when analyzing the behavior of the prototype with an analytical method. This analytical method can be, of course, applied to the model test. In this case, the rigorous satisfaction of principles of similarity is not of first importance. The second objective is to quantitatively estimate the behavior of the prototype from the results of a model test. In this case the principles of similarity should be determined

and satisfied. At present, it is of first importance to find the controlling factors in the behaviors of structures in liquefying soils. Therefore, reported herein is the result of tests of the first category. Accordingly, the similitude between the prototype and the model was satisfied in terms of the following, but others, for example the permeability of sand layers could not satisfy the principles of similarity.

(i) Length (L): Scale ratio considered for length was of the order of 1:10 to 1:30. Therefore, model sand layers with the height of 70 cm correspond to actual sand deposits of 7 to 21 m in depth.

(ii) Density of sand layer (MLT^{-2}): Since actual sands were used, the ratio of density of soils was on the order of 1:1.

(iii) Acceleration (LT^{-2}): Since body force induced by gravity is the same between models and prototypes, the ratio for body force induced by horizontal motions should be on the order of 1:1. And the ratio for horizontal acceleration was also on the order of 1:1.

(iv) Time (T): From (i) and (iii) the ratio for time is determined as $1:\sqrt{L} = 1:\sqrt{10}$ to $1:\sqrt{30} = 1:3.2$ to $1:5.5$.

(v) Natural frequency of model pile foundations submerged in water (f_p), frequencies of input motion (f_i), and fundamental natural frequencies of sand layers with the excessive porewater pressure of zero and the shear strain level of $\gamma = 10^{-4}$ or less (f_g) (T^{-1}): These scale ratios are determined from (iv) as $1:\frac{1}{T} = 1:\frac{1}{3.2}$ to $1:\frac{1}{5.5} = 1:0.32$ to $1:0.18$.

Estimated frequencies for f_g model sand layers were around 18 and 24 Hz as shown in Table 1. These values correspond to the natural frequencies of the prototype sand layers, 3.2 to 7.7 Hz. On the other hand, the fundamental natural frequency of soil deposit f_1 (H_z) can be approximately estimated as

$$f_1 = \frac{V_s}{4H} \quad (Hz) \quad \dots \quad (1)$$

in which V_s denotes the average shear wave velocity (m/sec) and H means the depth (m) of soil deposit of interest. As the ordinary value of V_s of shallow alluvial sand deposits ranges from 150 to 250 m/sec, the fundamental frequency of sand deposit with the depth of 7 to 21 m ranges from $150/(4 \times 21) = 1.8 H_z$ to $250/(4 \times 7) = 8.9 H_z$. These values correspond well to the values derived from scale ratios and the fundamental frequencies of model sand

layers (3.2 to 7.7 Hz).

Since predominant frequencies of strong earthquake motion observed on the actual sand deposits are estimated less than the above values, the frequency of input motion in model tests f_i should be less than those of natural frequencies model sand layers f_g (18 and 24 Hz). Therefore, as the values of f_i in model tests, 12 Hz for Iruma sand and 10 Hz and 20 Hz were selected.

Furthermore, it can be estimated that the fundamental natural frequencies of ordinary bridges with pile foundations, f_p are less than the predominant frequency of strong earthquake motion observed on shallow sand deposits when the value of f_p are determined on the assumption that liquefied sand layer with the depth of 7 to 21^m are removed. Therefore, three of five model tests conducted were cases where the natural frequencies of model pile foundations submerged in water, f_p , are less than the frequency of input motion, f_i , corresponding to the actual ordinary cases. In addition to these cases where $f_p < f_i$, other two cases were examined as $f_p = f_i$ and $f_p > f_i$ in order to evaluate the importance of the relationship between these two values f_p and f_i in assessing the dynamic behavior of pile foundations in liquefying sand layers.

Test Records For Test 1, accelerations recorded at the table, at three positions in the sand layer and at three positions of the model pile foundation are shown in Fig. 3. Excessive porewater pressure in the sand layer and earthpressure acting on the pile foundation are also illustrated in the figure during the elapsed time of 8 to 14 seconds counting from the beginning of shaking. Similar records in the cases of Test 2 and of Test 4 are shown in Figs. 4 and 5, respectively. One of the most peculiar performance observed in these tests is non-stationary behaviors of the sand layer and the pile foundation and non-stationary dynamic interaction between them, while the table movement is almost stationary. Furthermore, it can be noted that the responses of different pile foundation models are not similar to each other. Especially, the variation of the response of the pile foundation in Test 4 is quite different from those in Test 1 and Test 2.

In order to clarify the dynamic behavior of the sand layer and the

pile foundation, amplitudes of dynamic quantities recorded were illustrated in Fig. 6 for Test 1. Since in the case of Test 1 excessive porewater pressures reached their maximum stationary values at the elapsed time between 11 and 12 seconds and the acceleration in the entire sand layer decreased to almost zero at the same time, it could be estimated that liquefaction in the entire sand layer initiated at the elapsed time between 11 and 12 seconds in the case of Test 1. It should be noted in the case of Test 1 that the maximum response of the pile foundation occurred at the elapsed time of about 10 seconds when excessive porepressure did not reach the maximum value yet and the sand layer vibrated in ordinary fashion. And at $t = 10$ seconds, dynamic earthpressure acting on the pile foundation and strains in the pile had also their maximum values. In addition to this, it is also noted in the case of Test 1 that the response of the pile foundation decreased to the swaller value after the entire sand layer had liquefied. Corresponding to this, earth pressure acting on the piles and strains in the pile also decreased to the small values. Following this, recovery from liquefaction started at the elapsed time of about 30 seconds, and after this moment the response of pile foundation increased again. Although this recovery phenomenon is also very interesting, this will not be discussed herein because such a phenomenon is unlikely to take place in the prototype during severe ground shaking in an actual earthquake. In this test, the ratio of time in the model and that in the prototype is considered on the order of $1 : 3.2 \sim 1 : 5.5$. Therefore, the elapsed time of 30 seconds in the model corresponds to 96 to 165 seconds in the prototype. This duration time seems longer than the duration of major ground motion during a strong earthquake. Furthermore, it is recognized that recovery from liquefaction began after the cease of earthquake motion in the actual earthquakes experienced. From these reasons, phenomena observed only before the recovery from liquafaction will be discussed in this report.

Variations of Responses of Pile Foundation Models in the Course of Liquefaction In Fig. 7 shown is the variation of response ratio β_p of the pile foundation with respect to frequencies of input motion in the case of Test 1. The response ratio β_p is defined as

$$\beta_p = \frac{|\alpha_{A6}|}{|\alpha_{A1}|} \dots\dots\dots (2)$$

where $|\alpha_{A1}|$ represents the amplitude of the acceleration at the base and $|\alpha_{A6}|$ represents the amplitude of the acceleration at the bottom of the top mass of the pile foundation. The solid curve in Fig. 7 represents the response curve of the pile foundation model measured when submerged in water and black circles denote the response ratio of the pile foundation model measured in liquefying sand layer at several moments of elapsed time. Numbers near black circles represent the elapsed time in seconds from the beginning of shaking. It is seen from this figure that the response of the pile foundation model β_p once increased before $t = 10$ seconds and then decreased abruptly to the smaller value of 0.2 at $t = 12$ seconds. Also illustrated in Fig. 7 are the response ratio of sand layer $\beta_g = |\alpha| / |\alpha_{A1}|$, excessive pore water pressure ratio u/u_{max} and the amplitude of dynamic earth pressure acting on the pile foundation $|p_d|$. These values are illustrated in relation with β_p . It is clearly seen from this figure that at the moment of the maximum response of the pile foundation model, sand layer had not liquefied yet and the dynamic earth pressure had the maximum value and that after the occurrence of liquefaction in the entire sand layer the response of pile foundation and the dynamic earth pressure also decreased to the small values.

Similarly to Fig. 7, Figs. 8, 9 and 10 represent the variation of responses of pile foundation models in the cases of Test 2, Test 4 and Test 5, respectively. In these figures, hollow squares denote responses of pile foundation models before liquefaction does not initiate in any part of the sand layer, hollow triangles show responses at the moment when liquefaction has took place in the upper part of the sand layer and hollow circles represent responses when liquefaction has took place in the entire sand layer. It can be noted from these figures that while the tendency of the variation of the response of the pile foundation model in the case of Test 2 is quite similar to that in the case of Test 1, those in the case of Test 4 and in the case of Test 5 are considerably different from that in the case of Test 1 or that in the case of Test 2. The cause which may induce these differences

among these tests will be discussed later.

Fig. 11 illustrates instantaneous positions of the sand layer and the pile foundation model at several instants of time ($t = 8.0$ to 12 seconds) in the case of Test 1. In this figure, each stage (a) through (d) in each column represents the variation of positions of the sand layer and the pile foundation model with a short interval during the half of the vibration period ($= 1/24$ second). In each figure, hollow circles represent the value of $-\alpha/|\alpha_{A1}|$ in which α denotes the instantaneous value of acceleration recorded by the accelerometers A2 through A4 equipped in the sand layer, and $|\alpha_{A1}|$ represents the amplitude of the base acceleration. In an ideal sinusoidal movement, $-\alpha/|\alpha_{A1}|$ can be an index representing the displacement modes. And black circles represent those for the pile foundation model. Arrows represent the instantaneous directions of the movements at each point and the short vertical straight lines represent the maximum values of $-\alpha/|\alpha_{A1}|$ at each point during the half of the vibration period. Hollow triangles denote the instantaneous values of earthpressures acting on the pile. The positive sign corresponds to the increase of earthpressures sensed by the earthpressure cell which faced to the right direction in these figures. At the bottom of Fig. 11 shown are the excessive porepressure ratio u/u_{max} . It is seen from this figure that at the elapsed time of 8 seconds when excessive porewater pressures were very small, movements of the sand layer and those of the pile foundation model are close to each other during the half of the vibration period. Then, as time goes on and excessive porepressures increase, the difference between the movement of the sand layer and that of the pile foundation model increases gradually. This would mean that the capability of the sand layer to support the pile foundation model decreases with the decrease in effective stress in sand layer which corresponds to the increase in excessive porewater pressure. As seen in Figs. 3 through 11, liquefied parts in the sand layer expand from the surface toward the bottom. Therefore, the free length of the pile foundation model which are not supported by the surrounding sand layer increased with the increase in excessive porepressure in the sand layer. This means that the natural frequency of the pile foundation in the sand layer, which is defined as f_p^* , decreased with the increase in the

free length of the pile foundation.

Then, it would be beneficial to examine the response of the pile foundation model from the viewpoint of structural dynamics. First, suppose a single - degree - of - freedom vibrating system which corresponds to the pile foundation model in the sand layer. The maximum value of the response $\beta_p = 3.6$ of the pile foundation model which took place at the elapsed time of 10 seconds could be regarded as that of the system which resonates to the sinusoidal input motion with the frequency of $f_i = 12$ Hz. Then, the apparent damping ratio (h) of the system could be derived as

$$h \doteq \frac{1}{2 \times 3.6} \doteq 0.14 \dots\dots\dots (3)$$

In Fig. 12 shown are the response curve of the equivalent single-degree - of - freedom vibrating system with the damping of 0.14 and the phase relation between the input force and the motion of the system. Using this response curve, the apparent natural frequency of the pile foundation model embedded in the sand layer f_p^* in the case of Test 1 were obtained as follows. It could be supposed that the natural frequency of the pile foundation embedded in non-liquefied sand layer at the strain of around 10^{-4} is similar to the natural frequency of the sand layer, that is $f_g = 18$ Hz in the case of Test 1. Therefore, just after the beginning of shaking, the natural frequency of the pile foundation embedded in the sand layer f_p^* would be around 18 Hz. This means that $f_i / f_p^* = 12/18 < 1.0$. Therefore, the approximate value of f_i / f_p^* at the elapsed time of $t = 8.0$ seconds can be obtained from the intersection point between $\beta_p = 2.1$, the value of β_p at $t = 8.0$ seconds, and the response curve for $f_i / f_p^* < 1.0$. By the same procedure, the values of f_i / f_p^* were obtained for the following elapsed time $t = 10.0, 10.8, 11.3, 12.2$ seconds as shown in Fig. 12. It is seen from this figure that the ratio of the frequency of input motion to the natural frequency of the pile foundation model embedded in the sand layer f_i / f_p^* increased with time and eventually approached to the value in the case of the pile foundation model submerged in water $f_i / f_p = 12/3.8 = 3.2$. Since the value of f_i was kept constant as 12 Hz, the increase of f_i / f_p^* means the decrease of f_p^* . Also shown in Fig. 12 is the phase relation ϕ between the motion at the bottom of the top mass and the motion at the

base. In this figure, the values of ϕ were obtained from Fig. 11 and the values of f_i / f_p^* were obtained from the procedure described above. The general coincidence of the measured phase relation with that of the single-degree - of - freedom vibrating system with $h = 0.14$ means that the variation of the response of the pile foundation β_p in liquefying sand layer was caused by the variation of the apparent natural frequency of the pile foundation model in the sand layer f_p^* .

As a next step, the effects of the response of the sand layer on the response of the pile foundation model should be taken into account, because the response of sand layer varied in the course of liquefaction as shown in Figs. 3 through 11. In the case of the pile foundation with a very small rigidity, the response of this pile foundation would be completely affected by the response of the sand layer before effective stress in the sand reduces to zero. On the other hand, this effect would be rather small in the case of the pile foundation with a relatively large rigidity. This would be the cases of these tests (Tests 1 to 5). For example in the case of Test 1, the maximum response ratio of the pile foundation β_p is about 3.6 at $t = 10$ seconds and at this moment the response ratio of the sand layer near the surface was about 1.9 which is less than $\beta_p = 3.6$. This means that the large response of the pile foundation model at $t = 10$ seconds was caused mainly by the resonance of the pile foundation model embedded in the sand as illustrated in Fig. 12.

A Summary of Model Tests of Pile Foundation Tests

A summary of the whole test results is shown in Fig. 13 where the relationship between the response ratios (β_p) of the pile foundation models and the ratios (f_i / f_p) of frequencies of input motions to the natural frequencies of the pile foundation models submerged in water. This figure indicates clearly that a comparative relation among the frequencies of input motions f_i , the natural frequencies of the pile foundation models submerged in water f_p and the initial natural frequencies of the sand layer f_g is fundamental in determining the dynamic behaviors of the pile foundation models in liquefying sand layer. As all of the tests shown in Fig. 13 are the cases where f_g is larger than f_i or f_p , these tests can be divided

into the following three groups:

(a) $f_g > f_i > f_p$ (Tests 1 to 3) : In these tests, the response ratios of the pile foundation models β_p are small at the beginning of shaking as indicated by hollow squares in Fig. 13 and as the sand layer is going to liquefy, β_p begins to increase and then β_p reaches its maximum value as indicated by black triangles. At this moment, the sand layer have not liquefied completely. And following this moment, β_p begins to decrease and eventually β_p becomes the small value as indicated by hollow circles in Fig. 13. These small values of β_p correspond well to those of the response ratio of the pile foundation models submerged in water. The phenomenon described above could take place in the prototype as the worst case. Therefore, the maximum value of β_p in the course of liquefaction in sand deposits should be estimated accurately in the case of the prototype where this phenomenon be taken into account in the aseismic design of pile foundations.

(b) $f_g > f_i = f_p$ (Test 4) : In this test, β_p simply increases from the beginning of shaking to the state of the liquefaction in the entire sand layer. The eventual response ratio in the completely liquefied sand layer is as large as the value at the resonance of the pile foundation model submerged in water. If there could be such a case where $f_i = f_p$ in the prototype, severe damages could take place, because the response gets quite large.

(c) $f_g > f_p > f_i$ (Test 5) : In this test, the tendency of the variation of β_p is similar to that of Test 4. However, differently from Test 4, the eventual value of β_p in the completely liquefied sand layer is rather small, because the response ratio of the pile foundation model submerged in water is small due to the fact that the value of f_i / f_p is much less than 1.0. In this case, the damage to the pile foundation would be minor. However, this case would be unlikely to exist in the prototype, because the value of f_p in this case would be unnaturally large in the prototype.

§ 4. SIMPLIFIED PROCEDURES IN EVALUATING LIQUEFACTION POTENTIAL

Present Status

In the specifications for Earthquake Resistant Design of Highway

Bridges (Japan Road Association - 1971), a special attention is given to the design of highway bridges constructed in sandy layers vulnerable to liquefaction during earthquakes and in very soft cohesive layers. The bearing capacities of these layers are neglected in the design, in order to assure high earthquake resistance of structures built in these layers. An article of "Sandy Soil Layers Vulnerable to Liquefaction" are specified as follows:

—— Saturated sandy soil layers which are within 10 meters below the actual ground surface, have a standard penetration test N-value less than 10, have a coefficient of uniformity less than 6, and also have a D_{20} -value of the grain size accumulation were between 0.04 mm and 0.5 mm, shall have a high potential for liquefaction during earthquakes. Bearing capacities of these layers shall be neglected in design.

Saturated sandy soil layers which have a D_{20} -value between 0.004 and 0.04 mm or between 0.5 and 1.2 mm may liquefy during earthquakes, and shall be given an attention. Estimation in the case shall be made in accordance with the available information on liquefaction problems.

When a special investigation is performed, the above article may not be required to apply. ———

On the other hand, it is well known that standard penetration N-values may vary considerably due to the variation of the grain size of sand deposits. For example, it is a general tendency that even in so-called alluvial sandy deposits, N-values are considerably large for gravelly sand deposits and are quite small for silty sand deposits. Therefore it can be easily supposed that N-values do not hold a constant value even for the cases where the dynamic shear strength is coincident. The difference of N-values in the two cases would be caused by the other factors on soil properties, such as the grain size rather than the dynamic shear strength. Accordingly, if an identical critical N-value, say 10, is applied to all sorts of sand deposits in evaluating liquefaction potential, this critical N-value may be too small for gravelly sand deposits and may be too large for silty sand deposits. For these reasons, it is specified in the above-mentioned specifications that the uniformity coefficient U_c and the D_{20} - value should be taken into account in the evaluation of liquefaction potential in addition to the N-value. However, it has become clear recently that there is no evidence that the U_c -values of

sands have a definite relation to liquefaction potential. Furthermore, it seems that the D_{20} -values specified in the above-mentioned specifications are not always proper as an index representing grain size for the evaluation of liquefaction potential.

For these reasons, indicated in this study was the relationship among standard penetration N-values, grain size distribution characteristics and dynamic shear strengths which were obtained from dynamic triaxial tests on undisturbed sandy samples. It was found from the relationship that the critical N-value in the evaluation of liquefaction potential could be smaller for sand deposits with small grain sizes.

Dynamic Triaxial Tests on Undisturbed Sandy Samples

Sampling of undisturbed sandy samples and standard penetration tests were performed at two sites, i) Akebono - Tatsumi Site and ii) Haneda Site. Both sites are along the bay of Tokyo. A typical boring log at Akebono-Tatsumi Site is shown in Fig. 32. Undisturbed samples were secured by a reformed Bishop sampler in which the inner diameter of the sampler is 5.4 cm and the length of the sampler is 65.0 cm. Sandy samples with low contents of fine soils were frozen at site by dry ice to avoid disturbances which may be caused during the transportation from the site to the laboratory for testing. Silty samples were not frozen to avoid the disturbance due to volume expansion caused by freezing. Figs. 15 through 18 show typical grain size accumulation curves. Almost all of sand samples secured are fine sands with D_{50} less than 0.3 mm and with high contents of fine soils less than 0.074 mm.

Dynamic triaxial tests were conducted on samples with the diameter of 5.0 cm and the height of 10.0 cm. Applied isotropical effective confining pressures were equal to insitu effective overburden pressures and enough back pressures were applied to make the Skempton's B-value larger than 0.96 (Fig. 19). Figs. 20 and 21 show typical test results where shown are the relationship between the applied stress ratio and the number of repetitions in both cases of undisturbed samples and of disturbed samples. Disturbed samples were made by pouring completely disturbed de-aired saturated soils which were made from undisturbed samples into a mold.

It was seen from this figure that the strength of disturbed samples is rather smaller than those of undisturbed samples for the same density. In this study, the dynamic shear strength in dynamic triaxial tests was defined as

$$R_{\ell} = \left(\frac{\sigma_{dp}}{2\sigma'_c} \right)_{\ell} = \left[\begin{array}{l} \text{the stress ratio } \left(\frac{\sigma_{dp}}{2\sigma'_c} \right) \text{ at the number} \\ \text{of repetitions } N = 20 \text{ where the simple} \\ \text{amplitude of axial strain } \Delta \varepsilon_a \text{ (SA)} \\ \text{becomes } 3\%. \end{array} \right] \quad (4)$$

The value of R_{ℓ} were read off from the relationship between $\sigma_{dp}/(2\sigma'_c)$ and N as shown in Figs. 20 and 21.

Dynamic Shear Strength (τ / σ_v)_ℓ comparable with Dynamic Shear Stress Ratio (τ / σ_v)_e

Dynamic shear stress ratios $(\tau / \sigma_v)_e$ which are induced in grounds by earthquake motions were defined as the maximum shear stress ratio in the computed time history of shear stress ratio τ / σ_v' at the depth of interest. Earthquake response analysis of grounds were performed by the wave propagation theory with the equivalent linearization method with respect to deformation properties of soils. In this study, underground acceleration records were used as input base motions. The maximum acceleration in computations was taken as 150 gal at the base assuming at the depth of 127m.

In evaluating liquefaction potential, the dynamic shear strength should be compared with the dynamic shear stress ratio due to earthquake motions. (See Fig. 14) In this procedure, to obtain the dynamic shear strength $(\tau / \sigma_v')_{\ell}$ which is compared with the computed dynamic stress $(\tau / \sigma_v')_e$, some corrections should be made on the dynamic shear stress R_{ℓ} which were obtained from dynamic triaxial tests. These corrections are expressed as

$$(\tau / \sigma_v')_{\ell} = C_1 \cdot C_2 \cdot C_3 \cdot C_4 \cdot C_5 \cdot R_{\ell} \cdots \cdots (5)$$

(i) C_1 ; This is the correction factor for confining pressure. In this study, the in situ earthpressure at rest K_0 was estimated to be around 0.5. In this case, the effective mean principal stress in the in situ stress condition becomes $(1 + 2 K_0) \cdot \sigma_v' / 3$ where σ_v' is the effective overburden pressure. On the other hand, the effective mean principal stress in dynamic

triaxial tests is σ_v' as shown in Fig. 19. As it was shown by Ishihara and Li (1972) that dynamic shear strength of samples under stress conditions with different K_0 -values should be evaluated for the same effective mean principal stress, the correction factor C_1 becomes as

$$C_1 = \frac{1 + 2K_0}{3} = \frac{2}{3} \dots\dots\dots (6)$$

(ii) C_2 ; This is the correction factor for loading wave form. While actual earthquake motions are random with respect to time, loading wave forms used in dynamic triaxial tests were sinusoidal with constant amplitudes. According to Ishihara and Yasuda (1972), the value of C_2 ranges from 1/0.55 to 1/0.7 in which C_2 has a larger value for a shock-type earthquake motion and has a smaller value for a vibrational-type one. In this study, a moderate value equal to 1.5 was adopted.

(iii) C_3 ; This is the correction factor for the effect of disturbance which are inevitable in the processes of sampling, freezing, transportation and several procedures in dynamic triaxial tests. It can be supposed that the dynamic shear strength of soils may decrease due to these disturbances. Fig. 22 shows the relationship between the values of R_ℓ of undisturbed samples defined by eq. 4 and those of completely disturbed samples. It is clearly seen from this figure that the difference between two values is too large to be neglected. Therefore, it seems that the value of C_3 is larger than 1.0. However, at present, the precise value of C_3 can not be evaluated.

(iv) C_4 ; This is the correction factor for the effect of the densification which are also inevitable during the above - mentioned operations. In fact, some densifications were observed in samples. As denser sand samples have large dynamic shear strengths, C_4 can be, in general, less than 1.0. Considering that densifications of samples are caused by disturbances during operations, it was estimated in this study that $C_3 \cdot C_4 = 1.0$.

Eventually, the product of all of the correction factors became about 1.0. Therefore, in this study the dynamic shear strength were obtained as

$$(\tau / \sigma_v')_\ell = R_\ell \dots\dots\dots (7)$$

To estimate the value of $(\tau / \sigma_v')_\ell$ at the point where only N-values or both of N-values and other soil properties such as mean grain size and contents of fine soils were obtained but no undisturbed samples were

available, the relationship among $(\tau/\sigma_v')_L$ -values, N-values and soil property parameters should be established. In this study, it was supposed that $(\tau/\sigma_v')_L$ -values can be represented as

$$(\tau/\sigma_v') = R_\ell = F(f(N), S, D_{50}) \dots\dots\dots (8)$$

in which N means the Standard Penetration Test N-value, S denotes the content of fine soils ($< 74\mu$) in a percentage and D_{50} denotes the mean grain size in mm. As a great deal of dynamic triaxial tests on clean sands have been conducted at various laboratories, it is necessary to establish the method of evaluation of clean sand deposits as the first step. For this purpose, the in situ relative density should be evaluated. As an indirect method, Meyerhof (1957) proposed the following equation:

$$D_r = 21 \sqrt{\frac{N}{\sigma_v' + 0.7}} \dots\dots\dots (9)$$

where D_r is relative density in percentages, N is the standard penetration N-value and σ_v' is the effective overburden stress in kg/cm^2 .

This equation was proposed on the basis of the test results on clean sands by Gibbs and Holtz (1957). Figs. 23 and 24 represent the comparison between the relative density obtained from eq. (9) and that obtained directly as $D_r = \frac{e_{\max} - e}{e_{\max} - e_{\min}}$ where the values of void ratio e were obtained from undisturbed sand samples by a reformed Bishop sampler. These samplings were performed at Kawagishi-cho, Niigata city by JSSMFE* (1976). The values of $e_{\max}$ and $e_{\min}$ were obtained on completely disturbed air-dry samples. These figures indicate that the value of the relative density D_r obtained by eq. (9) is almost identical with that obtained directly as $D_r = (e_{\max} - e) / (e_{\max} - e_{\min})$. Fig. 25 shows the relationship between the dynamic shear strength from dynamic triaxial tests on clean sands with no fine soils less than 0.074 mm and the relative densities $D_r = (e_{\max} - e) / (e_{\max} - e_{\min})$. These tests were conducted at various laboratories as listed in Table 2. The average relationship among data plots in Fig. 25. can be represented by

$$R_\ell = 0.0042 D_r \dots\dots\dots (10)$$

where D_r denotes $(e_{\max} - e) / (e_{\max} - e_{\min})$ in percentages. If there is not a large difference between the strength of a undisturbed clean sand and that of a completely disturbed clean sand, the dynamic shear strength

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$(\tau/\sigma_v')_\ell$ can be estimated by the following procedure.

$$(N, \sigma_v') \rightarrow D_r = 21 \sqrt{\frac{N}{\sigma_v' + 0.7}} \quad (\text{eq. (9)}) \rightarrow R_\ell = 0.0042 D_r \quad (\text{eq. (10)})$$

$$\rightarrow (\tau/\sigma_v')_\ell = R_\ell \quad (\text{eq. (7)}) \dots\dots\dots (11)$$

Fig. 26 shows the relationship between $(\tau/\sigma_v')_\ell$ obtained by dynamic triaxial tests on undisturbed samples which were conducted in this study and the value of $D_r = 21 \sqrt{N/(\sigma_v' + 0.7)}$. It can be clearly seen that the $(\tau/\sigma_v')_\ell \sim D_r$ relationship of silty fine sands which were tested in this study is quite different from the average $(\tau/\sigma_v')_\ell \sim D_r$ relationship of clean sands. Therefore, it seems that the direct application of eq. (11) to this case gives considerably under-estimated values of $(\tau/\sigma_v')_\ell$.

The Estimation of $(\tau/\sigma_v')_\ell$ -Values at Akebono-Tatsumi Site

The values of $(\tau/\sigma_v')_\ell$ at the site of Akebono-Tatsumi were estimated by the following procedure, because it was found that eq. (11) can not be applied directly to this case.

(a) $(\tau/\sigma_v')_\ell$ -values were obtained as $(\tau/\sigma_v')_\ell = R_\ell$ at the points where undisturbed samples were obtained and the values of R_ℓ were obtained by dynamic triaxial tests.

(b) $(\tau/\sigma_v')_\ell$ -values at the points where only N-values were measured and no other informations are available were estimated as

$$(N, \sigma_v') \rightarrow D_r = 21 \sqrt{\frac{N}{\sigma_v' + 0.7}} \quad (\text{eq. (9)})$$

$$\rightarrow R_\ell = 0.21 + 0.0021 D_r \quad (\text{eq. (13)}) \rightarrow (\tau/\sigma_v')_\ell = R_\ell \quad (\text{eq. (7)})$$

$$R_\ell = 0.21 + 0.0021 \cdot D_r \quad \dots\dots\dots (13)$$

Eq. (13) represents the average relationship of data of tests conducted in this study which are shown in Fig. 26.

(c) $(\tau/\sigma_v')_\ell$ -values at the points where N-values and grain size accumulation curves of samples were obtained were estimated as follows. First, it was supposed that the reason why the $(\tau/\sigma_v')_\ell$ -values of sands tested in this study are larger than those of clean sands for the same value of $D_r = 21 \sqrt{N/(\sigma_v' + 0.7)}$ is that sands tested in this study are fine sands and include fine soils less than 0.074 mm to some extent. Therefore, DR_ℓ was

defined as the difference in the dynamic shear strength $(\tau/\sigma_v')_\ell = R_\ell$ for the same value of D_r between the value of the data obtained in this study and the average value of clean sands. This definition is illustrated in Fig. 27. As almost all of the samples tested in this study were fine sands where the mean grain size D_{50} were less than 0.3 mm, only the correlation between the DR_ℓ -value and the content of fine soils were examined as shown in Fig. 28. While there is a wide scattering in the data plots it may be seen from this figure that DR_ℓ -values increase with the increase in the content of fine soils S . This relationship may be expressed as

$$D(\tau/\sigma_v')_\ell = DR_\ell = 0.022S \sim 0.0075S \leq 0.175 \quad \dots (14)$$

Therefore, the $(\tau/\sigma_v')_\ell$ -values in this case were estimated as follows.

$$(\tau/\sigma_v')_\ell = \left\{ (\tau/\sigma_v')_\ell \text{ by eq. (11)} \right\} + \left\{ D(\tau/\sigma_v')_\ell \text{ by eq. (14)} \right\} \quad \dots (15)$$

The Comparison between $(\tau/\sigma_v')_e$ and $(\tau/\sigma_v')_\ell$ at Akebono-Tatsumi Site

Firstly, the critical N -value = 10 specified in the specifications for the Earthquake-Resistant Design of Highway Bridges were evaluated as follows. In Fig. 30 shown is the $(\tau/\sigma_v')_e$ which was calculated at a point at Akebono-Tatsumi site. In this calculation, the maximum acceleration at the base rock was taken as 150 gal and four different earthquake motions recorded at the depth of 127 m near this site were utilized. Also shown in this figure are the values of $(\tau/\sigma_v')_\ell$ which were obtained from eq. (11) on the assumption that the N -values at this site were 10 irrespectively of depth and the sands in this point were clean. Therefore, the dynamic shear strength R_ℓ of these sands can be obtained from eq. (10). It can be seen from Fig. 30 that the value of $(\tau/\sigma_v')_\ell$ obtained as above are almost identical to the value of $(\tau/\sigma_v')_e$ for the entire depth of interest. Therefore, it was evaluated that the critical N -value of 10 is the proper value for clean sand deposits for the value of $(\tau/\sigma_v')_e$ which were estimated in this study.

Nextly, the values of $(\tau/\sigma_v')_\ell$ obtained by eq. (11) using the measured N -values and σ_v' were compared with the calculated values of $(\tau/\sigma_v')_e$ as shown in Fig. 31. It can be seen from this figure that the overall values of $(\tau/\sigma_v')_\ell$ are less than those of $(\tau/\sigma_v')_e$. This means that this deposit

may liquefy during severe earthquake motions. However, as eq. (11) is based on the test results on clean sands, this conclusion may be improper.

Then, $(\tau/\sigma_{v'})_L$ -values were estimated by the three methods (a), (b) and (c) which were denoted above. In Fig. 32, black aquars represent the $(\tau/\sigma_{v'})_L$ -values obtained by the method (a), hollow circles and triangle circles represent the $(\tau/\sigma_{v'})_L$ -values obtained by the method (b) and the marks ---| represent the ranges of $(\tau/\sigma_{v'})_L$ -value obtained by the method (c). And the hatched zone shows the estimated range of $(\tau/\sigma_{v'})_L$. It can be seen from this figure that the estimated values of $(\tau/\sigma_{v'})_L$ are, in general, larger than the estimated values of $(\tau/\sigma_{v'})_e$. This conclusion is rather different from that which can be derived from Fig. 31. This means that when the liquefaction potential of sand deposits are evaluated on the basis of dynamic triaxial tests, these tests should be conducted on undisturbed samples if these samples include much fine soils.

An additional survey was performed at the Haneda site. The data obtained from this survey were added to those of the Akebono-Tatsumi site as shown in Figs. 33 and 34. In these figures, data points with letters A means those of the Akebono-Tatsumi site. It is seen from these figures that the general trend is almost similar for both sites.

The Critical N-values Based on Dynamic Triaxial Tests on Undisturbed Sand Samples

Fig. 33 shows the relationship between the dynamic shear strength of undisturbed samples R_L and the relative density obtained from N-values and in situ effective overburden pressures $\sigma_{v'}$ as

$$D_r = 21 \sqrt{\frac{N}{\sigma_{v'} + 0.7}}.$$

On the basis of this relationship, the critical N-values were determined for fine sands with different content of fine soils as follows. It should be noted that the almost mean grain diameter D_{50} of sands tested are less than 0.3 mm. Therefore, the following discussions can be applied only for sands with D_{50} less than 0.3mm. While there is a large scattering in the data shown in Fig. 34, the average relationship of these data plots can be represented conservatively as

$$DR_L = 0.0035 S \dots\dots\dots (16)$$

where S denotes the content of fine soils in percentages. The value of R_ℓ is represented as

$$R_\ell = 0.0042 \times 21 \sqrt{\frac{N}{\sigma_v' + 0.7}} + 0.0035 S$$

$$= 0.0882 \sqrt{\frac{N}{\sigma_v' + 0.7}} + 0.0035 S \quad \dots\dots\dots (17)$$

To obtain the relationship of fine sands with D_{50} less than 0.3 mm between the critical N -value for the evaluation of liquefaction potential and the content of fine soils, the critical N -value of 10 was selected as the value for clean sand deposits with $\sigma_v' = 0.5 \text{ kg/cm}^2$. This value was evaluated from the above-mentioned specification. In this specification, it is specified that saturated sandy soil layers within 10 m of the actual ground surface may be vulnerable to liquefaction with a standard penetration test N -value less than 10. Therefore, as the mean value of σ_v' of this sandy soil layer, 0.5 kg/cm^2 was selected. If this critical N -value of 10 can be applied to clean sand deposits like those at Niigata city, the standard value of R_ℓ , $R_{\ell 0}$ can be obtained by substituting $N = 10$, $\sigma_v' = 0.5$ and $S = 0$ into eq. (17) as

$$R_{\ell 0} = 0.255 \quad \dots\dots\dots (18)$$

Nextly, by substituting $R_\ell = 0.255$ and $\sigma_v' = 0.5$ into eq. (17), the relationship between the critical N -value for sands including fine soils and the content of fine soils S as

$$N = 0.00189 (72.9 - S)^2 \quad \dots\dots\dots (19)$$

From eq. (19), critical N -values for various values of S can be obtained

as	$N = 10$	for	$S = 0\%$,
	$N = 7.5$	for	$S = 10\%$,
	$N = 5.3$	for	$S = 20\%$,
	$N = 3.5$	for	$S = 30\%$ and
	$N = 2.0$	for	$S = 40\%$.

From these values, the design critical N -values for sands deposits whose D_{50} -values are less than 0.3 mm can be tentatively proposed as

$N \geq 10$	for	$S = 0 - 10\%$,
$N \geq 7$	for	$S = 10 - 20\%$,
$N \geq 5$	for	$S = 20 - 40\%$ and
$N \geq 3$	for	$S = 40 - 60\%$.

Since these values were obtained from the less number of field data, more research works are necessary. Therefore, the critical N-values recommended for design purposes should be determined after more field data are accumulated.

§ 5. CONCLUSIONS

Dynamic behavior of pile foundations in liquefying soil deposits is considered to be one of soil-structure interaction problems during earthquakes. As this problem is very complicated, only qualitative results can be derived from the experimental tests described in the preceeding sections. The following remarks can be concluded.

1) Since liquefaction phenomena progress with respect to time and with respect to space, the effects of liquefied sands on the piles vary as functions of time and space. It can be normally recognized that the response of a pile foundation with a normal rigidity is small before liquefaction initiates, increases abruptly large after liquefaction begins, and again decreases when complete liquefaction takes place. During the complete liquefaction soils behave as heavy liquid, and piles are subjected to hydrodynamic pressures due to the liquefied soils.

2) The relationship among the natural frequency of the pile foundation supported by the surrounding soil deposits before liquefaction initiates, the natural frequency of the pile foundation submerged in completely liquefied soils and the predominant frequency of input motion is the most important factor. Dynamic behaviors of pile foundations in the course of liquefaction of sand layers will be considerably affected by this relationship.

3) While the complete liquefaction continues, soil-pile foundation system behave as a submerged structure. Since its natural frequency gets lower, it may resonate when subjected to seismic motion with longer periods. Accordingly it will be important to assess the components of longer periods involved in induced seismic motions.

For simplified procedure in evaluating liquefaction potential, critical N-values are proposed on the basis of dynamic triaxial tests on undisturbed samples and the standard penetration tests.

While additional research works are necessary, tentatively proposed critical N-values for sands whose D₅₀-values are less than 0.3 mm are as follows: N $\div$ 10 for the content of fine soils S of 0 to 10%,

N $\div$ 7 for S = 10 - 20%,

N $\div$ 5 for S = 20 - 40% and

N $\div$ 3 for S = 40 - 60%

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TEST CASE	SAND	Dr ³⁾ (%)	Number ⁴⁾ of Piles	Head Weight (kg)	f _p (Hz)	f _i (Hz)	a ₁₁ (gal)	Shaking Period (sec.)	f _g (Hz)
T1	Iruma ¹⁾	40	9	44	3.8	12	40 ~ 100	60	18
T2	Toyoura ²⁾	27	4	53	4	10	60 ~ 80	30	24
T3		34	6	18	10	20	80 ~ 130	15	
T4		24	6	18	10	10	60 ~ 80	30	
T5		40	9	13	14	10	60 ~ 120	30	

- 1) Iruma Sand ($G_s=2.88$, $D_{50}=0.52\text{mm}$, $U_c=1.52$, $e_{\max}=0.93$, $e_{\min}=0.66$)
- 2) Toyoura Sand ($G_s=2.64$, $D_{50}=0.16\text{mm}$, $U_c=1.46$, $e_{\max}=0.96$, $e_{\min}=0.64$)
- 3) Relative Density
- 4) Each Pile is made of aluminum bars with 70cm in length and 2cm in diameter.
- 5) Natural Frequency of Pile Foundation in Water
- 6) Frequency of Input Motion
- 7) Amplitude of Base Acceleration before Perfect Liquefaction
- 8) Estimated Natural Frequency of Non-liquefied Sand Layer ($\gamma=10^{-4}$)

Table 1. List of Test Cases

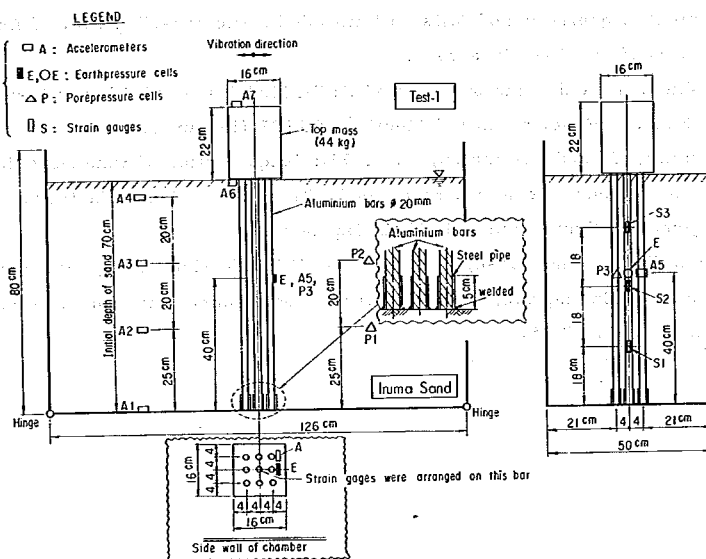


Fig. 1 Test Arrangements for Test 1

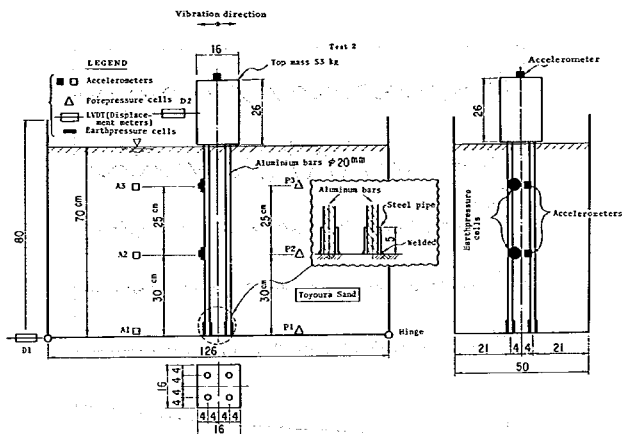


Fig. 2 Test Arrangements for Test 2

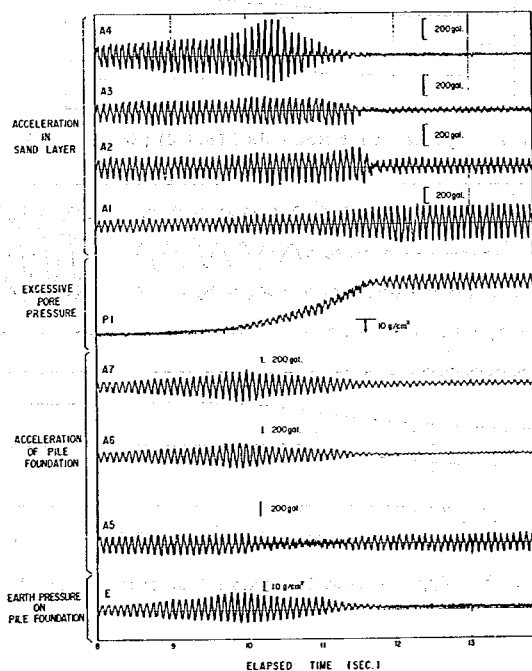


Fig. 3 Typical Test Records (Test 1) (See Fig. 1)

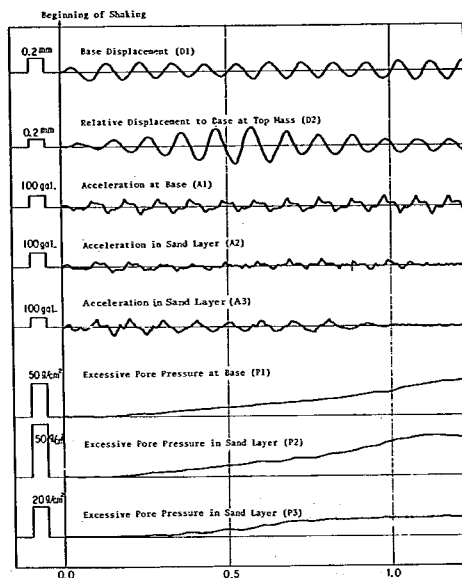


Fig. 4 Typical Test Records (Test 2) (See Fig. 2)

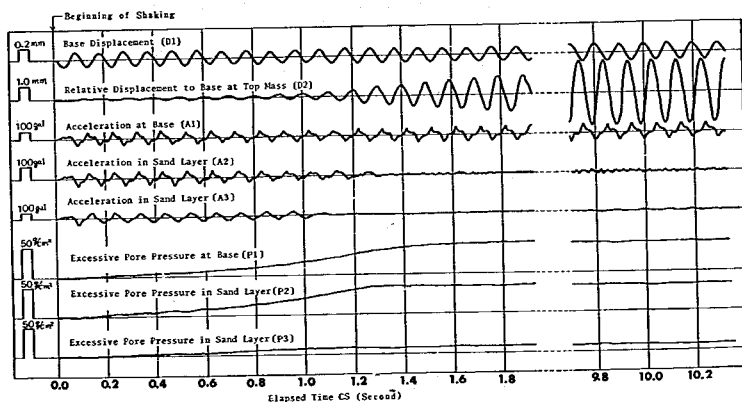


Fig. 5 Typical Test Records (Test 4) (See Fig. 2)

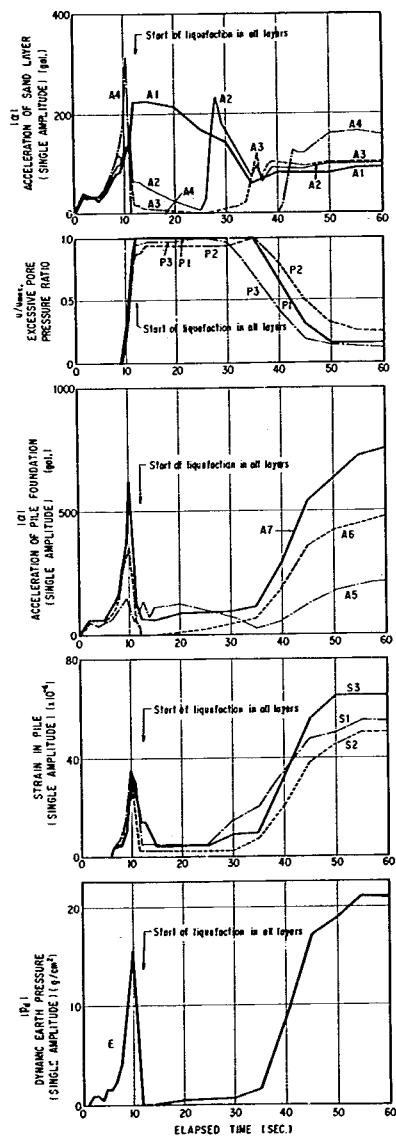


Fig. 6 Amplitudes of Recorded Quantities (Test 1)

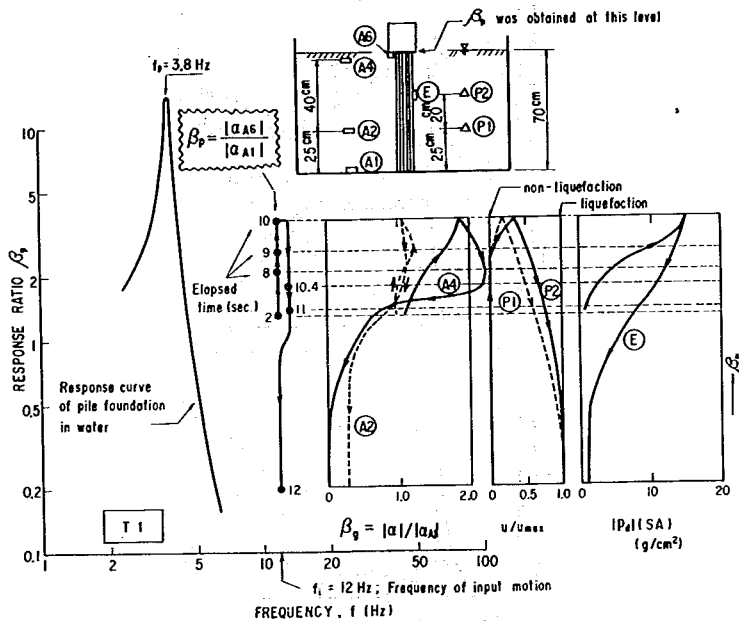


Fig. 7 Variation of Response of Pile Foundation in the course of Liquefaction of Sand Layer (Test 1)

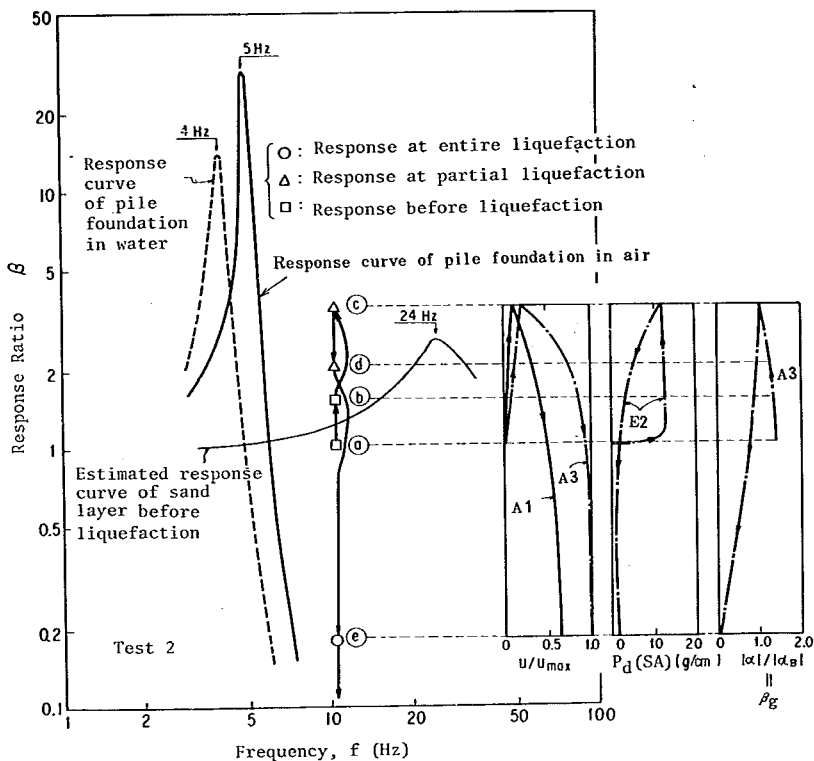


Fig. 8 Variation of Response of Pile Foundation in the course of Liquefaction of Sand Layer (Test 2)

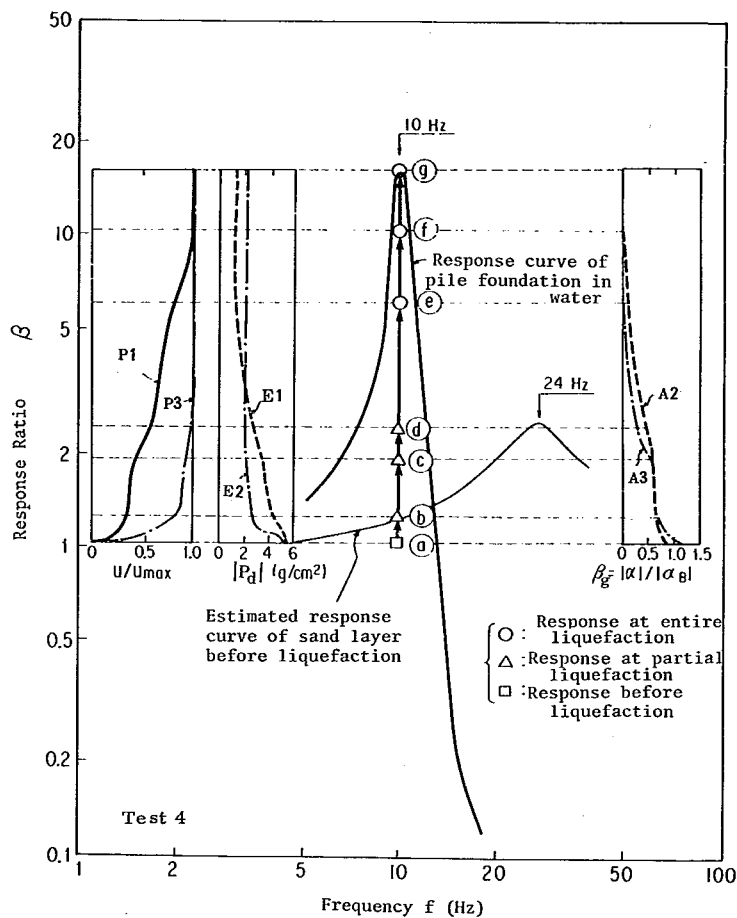


Fig. 9 Variation of Response of Pile Foundation in the course of Liquefaction of Sand Layer (Test 4)

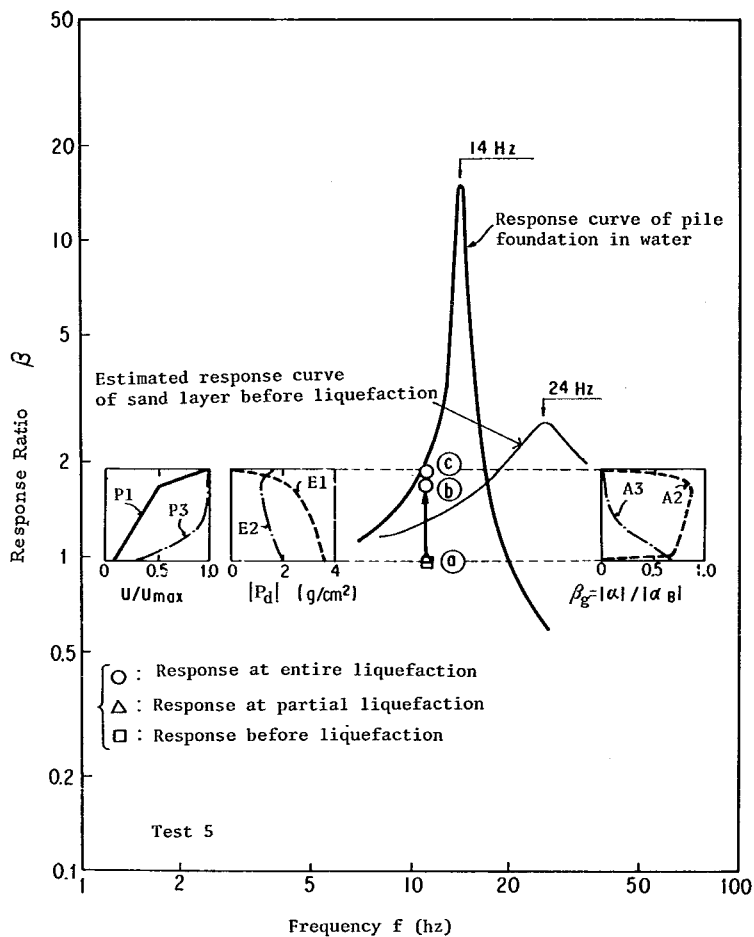


Fig. 10 Variation of Response of Pile Foundation in the course of Liquefaction of Sand Layer (Test 5)

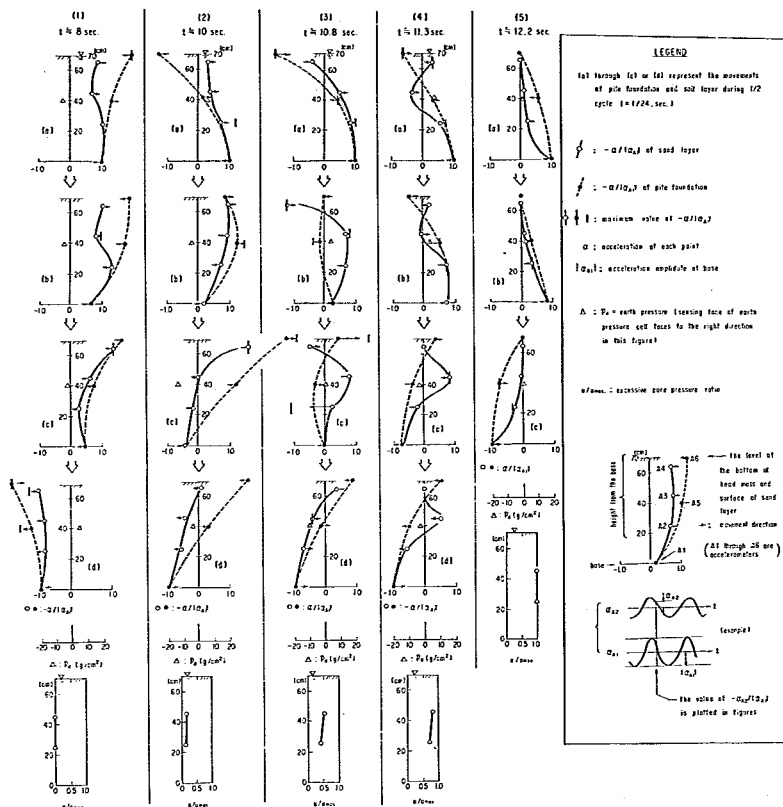


Fig. 11 Instantaneous Positions of Sand Layer and the Pile Foundation Model (Test 1)

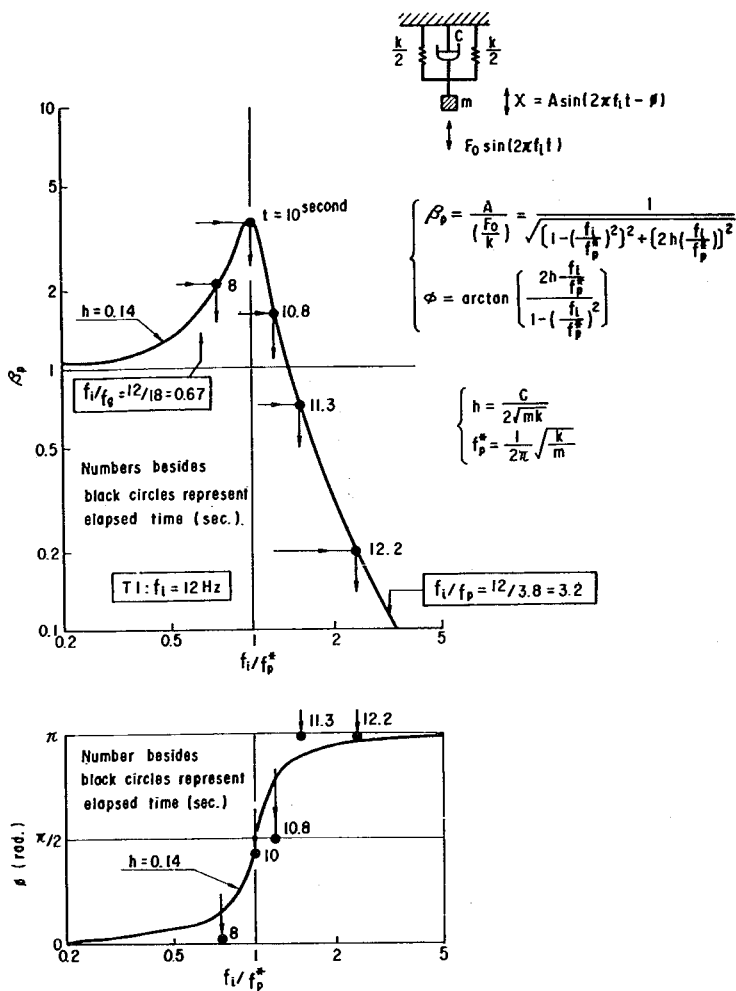


Fig. 12 Estimation of Apparent Natural Frequency of Pile Foundation Model Embedded in Liquefying Sand Layer (Test 1)

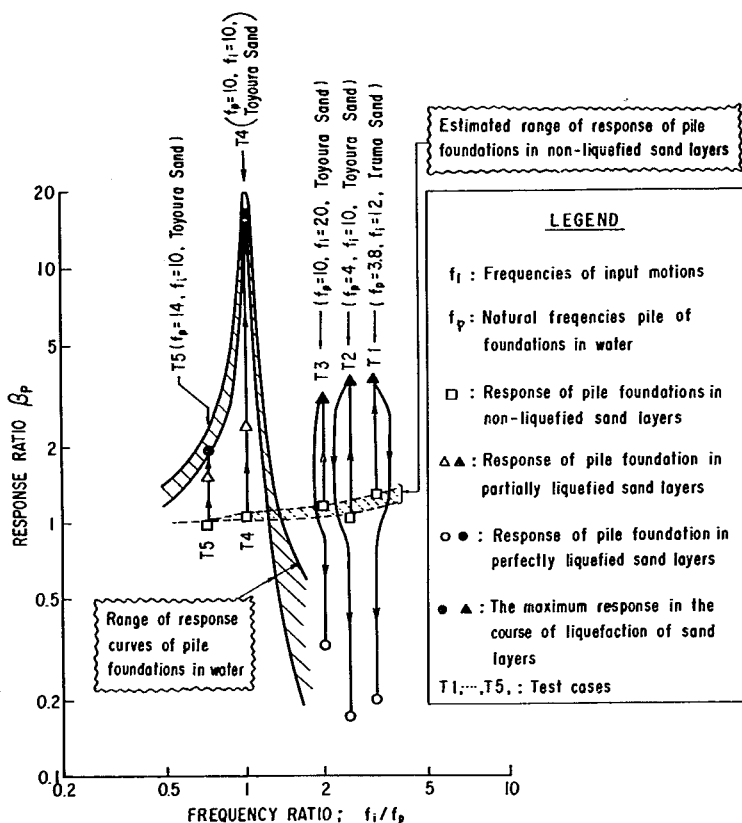


Fig. 13 Summary of Whole Tests

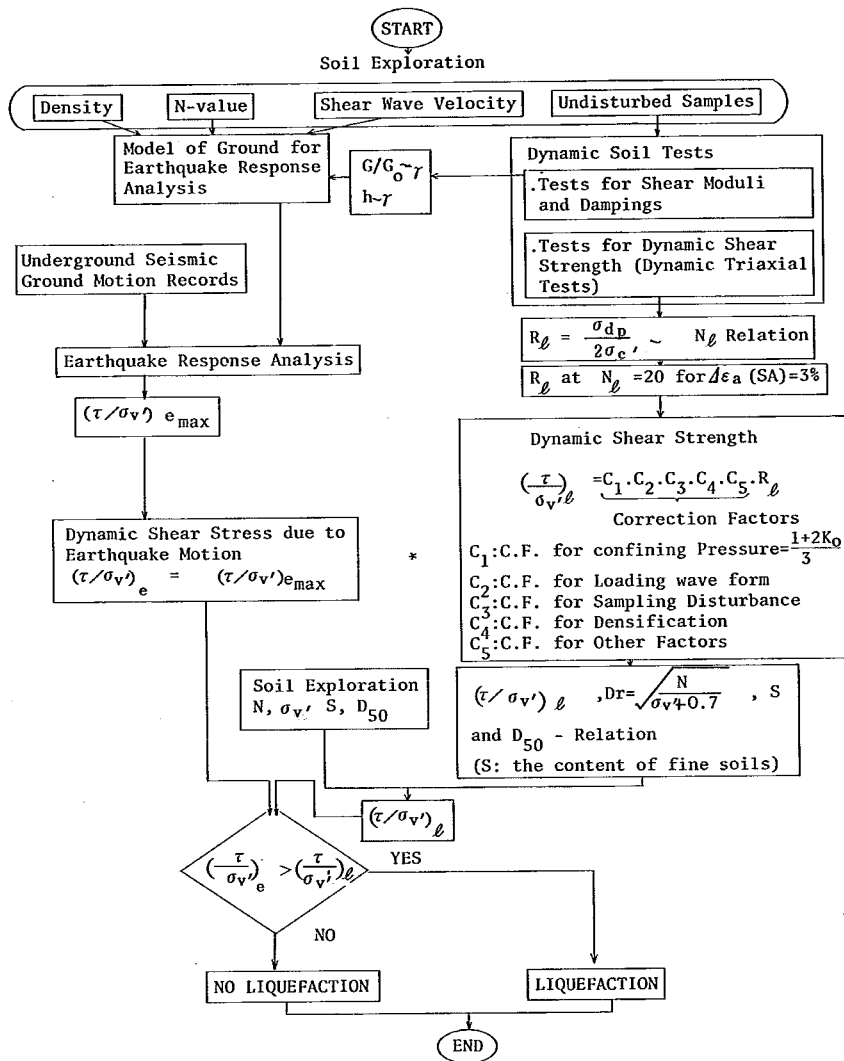


Fig. 14 Flow Chart of Precise Evaluation of Liquefaction Potential

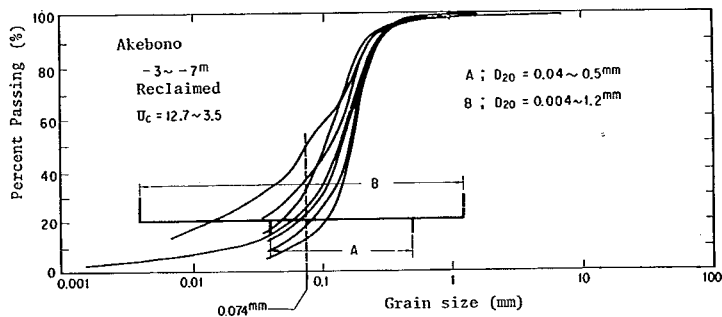


Fig. 15 Typical Grain Size Accumulation Curve
(Akebono, Reclaimed Sand Layer)

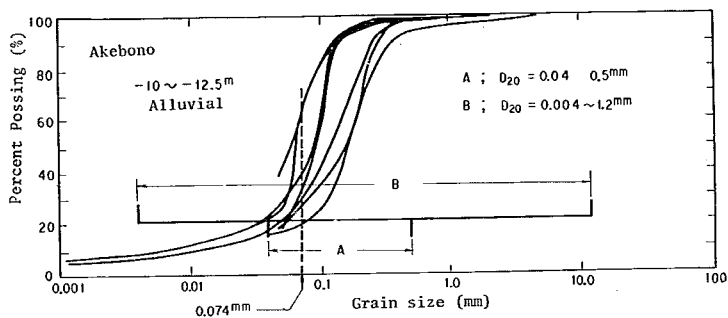


Fig. 16 Typical Grain Size Accumulation Curve
(Akebono, Alluvial Sand Layer)

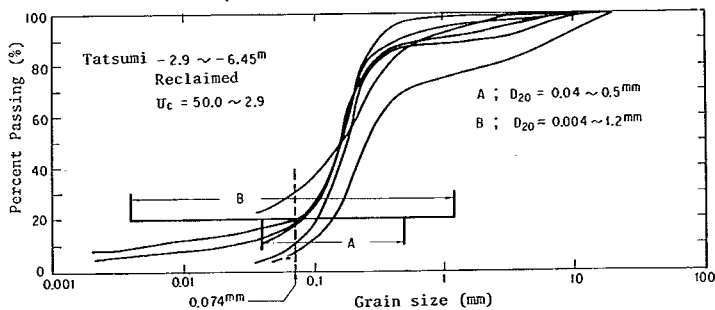


Fig. 17 Typical Grain Size Accumulation Curve
(Tatsumi, Reclaimed Sand Layer)

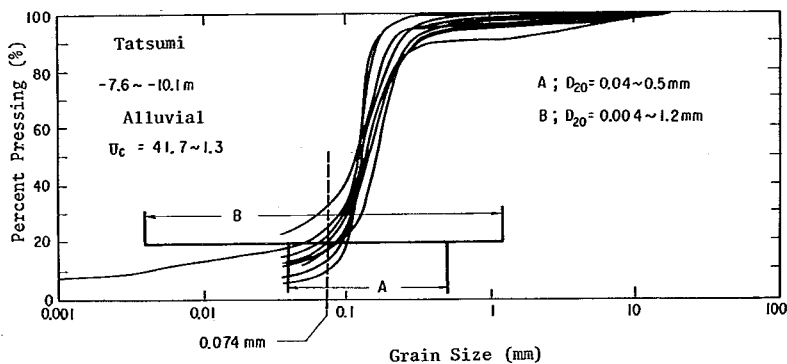
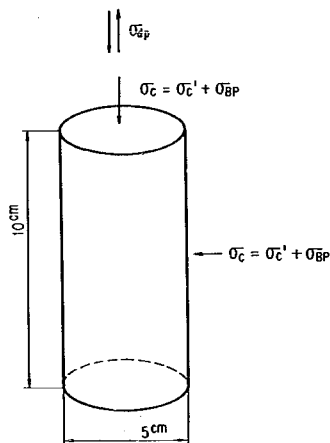


Fig. 18 Typical Grain Size Accumulation Curve
(Tatsumi, Alluvial Sand Layer)



σ_c' (Effective confining pressure) = σ_v' (Insitu effective overburden pressure)
 σ_{BP} (Back pressure) $\cong \gamma_{wh}$ (Insitu static porepressure)

All of the tests were performed on samples with Skempton's B-value larger than 0.96.

Fig. 19 Stress Condition in Dynamic Triaxial Tests

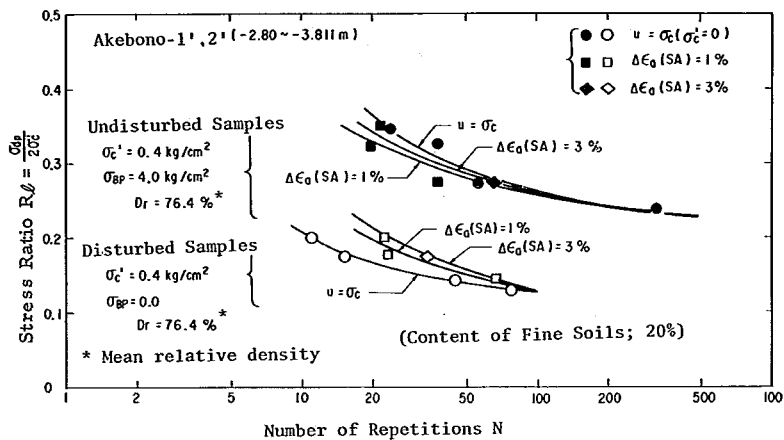


Fig. 20 Typical Dynamic Triaxial Test Results (Akebono)

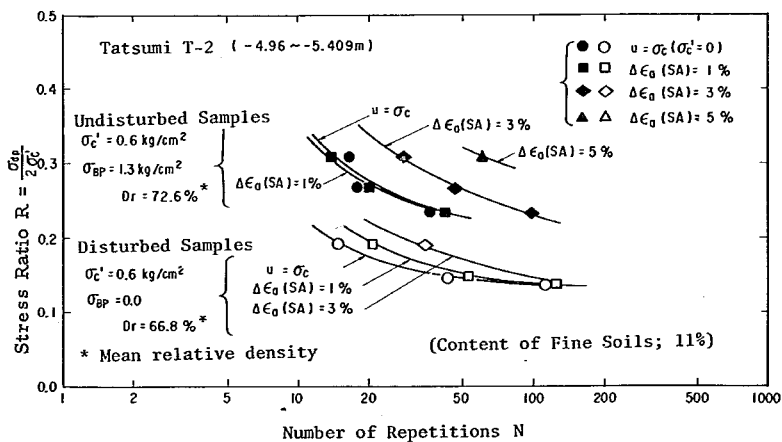


Fig. 21 Typical Dynamic Triaxial Test Results (Tatsumi)

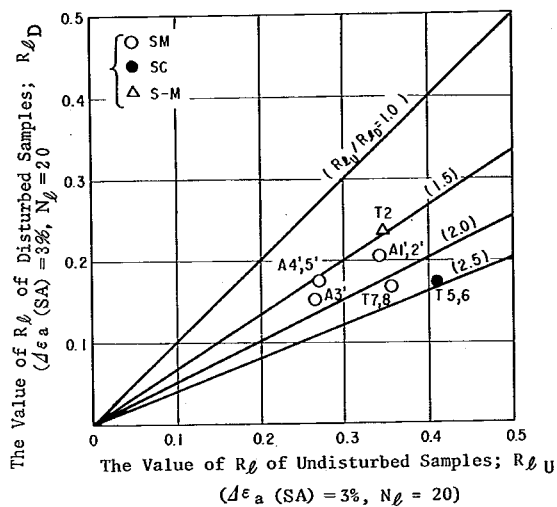


Fig. 22 The Relationship of Dynamic Shear Strength between Undisturbed Samples and Disturbed Samples

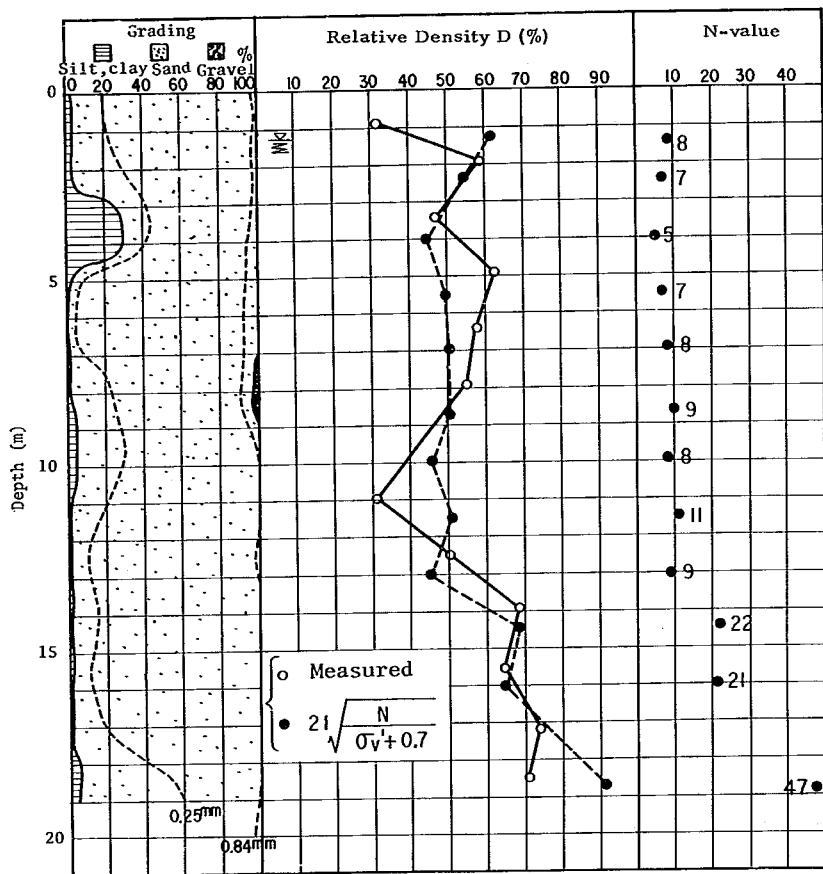


Fig. 23 The Comparison between Measured Relative Density

$$Dr = \frac{e_{\max} - e}{e_{\max} - e_{\min}} \times 100 \text{ and } Dr = 21 \sqrt{\frac{N}{\sigma_v' + 0.7}}$$

at Kawagishi-cho, Niigata

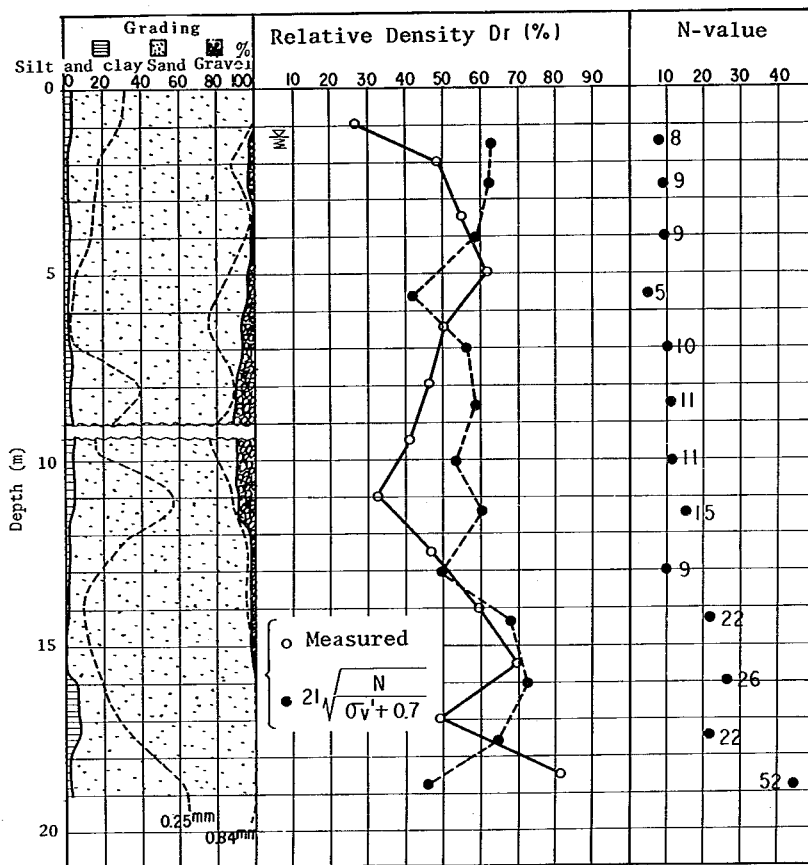


Fig. 24 The Comparison between Measured Relative Density

$$Dr = \frac{e_{max} - e}{e_{max}} \times 100 \text{ and } Dr = 21 \sqrt{\frac{N}{\sigma_v' + 0.7}}$$

at Kawagishi-cho, Niigata

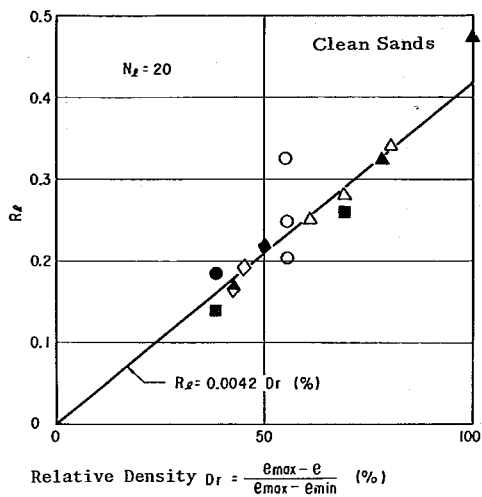


Fig. 25 R_f - D_r Relationship of Clean Sands

Table 2 Summary of Dynamic Triaxial Test Results on Clean Sands

References	D_{50}	$<74\mu$ (%)	e_{max}	e_{min}	$R_{f=20}$	D_r	Legend ¹ in Fig. 22	Note
Seed and Lee(1966)		0	1.03	0.61	0.185	38 %	●	0.297~0.149mm Sacrament River Sand
Lee and Seed(1967)		0	"	"	0.325	78	▲	"
					0.475	100		"
Lee and Fitton (1968)	6.5	0			0.427	50	◆	$e = \pm 25\%$
	0.68	0			0.287	"		"
	0.10	0			0.217	"		"
Finn, Pickering and Bransby(1971)	0.40	0	0.82	0.50	0.26	69	■	Ottawa Sand
					0.14	38		ASTM C109
Shibata (1972) Tanimoto			1.007	0.590	0.205	55	○	Niigata Sand
			"	"	0.25	"		
			"	"	0.325	"		
Hagino, Imai, Shinozaki (1974)	0.712	0	0.921	0.623	0.34	80	△	Sengenyama Sand
			"	"	0.28	69		
			"	"	0.25	61		
Ishihara & Yasuda (1972)	0.260	2	0.99	0.55	0.19	45	◇	Niigata Sand
" (1973)	0.40	0	1.03	0.48	0.165	42.5	◆	Fuji River Sand

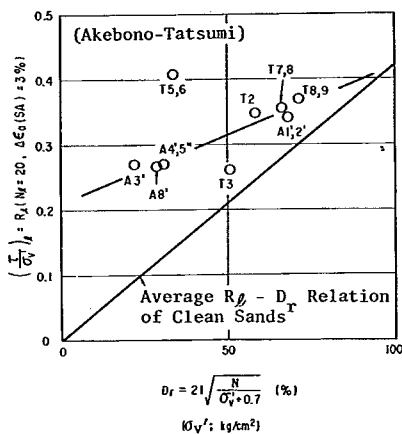


Fig. 26 Re - D_r Relationship of Sands of Akebono-Tatsumi Site

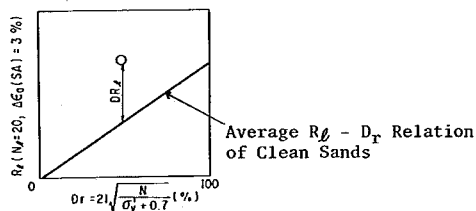


Fig. 27 The Definition of DR_{ell}

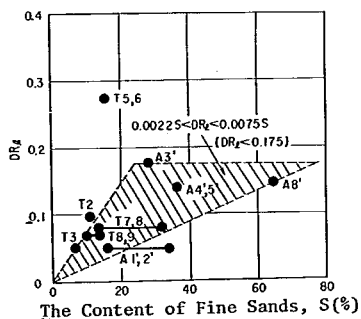


Fig. 28 $DR_{ell} - S$ Relationship

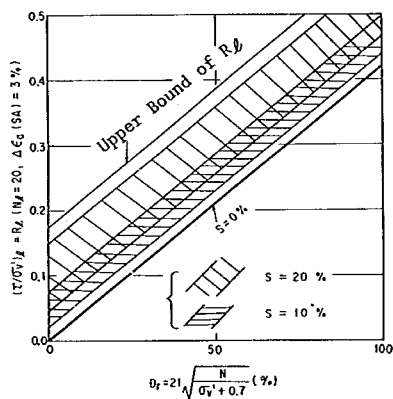


Fig. 29 R_L - D_r Relationship Based on Fig. 28

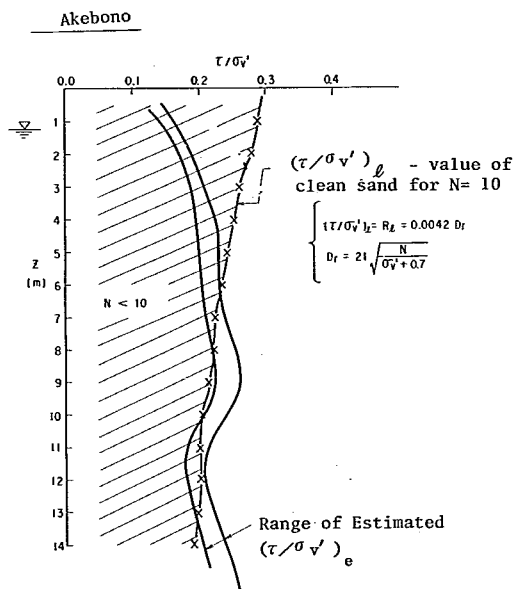


Fig. 30 Comparison of $(\tau/\sigma'v')_L$ of Clean Sand for $N = 10$ with $(\tau/\sigma'v')_e$

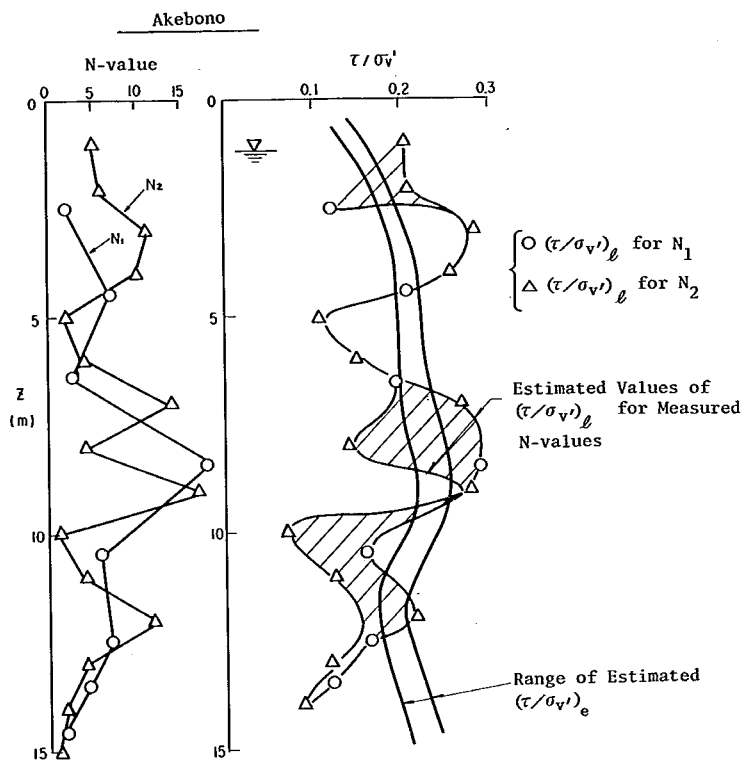


Fig. 31 Comparison of $(\tau / \sigma_v')_{\ell}$ of Clean Sand for Measured N-values with $(\tau / \sigma_v')_e$

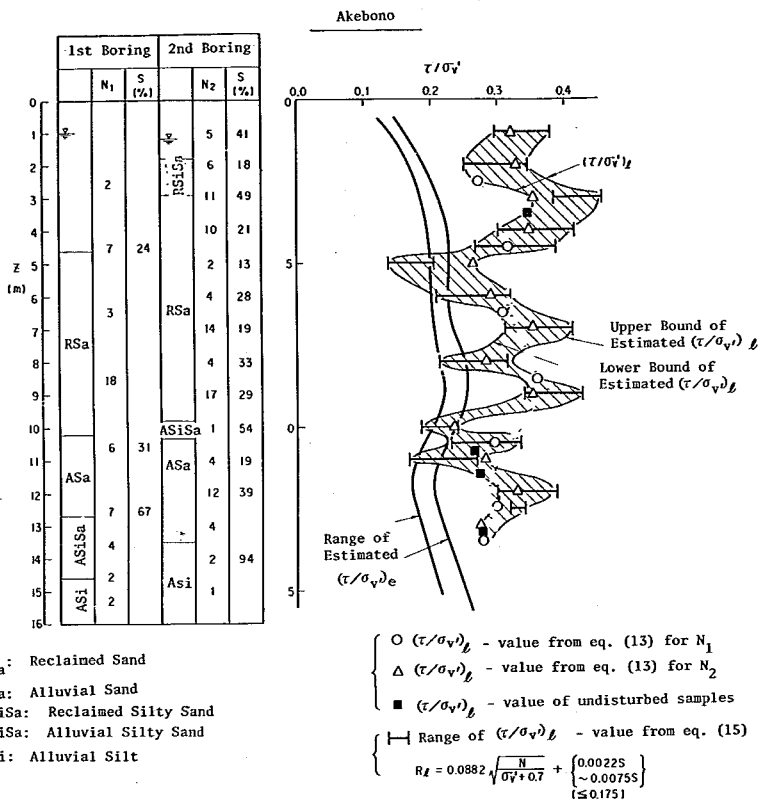


Fig. 32 Comparison of (τ/σ_v') Based on Dynamic Triaxial Tests on Undisturbed Samples with $(\tau/\sigma_v')_e$

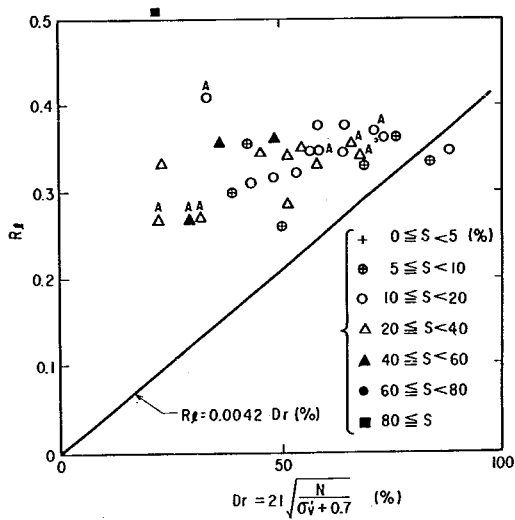


Fig. 33 R_L - Dr Relationship of Sands of Two Sites

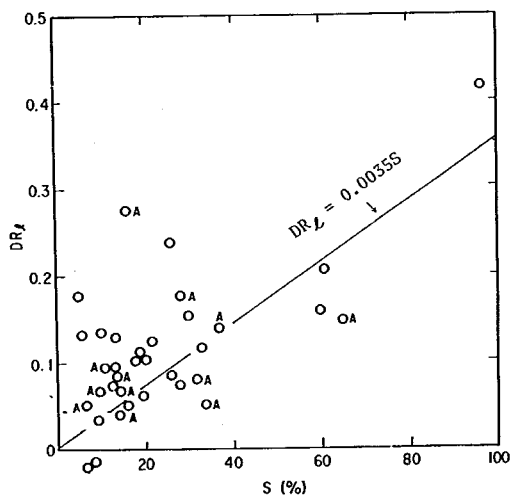


Fig. 34 DR_L - Dr Relationship of Sands of Two Sites

EFFECTS OF SOIL LIQUEFACTION ON DYNAMIC BEHAVIOR OF PILE FOUNDATIONS

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** Public Works-Research Institute, Ministry of Construction

SYNOPSIS In order to evaluate dynamic behavior of pile foundations or embedded structures during earthquakes, it is essential to estimate the effects of the surrounding soils on those structures. The authors have conducted a literature survey on the effects of liquefied soils on bridge foundations and also laboratory experiments using models of structures including surrounding soils intending to clarify the effects of liquefied soils on pile foundations and to establish design methodology of pile foundations considering the effects of soil liquefaction.

INTRODUCTION

In order to appropriately evaluate dynamic behavior of pile foundations or embedded structures during earthquakes, it is essential to estimate the effects of the behavior of the surrounding soils on those structures. It can be supposed that the surrounding soils do not move severely in normal cases and normally resist movements of the structures, but sometimes the surrounding soils exert large movements of the structures when the soils vibrate severely as a result of a resonant state of the soils and furthermore in special cases the surrounding soils do not support these structures when soil liquefaction takes place.

To provide a proper solution to this problem, it seems necessary to precisely analyze the behavior of the structures considering the effects of soil-structure interactions. When the surrounding soils liquefy during earthquakes, the problem becomes more complicated. In this case, the effects of liquefied soils on the structures will be much severer.

The authors have conducted a literature survey on the effects of liquefied soils on bridge foundations and also laboratory experiments using models of structures including surrounding soils. This attempt is to clarify the effects of liquefied soils on pile foundations, and to establish design methodology of pile foundations considering the effects of soil liquefaction.

The results of this study can be summarized as follows:

- 1) The supporting capacities of surrounding soils may become considerably small when liquefaction takes place. In some cases, dynamic forces of soils acting on piles may become considerably large in the course of liquefaction. When the complete liquefaction takes place, both of the supporting capacities and the dynamic forces

can be approximately estimated by assuming that a liquefied sand layer is heavy liquid.

- 2) In the course of liquefaction, soil-pile foundation systems may resonate to the input motions, and may vibrate severely.

DAMAGE TO PILE FOUNDATIONS DUE TO LIQUEFACTION

It has been recognized that one of the main causes of the damage to bridges which were constructed in alluvial deposits or reclaimed lands is unstable performances of ground soils. It is also well known that liquefaction of sandy soils induce catastrophic movements of grounds and loss of bearing capacities of grounds. A number of damages to bridge structures with pile foundations were observed during the Niigata Earthquakes of 1964 in Japan (Iwasaki (1973)) and during the Alaska Earthquake of 1964 in United States (Kachadoorian (1968)). Some of similar damage to the above are reported for other earthquakes in Japan (Iwasaki (1973)) and for the San Francisco Earthquake of 1906 (Wood and Hoose (1976)). On the other hand, liquefaction has been observed in alluvial deposits and reclaimed lands during major earthquakes including the above earthquakes. Only in Japan, 44 earthquakes induced liquefaction phenomena during the period from 1872 to 1968 (Kuribayashi and Tatsuoaka (1974)).

In viewing damages to engineering structures by soil liquefaction, numerous studies on evaluation of liquefaction potential of sandy deposits have been conducted by using dynamic soil testing equipments such as a dynamic triaxial apparatus and a dynamic simple shear apparatus. Furthermore, conducted are some studies using shaking table on the effects of the loss of bearing capacities caused by liquefaction on the behavior of heavy structures

(Yoshimi et al. (1975), Ishihara et al. (1975)). However, a little efforts have been made on studies of effects of soil liquefaction on the dynamic behavior of pile foundations (Shikamori, Sato and Hakuno (1973), Yoshida and Uematsu (1975), Iwasaki, Tatsuoaka and Sakaba (1976)). Accordingly methodology of aseismic design of pile foundations considering the effects of liquefaction is still in the process of formation.

Intending to establish a rational aseismic design criterion of pile foundations which are constructed in soil deposits with high liquefaction potential, two series of model tests of embedded structures and of pile foundations placed in liquefying sand deposits were performed using shaking tables.

PRELIMINARY EXPERIMENT

A preliminary study on the effects of soil liquefaction on embedded structures was performed at the Earthquake Research Institute, the University of Tokyo. Fig. 1 indicates a schematic drawing of the preliminary experiment. Two small plastic models were embedded into a sand layer placed on a shaking table. The plastic models are hung by a lever on which a strain gauge is put on to indirectly measure the behavior of the box during the vibration.

Fig. 2 shows an example of test results, in which time histories of acceleration and displacement of the table, acceleration in the sand and strain in the lever are illustrated. Although the acceleration in the sand is rather stationary, the strain which can be proportional to the seismic force acting on the model, suddenly becomes large at a certain moment, takes its maximum after a short time interval, and decreases gradually. This change of strain level is corresponding to the degree of liquefaction of soils. When complete liquefaction takes place, the strain levels gets small considerably.

EXPERIMENT ON MODELS OF PILE FOUNDATIONS

The second series of experiments was performed at the Public Works Research Institute. In this experiment small models of pile foundations were prefabricated on another shaking table. Five different tests listed in Table 1 were conducted. Typical test arrangements in the case of Test 1 are illustrated schematically in Fig. 3. A saturated loose sand layer including a small model of pile foundation was made in a container box on a shaking table. To actuate the shaking table, a electromagnetic actuator was used for Test 1. A displacement-controlled hydraulic actuator was used for Tests 2 to 5. Sand layers were prepared by pouring air dry sand into water. Initial relative densities of sand layers ranged from 24 to 40 per centages. The water level coincided with the sand surface at the beginning of shaking. As shown in Fig. 3 accelerations and strain gauges were installed to measure movements of the shaking table, sand layers and pile foundation models. Earthpressure cells and porepressure cells were also equipped to measure earthpressures acting on piles and excessive porewater pressures. Sinusoidal table motions were applied in the whole tests (Tests 1 to 5). The actual acceleration form measured at the table was of triangular form in the case of Test 1 as shown in Fig. 4 where the table acceleration is denoted as A_1 . However, in the other tests (Test 2 to Test 5) where a hydraulic

actuator was used, the measured table acceleration was close to the sinusoidal motion. Several motions with different frequencies were adopted as input motions.

Frequencies were selected on the basis of the natural frequencies of pile foundation models submerged in water. The base acceleration was determined in view of outbreak of gradual increase in excessive porewater pressure in sand layers. Actuators used in the experiments were not enough to keep a constant base acceleration during a test run. This is due to a great change in rigidity of sand layers which is caused by soil liquefaction. Actually, the base acceleration changed in the course of liquefaction of sand layers.

Four different pile foundation models were examined. Their natural frequencies when submerged in water are 3.8, 4.0, 10 and 14 Hz, respectively. Their natural frequencies in air are slightly higher than those in water. Each of these models consists of several solid aluminum bars with the diameter of 2 cm and a top mass. The values of the natural frequencies of models were adjusted by choosing the number of bars and the weight of the top mass. These models were fixed at the bottom as shown in Fig. 3 to facilitate the boundary condition. The degree of the fixation would be considered to be larger than those in actual cases.

Frequencies of input motion and natural frequencies of pile foundation models submerged in water were determined in the following manner. Objectives of model tests in the field of soil mechanics can be classified into two. The first objective is to find fundamental parameters required to be taken into account when analyzing the behavior of the prototype with an analytical method. This analytical method can be, of course, applied to the model test. In this case, the rigorous satisfaction of principles of similarity is not of first importance. The second objective is to quantitatively estimate the behavior of the prototype from the results of a model test. In this case principles of similarity should be determined and satisfied. At present, it is of first importance to find the controlling factors in the behaviors of structures in liquefying soils. Therefore, reported herein is the results of tests of the first category. Accordingly, the similitude between the prototype and the model was satisfied in terms of the following, but others, for example permeability of sand layers could not satisfy the principles of similarity.

(i) Length (L): Scale ratio considered for length was of the order of 1:10 to 1:30. Therefore, model sand layers with the height of 70 cm correspond to actual sand deposits of 7 to 21 m in depth.

(ii) Density of sand layer (M/L^3): Since actual sands were used, the ratio of density of soils was 1:1.

(iii) Acceleration (LT^{-2}): Since body force induced by gravity is the same between models and prototypes, the ratio for body force induced by horizontal motions should be on the order of 1:1. And the ratio for horizontal acceleration was also on the order of 1:1.

(iv) Time (T): From (i) and (iii) the ratio for time is determined as $1/L = 1/10$ to $1/30 = 1:3.2$ to 1:5.5.

(v) Natural frequency of model pile foundation submerged in water (f_p), frequency of input motion (f_1), and fundamental natural frequency of sand layer with excessive porewater pressure of zero and at shear strain level of $\gamma = 10^{-4}$ or less (f_0) (T^{-1}): These scale ratios are determined from (3v) as $1:1/T=1:1/3.2$ to $1:1/5.5=1:0.32$ to $1:0.18$. Estimated frequencies for f_0 of model sand layers were 18 and 24 Hz. These values correspond to the natural frequencies of the prototype sand layers, 3.2 to 7.7 Hz. On the other hand, the fundamental natural frequency of soil deposit f_1 (Hz) can be approximately estimated as

$$f_1 = \frac{V_s}{4H} \quad (\text{Hz}) \quad (1)$$

in which V_s denotes the average shear wave velocity (m/sec) and H means the depth of soil deposit of interest (m). As the ordinary value of V_s of shallow alluvial sand deposits ranges from 150 to 250 m/sec., the fundamental frequency of sand deposits with the depth of 7 to 21 m ranges from $150/(4 \times 21) = 1.3$ Hz to $250/(4 \times 7) = 8.9$ Hz. These values correspond well to the values derived from scale ratios and the fundamental frequencies of model sand layers (3.2 to 7.7 Hz).

Since predominant frequencies of strong earthquake motion observed on the actual sand deposits are estimated less than the above values, the frequency of input motion in model tests f_1 should be less than those of natural frequencies model sand layers f_0 (18 and 24 Hz). Therefore, as the values of f_1 in the model tests, 10 Hz and 12 Hz were selected.

Furthermore, it can be estimated that the fundamental natural frequencies of ordinary bridges with pile foundations, f_p are less than the predominant frequency of strong earthquake motion observed on shallow sand deposits when the value of f_p are determined on the assumption that liquefied sand layers with the depth of 7 to 21 m are removed. Therefore, three of five model tests conducted were cases where the natural frequencies of model pile foundations submerged in water, f_p , are less than the frequency of input motion, f_1 , corresponding to the actual ordinary cases. In addition to these cases where $f_p < f_1$, other two cases were examined as $f_p = f_1$ and $f_p > f_1$ in order to evaluate the importance of the relationship between these two values f_p and f_1 in assessing the dynamic behavior of pile foundations in liquefying sand layers.

For Test 1, accelerations recorded at the table, at three positions in the sand layer and at three positions of the model pile foundation are shown in Fig. 4. Excessive porewater pressure in the sand layer and earthpressure acting on the pile foundation are also illustrated in the figure during the elapsed time of 8 to 14 seconds counting from the beginning of shaking. One of the most peculiar performance observed in these tests is non-stationary behaviors of the sand layer and the pile foundation and non-stationary dynamic interaction between them, while the table movement is almost stationary. In order to clarify the dynamic behavior of the sand layer and the pile foundation, amplitudes of dynamic quantities recorded were illustrated in Fig. 5. Since excessive porewater pressures reached to their maximum stationary values at the elapsed time between 11 and 12 seconds and the acceleration in the entire sand layer decreased to almost zero at the same time, it could be estimated that liquefaction in the entire sand layer has initiated at the elapsed time between 11 and 12 seconds. It should be noted that the maximum

response of the pile foundation occurred at the elapsed time of about 10 seconds when excessive porepressure did not reach to the maximum value yet and the sand layer vibrated in ordinary fashion. And at $t=10$ seconds, dynamic earthpressure acting on the pile foundation and strains in the pile had also their maximum values. In addition to this, it is also noted that the response of the pile foundation decreased to the smaller value after the entire sand layer had liquefied. Corresponding to this, earth pressure on the piles and strains in the pile also decreased to the small values. Following this, recovery from liquefaction started at the elapsed time of about 30 seconds, and after this moment the response of pile foundation increased again. Although this recovery phenomenon is also very interesting, this will not be discussed herein because such a phenomenon is unlikely to take place in the prototype during severe ground shaking in an actual earthquake. In this test, the ratio of time in the model and that in the prototype is considered on the order of $1:3.2 \sim 1:5.5$. Therefore, the elapsed time of 30 seconds in the model corresponds to 96 to 165 seconds in the prototype. This duration time seems longer than the duration of major ground motion during a strong earthquake. Furthermore, it is recognized that recovery from liquefaction began after the cease of earthquake motion in the actual earthquakes experienced. From these reasons, phenomena observed before the recovery from liquefaction will be discussed in this report.

In Fig. 6 shown is the variation of response ratio β_p of the pile foundation with respect to frequencies of input motion. The response ratio β_p is defined as

$$\beta_p = \frac{|a_{M1}|}{|a_{A1}|} \quad (2)$$

where $|a_{A1}|$ represents the amplitude of the acceleration at the base and $|a_{M1}|$ represents the amplitude of the acceleration at the bottom of the top mass on the pile foundation. The solid curve in Fig. 6 represents the response curve of the pile foundation model submerged in water and black circles denote the response ratio of the pile foundation model measured in liquefying sand layer at several moments of elapsed time. Numbers near black circles represent the elapsed time in seconds from the beginning of shaking. It is seen from this figure that response of the pile foundation model β_p once increased before $t=10$ seconds and then decreased abruptly to the smaller value of 0.2 at $t=12$ seconds. Also illustrated in Fig. 6 are the response ratio of sand layer $\beta_s = |a_s|/|a_{A1}|$, excessive porewater pressure ratio u/u_{max} and the amplitude of dynamic earthpressure acting on the pile foundation $|p_e|$. These values are illustrated in relation with β_p . It is clearly seen from this figure that at the moment of the maximum response of the pile foundation model the sand layer had not liquefied yet and the dynamic earth pressure had the maximum value and that after occurrence of liquefaction in the entire sand layer the response of pile foundation and the dynamic earth pressure also decreased to the small values.

Fig. 7 illustrates instantaneous positions of the sand layer and the pile foundation model at several instants of time ($t=8.0$ to 12 seconds). In this figure, four stages (a) through (d) in each column represent the variation of positions of the sand layer and the pile foundation model with a

short interval during the half of the vibration period ($\approx 1/24$ second). In each figure, hollow circles represent the value of $-\alpha/|a_H|$ in which α denotes the instantaneous value of acceleration recorded by the accelerometers A2 through A4 equipped in the sand layer, and $|a_H|$ represents the amplitude of the base acceleration. In an ideal sinusoidal movement, $-\alpha/|a_H|$ can be an index representing the displacement modes. And black circles represent those for the pile foundation model. Arrows represent the instantaneous directions of the movements at each point and the short vertical straight lines represent the maximum values of $-\alpha/|a_H|$ at each point during the half of the vibration period. Hollow triangles denote the instantaneous values of earthpressures acting on the pile. The positive sign corresponds to the increase of earthpressures sensed by the earthpressure cell which faced to the right direction in these figures. At the bottom of Fig. 7 shown are the excessive porepressure ratio u/u_{max} . It is seen from this figure that at the elapsed time of 8 seconds when excessive porewater pressures were very small, and movements of the sand layer and those of the pile foundation model are close to each other during the half of the vibration period. Then, as time goes on and excessive porepressures increase, the difference between the movement of the sand layer and that of the pile foundation model increases gradually. This would mean that the capability of the sand layer to support the pile foundation model decreases with the decrease in effective stress in sand layer which corresponds to the increase in excessive porewater pressure. As seen in Figs. 4 through 7, liquefied parts in the sand layer expand from the surface toward the bottom. Therefore, the free length of the pile foundation model which are not supported by the surrounding sand layer increased with the increase in excessive porepressure in the sand layer. This means that the natural frequency of the pile foundation in the sand layer, which is defined as f_p^* , decreased with the increase in the free length of the pile foundation.

Then, it would be beneficial to examine the response of the pile foundation model from the viewpoint of structural dynamics. First, suppose a single-degree-of-freedom vibrating system which corresponds to the pile foundation model in the sand layer. The maximum value of the response $\beta_p = 3.6$ of the pile foundation model which took place at the elapsed time of 10 seconds could be regarded as that of the system which resonates to the sinusoidal input motion with the frequency of $f_i = 12$ Hz. Then, the apparent damping ratio (h) of the system could be derived as

$$h = \frac{1}{2 \times 3.6} = 0.14. \quad (3)$$

In Fig. 8 shown are the response curve of the equivalent a single-degree-of-freedom vibrating system with the damping of 0.14 and the phase relation between the input force and the motion of the system. Using this response curve, the apparent natural frequency of the pile foundation model embedded in the sand layer f_p^* was obtained as follows. It could be supposed that the natural frequency of the pile foundation embedded in non-liquefied sand layer at the strain of around 10^{-4} is similar to the natural frequency of the sand layer, that is $f_g = 18$ Hz. Therefore, just after the beginning of shaking, the natural frequency of the pile foundation embedded in the sand layer f_p^* would be around 18 Hz. This means that $f_i/f_p^* = 12/18 < 1.0$. Therefore, the approximate value of

f_i/f_p^* at the elapsed time of $t = 8.0$ seconds can be obtained from the intersection point between $\beta_p = 2.1$, the value of β_p at $t = 8.0$ seconds, and the response curve for $f_i/f_p^* < 1.0$. By the same procedure, the values of f_i/f_p^* were obtained for the following elapsed time $t = 10.0, 10.8, 11.3, 12.2$ seconds as shown in Fig. 8. It is seen from this figure that the ratio of the frequency of input motion to the natural frequency of the pile foundation model embedded in the sand layer f_i/f_p^* increased with time and eventually approached to the value in the case of the pile foundation model submerged in water, $f_i/f_p = 12/3.8 = 3.2$. Since the value of f_i was kept constant as 12 Hz, the increase in f_i/f_p^* means the decrease in f_p^* . Also shown in Fig. 8 is the phase relation ϕ between the motion at the bottom of the top mass and the motion at the base. In this figure, the values of ϕ were obtained from Fig. 7, and the values of f_i/f_p^* were obtained from the procedure described above. The general coincidence of the measured phase relation with that of the single-degree-of-freedom vibrating system with $h = 0.14$ means that the variation of the response of the pile foundation β_p in liquefying sand layer was caused by the variation of the apparent natural frequency of the pile foundation model in the sand layer f_p^* .

As a next step, the effects of the response of the sand layer on the response of the pile foundation model should be taken into account, because the response of sand layer varied in the course of liquefaction as shown in Figs. 4 through 7. In the case of the pile foundation with a very small rigidity, the response of this pile foundation would be completely affected by the response of the sand layer before effective stress in the sand reduces to zero. On the other hand, this effect would be rather small in the case of the pile foundation with a relatively large rigidity. This would be the case of Test 1, because the maximum response ratio of the pile foundation β_p is about 3.6 at $t = 10$ seconds and at this moment the response ratio of the sand layer near the surface was about 1.9 which is less than $\beta_p = 3.6$. This means that the large response of the pile foundation model at $t = 10$ seconds was caused mainly by the resonance of the pile foundation model embedded in the sand as illustrated in Fig. 8.

A summary of the whole test results is shown in Fig. 9 where the relationship between the response ratio (β_p) of the pile foundation models and the ratio (f_i/f_p) of frequencies of input motions to the natural frequencies of the pile foundation models submerged in water. This figure indicates clearly that a comparative relation among the frequencies of input motions f_i , the natural frequencies of the pile foundation models submerged in water f_p , and the initial natural frequencies of the sand layer f_g is essential in determining the dynamic behavior of the pile foundation models in a liquefying sand layers. As all of the tests shown in Fig. 9 are the cases where f_g is larger than f_i or f_p , these tests can be divided into the following three groups.

(a) $f_g > f_i > f_p$ (Tests 1 to 3): In these tests, the response ratios of the pile foundation models β_p are small at the beginning of shaking as indicated by hollow squares in Fig. 9 and as the sand layer is going to liquefy, β_p begins to increase and then β_p reaches its maximum value as indicated by black triangles. At this moment, the sand layer has not liquefied completely. And following this moment, β_p

begins to decrease and eventually β_p becomes the small value as indicated by hollow circles in Fig. 9. These small values of β_p correspond well to those of the response ratio of the pile foundation models submerged in water. The phenomenon described above could take place for the prototype in the worst case. Therefore, the maximum value of β_p in the course of liquefaction in sand deposits should be estimated accurately in the case of the prototype where this phenomenon be taken into account in the seismic design of pile foundations.

(b) $f_g > f_1 = f_p$ (Test 4): In this test, β_p simply increases from the beginning of shaking to the state of the liquefaction in the entire sand layer. The eventual response ratio in the completely liquefied sand layer is as large as the value at resonance of the pile foundation model submerged in water. If there could be such a case where $f_1 = f_p$ in the prototype, severe damage could take place because the response gets quite large. However, another question whether or not major earthquake motion could continue until complete liquefaction takes place in rather deep sand deposits will be raised.

(c) $f_g > f_1 > f_p$ (Test 5): In this test, the tendency of the variation of β_p is similar to that of Test 4. However, differently from Test 4, the eventual value of β_p in the completely liquefied sand layer is rather small, because the response ratio of the pile foundation model submerged in water is small due to the fact that the value of f_1/f_p is much less than 1.0. In this case, the damage to the pile foundation would be minor. However, this case would be unlikely to exist in the prototype, because the value of f_p in this case would be unnaturally large.

CONCLUSIONS

The problem of dynamic behavior of pile foundations in liquefying soil deposits is considered to be one of the special cases of soil-structure interaction problems during earthquakes. As this problem is very complicated, some qualitative results can be derived from the above-mentioned tests.

From the tests described in the preceding sections, the following remarks can be made.

- 1) Since liquefaction phenomena progress with respect to time and with respect to space, the effects of liquefied sands on the piles vary as functions of time and space. It can be normally recognized that the response of a pile foundation with a normal rigidity is small before liquefaction initiates, increases abruptly large after liquefaction begins, and again decreases when complete liquefaction takes place. During the complete liquefaction soils behave as heavy liquid, and piles are subjected to hydrodynamic pressures due to the liquefied soils.
- 2) The relationship among the natural frequency of the pile foundation supported by the surrounding soil deposits before liquefaction initiates, the natural frequency of the pile foundation submerged in completely liquefied soils and the predominant frequency of input motion is the most important factor. Dynamic behavior of pile foundation in the course of liquefaction of sand layers will be considerably affected by this relationship.
- 3) While the complete liquefaction continues, a soil-pile foundation system behaves as a submerged structure.

Since its natural frequency gets lower, it may resonate when subjected to seismic motion with longer periods. Accordingly it will be important to assess the components of longer periods involved in induced seismic motions.

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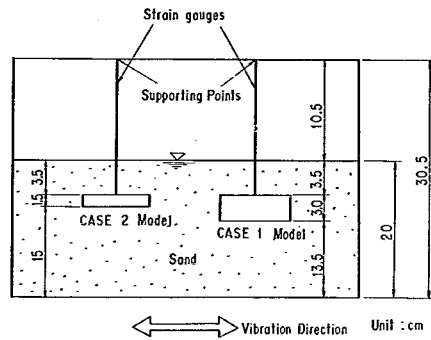


Fig. 1 Vibration Test for Buried Models

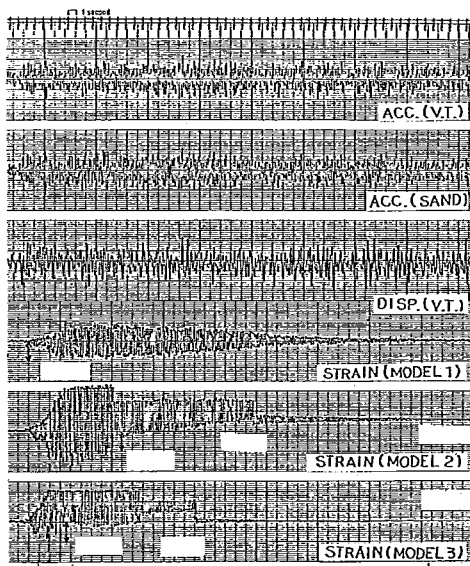


Fig. 2 Earthquake Force upon the Underground Body due to Surrounding Liquefied Sand

Table 1 List of Test Cases

TEST CASE	SAND	Dr ³⁾ (%)	Number ⁴⁾ of Piles	Head Weight (kg)	5) f_p (Hz)	6) f_i (Hz)	7) $ a_{A1} $ (gal)	Shaking Period (sec.)	8) f_g (Hz)
T1	Iruma ¹⁾	40	9	44	3.8	12	40 ~ 100	60	18
T2	2) Toyoura	27	4	53	4	10	60 ~ 80	30	24
T3		34	6	18	10	20	80 ~ 130	15	
T4		24	6	18	10	10	60 ~ 80	30	
T5		40	9	13	14	10	60 ~ 120	30	

- 1) Iruma Sand ($G_s=2.88$, $D_{50}=0.52\text{mm}$, $U_c=1.52$, $e_{\text{max}}=0.93$, $e_{\text{min}}=0.66$)
- 2) Toyoura Sand ($G_s=2.64$, $D_{50}=0.16\text{mm}$, $U_c=1.46$, $e_{\text{max}}=0.96$, $e_{\text{min}}=0.64$)
- 3) Relative Density
- 4) Each Pile is made of aluminum bars with 70cm in length and 2cm in diameter.
- 5) Natural Frequency of Pile Foundation in Water
- 6) Frequency of Input Motion
- 7) Amplitude of Base Acceleration before Perfect Liquefaction
- 8) Estimated Natural Frequency of Non-liquefied Sand Layer ($\gamma=10^{-4}$)

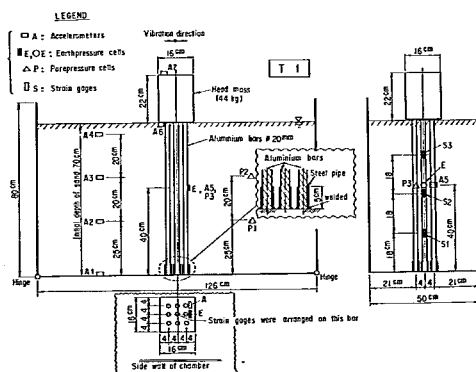


Fig. 3 Test Arrangements in the Case of Test 1

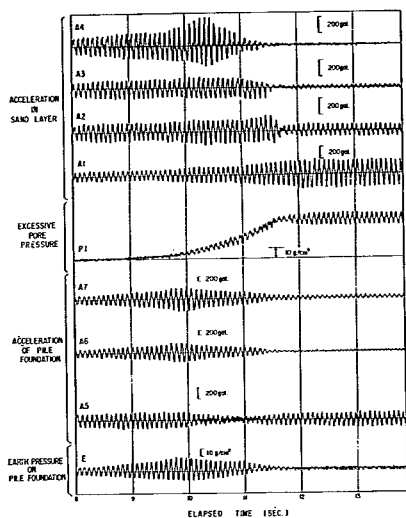


Fig. 4 Typical Test Records (Test 1)

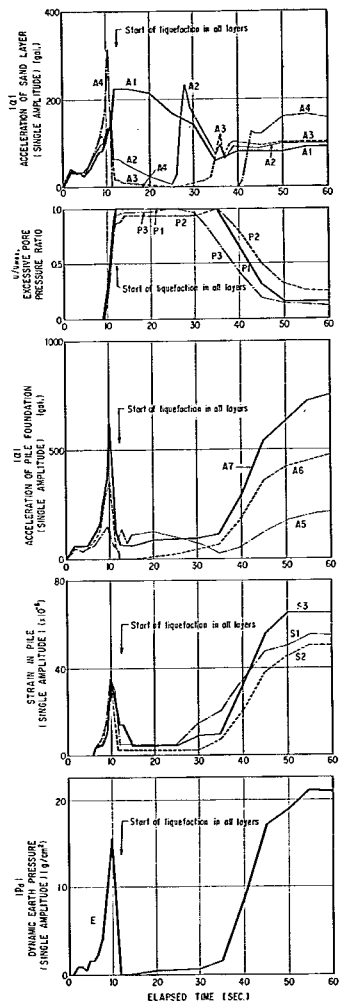


Fig. 5 Amplitudes of Dynamic Quantities Recorded (Test 1)

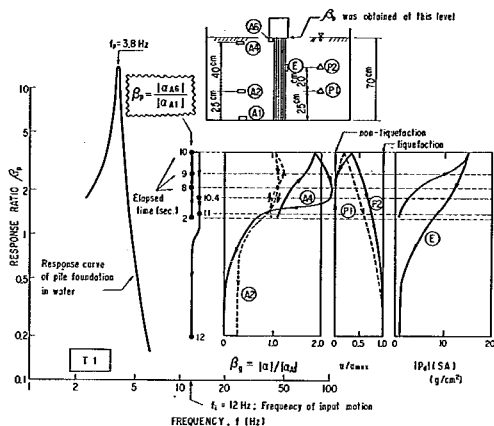


Fig. 6 Response of the Pile Foundation Model in Liquefying Sand Layer (Test 1)

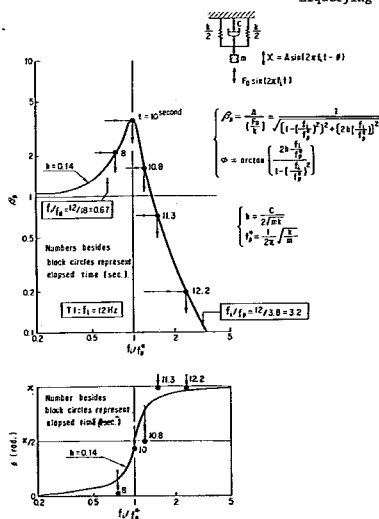


Fig. 8 Estimation of Apparent Natural Frequency of the Pile Foundation Model Embedded in Liquefying Sand Layer (Test 1)

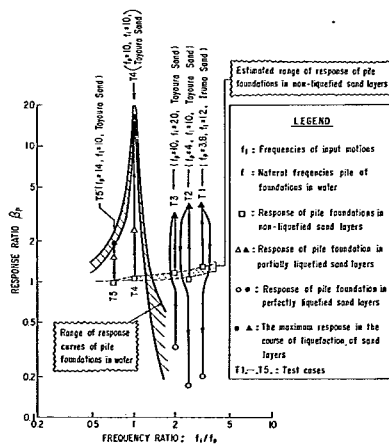


Fig. 9 Responses of Pile Foundation Models Embedded Liquefying Sand Layer (Tests 1 to 5)



— 138 —

Effects of Grain Size and Grading on Dynamic

Shear Moduli of Sands

by

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ABSTRACT

Resonant-column tests were performed intending to clarify the effects of grain size distribution on shear moduli of normally consolidated disturbed sands for shear strain ranging from 10^{-6} to 10^{-4} . The sands tested are clean sands which are uniform and do not contain fine particles, natural sands which are not uniform and contain fine particles and other artificially graded sands. The test results indicate the followings. Shear moduli of the clean sands tested at strain levels of $\gamma = 10^{-6}$, 10^{-5} and 10^{-4} are expressed approximately by the similar empirical equations irrespectively of grain shape and grain size. For the sands other than clean sands, however, shear moduli decrease with the increase in uniformity coefficient and also decrease with the increase in the content of fine particles. Therefore it is concluded that in estimating shear moduli of ordinary natural sands which are not so uniform and contain fine particles, it is preferable to conduct resonant-column tests on the sands of interest. When tests are not conducted, the effects of grain size distribution on shear moduli should be taken into account.

Key Words: Dynamic, Grain Size, Sand, Special Shear Test, Torsion, Vibration

IGC: D6/D7

INTRODUCTION

In analyzing dynamic behavior of soil deposits and soil-structure interactions during strong earthquake motions, it is essential to properly estimate strain-dependent shear moduli and damping properties of soils. It is widely known that shear moduli and damping properties of soils are strongly affected by the magnitude of shear strain amplitude, especially for the range greater than the strain of 10^{-4} . Insitu shear moduli for shear strain of around 10^{-6} can be obtained from shear wave velocities measured by seismic surveys, such as the down hole method. On the other hand, shear strains induced in surface deposits during strong earthquake motions are estimated to be around 10^{-4} to 10^{-2} . Therefore, it would be necessary to evaluate strain-dependent shear moduli for the estimation of shear moduli at γ of 10^{-4} to 10^{-2} . Shear modulus measurements for the wide range of strain can be performed by laboratory tests using several kinds of dynamic soil testing apparatus. Accordingly, to establish reliable relation between shear strains and shear moduli of soils, it may be necessary first to obtain shear moduli at γ of around 10^{-6} in the laboratory which can be compared with shear moduli from insitu seismic measurements under the similar conditions.

For determining shear moduli of soils under conditions similar to insitu seismic measurements, the resonant-column method has been available. The resonant-column method was initiated by K. Iida (1938) and major improvements were made by several investigators in the United States (Hardin and Richart (1963), Hardin and Music (1965) and Drnevich and Richart (1970)). A comprehensive series of resonant-column tests to obtain shear moduli at small strain amplitude for round and angular Ottawa

sands were performed using solid cylindrical samples by Hardin and Richart (1963). They established empirical equations for shear moduli G (kg/cm^2) at shear strain of $\gamma = 10^{-4}$ or less for two sands as

$$G = 700 \frac{(2.17 - e)^2}{1+e} p^{0.5} \quad (1)$$

for round Ottawa sands ($e < 0.80$) and as

$$G = 326 \frac{(2.97 - e)^2}{1+e} p^{0.5} \quad (2)$$

for angular Ottawa sands. In these equations e denotes void ratio and p means mean principal stress in kg/cm^2 . Equations (1) and (2) are based on tests of clean uniform Ottawa sands. On the other hand, usual natural sands are well graded and contain fine particles to some extent. Since it is supposed that shear moduli of such natural sands would be different from the shear moduli of clean sands, it would be necessary to perform resonant-column tests on other clean uniform sands, nonuniform sands and sands including fine particles, etc. to establish empirical equations for shear moduli of sands with a wide variety of grain size distributions.

Presented herein is the results of a laboratory testing program which was intended to clarify the effects of grain size distribution characteristics on shear moduli of a wide variety of normally consolidated disturbed sands at shear strain amplitude ranging from 10^{-6} to 10^{-4} using a resonant-column apparatus. It is concluded that shear moduli of sands are greatly affected by grain size distribution characteristics.

APPARATUS AND TESTING PROCEDURES

A resonant-column apparatus used in this study is of Drnevich-type (Drnevich and Richart (1970)) with a hollow cylindrical specimen of 25 cm in height, 10 cm in outside diameter and 6 cm in inside diameter. A sample is fixed at the bottom and sheared torsionally at the top. A pair of oscillators supplies the system with the torsional vibratory force. The total rotational inertia of the top mass is $0.1926 \text{ kg-cm-sec}^2$ and the spring constant is $1,000 \text{ kg-cm/radian}$ at the amplitude of 0.00043 radian and it varies slightly with the amplitude. This variation is evaluated in determining shear modulus of soil from resonant frequencies. The confining pressure equally applied to the outside and to the hollow of a specimen is supplied by air pressure and the additional axial load can be applied independently to the confining pressure. This arrangement permits anisotropic compressional stress conditions of $\sigma_a/\sigma_r > 1.0$ besides the isotropic stress condition of $\sigma_a/\sigma_r = 1.0$, where σ_a is the axial stress and σ_r is the confining pressure. Confining pressures adopted in this study took several values ranging from 0.20 kg/cm^2 up to 6.0 kg/cm^2 . All tests were conducted for normally consolidated samples. Resonant frequencies were measured at shear strain amplitude of 10^{-6} to 10^{-4} after an hour of consolidation at each confining pressure. After finishing measurements at a certain confining pressure, the confining pressure was increased to the next step.

Air-dry samples were made by pouring air-dry sands into a split mold. Saturated samples were prepared by spooning freshly boiled sands into a split mold filled with de-aired water. Densification of a sample was achieved by tapping the mold with a wooden hammer. Almost all of the tests were performed for air-dry specimens. Some tests, however, were performed

for saturated specimens with the drained condition.

Single amplitude shear strain γ developed in the specimen during a steady state vibration is defined as

$$\gamma = \frac{\bar{r}}{l} \theta_t \quad \text{(radian)} \quad (3)$$

where $\bar{r}$ is the average radius of the specimen, 4 cm in this case, l is the length of a specimen and θ_t is the torsional amplitude in radian at the top of the specimen. Details of the apparatus and the testing procedures are described in the previous paper (Kuribayashi, Iwasaki and Tatsuoka (1975)).

TEST MATERIALS

To evaluate the effects of grain size distribution characteristics on shear moduli of sands, various materials with a wide variety of grain size distributions were experimented. These materials are classified into the following three groups.

Group 1 (Clean sands): In this study, clean sands denote sands which have the uniformity coefficient $U_c = D_{60}/D_{10}$ less than 1.8 and do not contain grains finer than 0.074 mm in diameter. And fine soils denote soils which have diameters less than 0.074 mm. Clean sands tested are Toyoura Sand, two kinds of crushed sands and other sands with artificial gradings. The properties of these sands and the grain size distribution curves are shown in Fig. 1 and in Table 1. Among clean sands listed in Table 1, 30-50 Ottawa Sand was not tested in this study but data were quoted from Drnevich and Richart (1970). Clean sands with artificial gradings were made by sieving two kinds of natural sands, Sengenyama Sand and Iruma Sand. Maximum and minimum void ratios of sands tested were obtained by the Yoshimi-Tohno method (Yoshimi and Tohno (1972)) where maximum and minimum void ratios are measured with air-dry sands using a steel mold of 6 cm in inner diameter and 5.3 cm in inner height.

Group 2 (Natural sands which are well-graded and contain fine soils): Original natural sands tested have the uniformity coefficients larger than 1.8 and contain fine soils except Ban-nosu Sand A with U_c of 1.7. Properties of these soils are listed in Table 2 and the gradings were shown in Figs. 2, 3 and 4.

Group 3 (Artificially graded sands which are not clean): These sands

are further divided into three subgroups (1) Non-uniform sands with fine soils; Iruma Sands X1, X2 and X4 prepared from Iruma Sand, and Sengenyama Sand D prepared from Sengenyama Sand, (2) Non-uniform sands without fine soils, Iruma Sands S, X3, Y and W, Sengenyama Sand S, Tonegawa Sand S and Ohgishima Sand S. Iruma Sand W was made by removing fine soils from Iruma Sand X1. Among these sands, Sand S's are those sands which were made by removing fine soils from original natural sands, and (3) uniform sands with fine soils; Iruma Sand F and Iruma Sands Z1(1), Z1(2), Z1(4), Z1(8) and Z1(14) which were made by adding fine soils of 1%, 2%, 4%, 8% and 14%, respectively, to Iruma Sand Z1. Iruma Sand Z1 has the grading similar to that of Toyoura Sand. Properties of sands in Group 3 are listed in Table 3 and grain distribution curves are shown in Figs. 2 and 5.

Tests on anisotropic consolidated sands with $\sigma_a/\sigma_r = 2.0$ were conducted for Toyoura Sand. Tests on saturated samples were performed for Toyoura Sand, Ohgishima Sand and Iruma Sand. Other tests were performed on air-dry isotropically consolidated samples. Some sands were tested on densely and loosely packed samples, but others were tested only on densely packed samples.

Shear Moduli of Clean Sands

Shear moduli of typical clean sands at single-amplitude shear strains γ of 10^{-6} , 10^{-5} and 10^{-4} are illustrated in Figs. 6, 7 and 8, where p is the mean principal stress denoted by $p = (\sigma_a + 2\sigma_r)/3$ (σ_a is the axial stress and σ_r is the radial stress). OS, TO are the abbreviations of names of sands which are listed in Table 4. Shear moduli of 30-50 Ottawa Sand shown in these figures were quoted from the paper by Drnevich and Richart (1970). Ordinates in these figures are ratios of shear moduli to $\frac{(2.17 - e)^2}{1 + e}$, a function of void ratio e (see equation (1)). Furthermore, Figs. 9 and 10 show the relationship between shear moduli of clean sands at $p = 0.5 \text{ kg/cm}^2$ and 2.0 kg/cm^2 and void ratio e at $\gamma = 10^{-6}$ and 10^{-4} , respectively. Legends in these figures are indicated in Table 1. Note that the data in these figures include those of saturated Toyoura Sands. Figs. 6 through 11 indicate that while a slight scatter occurs among the data, all of the shear moduli of uniform clean sands without fine soils tested can be represented approximately by the following empirical equations for p ranging from 0.2 kg/cm^2 to 4 kg/cm^2 :

$$G = 900 \frac{(2.17 - e)^2}{1 + e} p^{0.40} \quad (\text{at shear strain of } \gamma = 10^{-6}) \quad (4)$$

$$G = 850 \frac{(2.17 - e)^2}{1 + e} p^{0.44} \quad (\text{at shear strain of } \gamma = 10^{-5}) \quad (5)$$

$$G = 700 \frac{(2.17 - e)^2}{1 + e} p^{0.50} \quad (\text{at shear strain of } \gamma = 10^{-4}) \quad (6)$$

In these equations, G is the shear modulus in kg/cm^2 , p is the mean

principal stress in kg/cm^2 and e is the void ratio.

The power of p in equation (4) was firstly determined as 0.38 in the previous papers (Kuribayashi, Iwasaki and Tatsuoka (1975) and Iwasaki and Tatsuoka (1977)). However, the additional test results performed thereafter showed that the power of 0.40 is better to express the whole data as shown in Fig. 6. Equation (6) coincides with equation (1) which was originally proposed by Hardin and Richart (1963) using solid cylindrical samples of round Ottawa Sands for the range of shear strain of 10^{-4} or less. On the other hand, test results shown in Figs. 6 through 10 were obtained from tests on hollow cylindrical samples where shear strains can be more uniform than tests using solid cylindrical samples. It is noted that shear moduli of 30-50 Ottawa Sand obtained by Drnevich and Richart (1970) on hollow cylindrical specimens can be expressed by equations (5) and (6) as shown in Figs. 7 and 8. Therefore, in the respect of the shear strain dependency of shear modulus, data from tests on hollow cylindrical samples can be considered to be more reliable than those from tests on solid cylindrical samples. Then, it is rational to determine different empirical equations of shear modulus for different shear strain amplitudes. Furthermore, it is found that the function of void ratio $\frac{(2.17 - e)^2}{1 + e}$ can be used for a wide variety of angular and subangular clean sands tested in this study as shown in Figs. 9 and 10.

Through examining carefully all the data, it was found that the following equation can be used to express shear moduli G of various clean sands:

$$G = A(\gamma) \cdot B \cdot \frac{(2.17 - e)^2}{1 + e} p^m(\gamma) \quad (7)$$

In equation (7) $A(\gamma)$ is a function of γ : A (at $\gamma = 10^{-6}$) = 900, A (at $\gamma = 10^{-5}$) = 850 and A (at $\gamma = 10^{-4}$) = 700, $m(\gamma)$ is also a function of

γ : m (at $\gamma = 10^{-6}$) = 0.40, m (at $\gamma = 10^{-5}$) = 0.44 and m (at $\gamma = 10^{-4}$) = 0.5, and B is a parameter dependent on the variety of a normally consolidated sand. The value of B can be determined irrespectively of γ , e and p . Shear moduli of a sand whose B -value is 1.0 are represented by equations (4), (5) and (6). In this study, B -value for each of the sands tested was determined using the following equation from the test results shown in Figs. 9 and 10:

$$B = \{B1 + B2 + B3 + B4\} / 4 \quad (8)$$

where

$$B1 = \frac{G \text{ from test results at } \gamma = 10^{-6} \text{ and } p = 0.5 \text{ kg/cm}^2}{G \text{ obtained from equation (4) for } p = 0.5 \text{ kg/cm}^2 \text{ and the void ratio of the sample tested}}$$

$$B2 = \frac{G \text{ from test results at } \gamma = 10^{-6} \text{ and at } p = 2.0 \text{ kg/cm}^2}{G \text{ obtained from equation (4) for } p = 2.0 \text{ kg/cm}^2 \text{ and the void ratio of the sample tested}}$$

$$B3 = \frac{G \text{ from test results at } \gamma = 10^{-4} \text{ and at } p = 0.5 \text{ kg/cm}^2}{G \text{ obtained from equation (6) for } p = 0.5 \text{ kg/cm}^2 \text{ and the void ratio of the sample tested}} \quad \text{and}$$

$$B4 = \frac{G \text{ from test results at } \gamma = 10^{-4} \text{ and at } p = 2.0 \text{ kg/cm}^2}{G \text{ obtained from equation (6) for } p = 2.0 \text{ kg/cm}^2 \text{ and the void ratio of the sample tested}}$$

It is found that the values of $B1$ through $B4$ for each sand are almost identical. Fig. 9 shows the relationship between B -value determined by equation (8) of all the clean sands tested and mean grain diameters D_{50} . It is seen from this figure that B -values of uniform clean sands tested are almost 1.0 for a wide range of grain size, although some scatterings are observed. This suggests that shear moduli of clean sands are not so related to the grain size in the range of D_{50} from 0.16 mm to 3.2 mm.

Shear Moduli of Sands other than Clean Sands

In Fig. 12 shown is the shear modulus at $\gamma = 10^{-4}$ for normally consolidated air-dry Tokyo-wan-o sand which is one of typical natural sands. This sand is well-graded ($U_c = 15.7$) and contains fine particles by 11.1%. The shear modulus of this sand is smaller than the value obtained from equation (6). This suggests that sands which are nonuniform and contain fine particles have smaller shear moduli than those of clean sands for the same values of p , e and γ . Shear moduli at $p = 0.5 \text{ kg/cm}^2$ and 2.0 kg/cm^2 of natural sands tested are shown in Fig. 13 for $\gamma = 10^{-6}$ and in Fig. 14 for $\gamma = 10^{-4}$ in the comparison with equation (4) or (6). It is noted that the data of tests on saturated specimens are involved in Figs. 13 and 14 and shear moduli of saturated specimens are almost identical to those of air-dry sands. These figures show clearly that shear moduli of original natural sands not being clean are, in general, less than those expressed by equations (4) and (6) which represent the average shear moduli of clean sands and that significant scatter occurs among the data of natural sands compared with the case of clean sands.

Shear moduli of normally consolidated artificially graded sands are shown in Figs. 15 and 16 in the same way as represented in Figs. 13 and 14. These results show that artificially graded sands which are not clean have shear moduli less than clean sands for the same values of p , e and γ and have a significant scatter. In this investigation, both of several clean sands and several sands which are not clean were prepared by sieving the several original sands. Note that while both were made from the same original sand, in any case, shear moduli of clean sands are larger than those of sands which are not clean. This is seen by comparing, for example,

shear modulus of Iruma Sand A which are shown in Figs. 9 and 10 with that of Iruma Sand XI which are shown in Figs. 15 and 16. From these it seems probable that the variation in shear moduli observed in this study is not caused by the characteristics of materials but by characteristics of grain size distribution.

Also for sands other than clean sands, B-value defined by equations (7) or (8) was adopted as a parameter representing the relative largeness of shear modulus, because it is found that shear moduli of sands other than clean sands are represented by equation (7), although B-value changes with varieties of sands. Furthermore, uniformity coefficient U_c and the content C (percent) of fine particles less than 0.074 mm of diameter are used in this study as indices of grain size characteristics. Fig. 17 shows the relationship between B-value and uniformity coefficient U_c for clean sands, original natural sands in Group 2 and other several sands in Group 3 except artificially graded sands prepared from Inuma Sand. It is seen from this figure that B-values for original natural sands and artificially graded sands which are not clean are, in general, less than 1.0. It is also noted that although a significant scatter occurs among the data, B-values will decrease with the increase in U_c . In Fig. 17, white marks with letter S represent data for Sand S's which were prepared from natural sands by removing fine particles. It is seen that these Sand S's have larger values of B than original sands although the change in U_c caused by removing fine particles is rather small. This suggests that shear moduli of sands artificially prepared by removing fine particles from original sands will become higher than those of original sands containing fine particles, when both sands have the same values of p , e and γ .

The scattering observed in Fig. 17 would be presumably caused by the

fact that these results were obtained from the tests on various sands. In order to avoid this, B-values were compared among the results for sands from one origin. Fig. 18 shows B-values for original Iruma Sand and for artificially graded sands prepared from original Iruma Sand. In this figure white marks denote the data of sands without fine particles and black marks denote the data of sands with fine particles. The letter C means the content of fine particles in percent. Fig. 18 indicates very clearly effects of grain size distribution on B-values. It is seen from the figure that for sand without fine particles the increase in U_c causes the decrease in B-value. Also this is the case for sands with fine particles. Furthermore, B-value will decrease with the increase in the content of fine particles. It is also noted that B-value of Iruma Sand W which was prepared from Iruma Sand X1 by removing fine particles is larger than that of Iruma Sand X1. The effects of containing fine particles on B-value is clearly shown in Fig. 19 where the relationship between B-values and the content of fine particles for Iruma Sands Z1, Z1(1), Z1(2), Z1(4), Z1(8) and Z1(14). It is shown from Fig. 19 that for the sands tested which have almost the same uniformity coefficients U_c except Iruma Sand Z1(14) B-value decreases significantly with the increase in the content of fine particles.

At two sites, one is an alluvial sand deposit and the other is a reclaimed sand deposit, shear moduli obtained by laboratory tests and those from field measurements were compared. The results of the comparison were reported elsewhere (Iwasaki and Tatsuoka, 1977). Laboratory resonant-column tests were carried out on normally consolidated disturbed air-dry and saturated samples which were sampled at the points where field seismic measurements by the down-hole method were conducted. It is revealed from

the comparison that shear moduli calculated by substituting estimated values of void ratio e and mean principal stress p into equation (3) are larger than shear moduli obtained from shear wave velocities measured. It is also seen that shear moduli obtained by substituting estimated values of e and p into the empirical equations for $\gamma = 10^{-6}$ in the form of equation (6) which were established on the test results on sands sampled at the points of seismic measurements were almost identical with shear moduli obtained from shear wave velocity measurements.

Both of test results presented in this paper and the comparison between the laboratory test results and the field measurements indicate that it is misleading to estimate shear moduli of sand deposits without taking into account the effects of grain size distribution on shear moduli.

The Relationship between Shear Moduli and Relative Densities

It is reported that shear moduli of sands can be estimated from the values of relative densities, shear strains and confining pressures (See and Idriss 1970). Test results shown in this paper, however, indicate that this is not always the case in estimating shear moduli of various sands. Firstly, approximate values of shear moduli of clean sands tested at shear strains of 10^{-6} , 10^{-5} and 10^{-4} can be evaluated by equations (4), (5) and (6) in which shear moduli are not expressed by a function of relative density but by a function of void ratio. For example, let us consider two kinds of clean sands. The first has shear moduli exactly expressed by equations (4), (5) and (6) and the minimum void ratio of 0.5. The second has the similar shear moduli and the minimum void ratio of 0.8. Then, when shear strain is 10^{-4} and p is 1.0 kg/cm^2 , the shear modulus of

the first sand is $1,301 \text{ kg/cm}^2$, while the shear modulus of the second sand is 730 kg/cm^2 , with these two values being obtained from equation (6). Although the relative densities of the two sands are both 100%, their shear moduli are considerably different. Furthermore, it is found from the comparison of shear moduli for all the sands tested which have nearly the same relative density that the variation of shear moduli for various sands is not small. Therefore, it could be concluded that it is not appropriate to evaluate shear moduli of various kinds of sands as a function of relative density.

Relationship between B-value and Damping

In this laboratory test program, damping properties of materials were also evaluated. Decay curves were obtained during the first five cycles of free vibration after turning off the input power at the resonant vibrations. In Fig. 20 shown is the relationship between the values of B and damping properties at $\gamma = 10^{-4}$ and $p = 1.0 \text{ kg/cm}^2$ of the sands tested. In this case, shear strain amplitude γ is the value at the resonating steady state vibration and the values of damping are denoted by $\eta = \frac{1}{2\pi} \Delta_t$ in which Δ_t is logarithmic decrement. It is seen from Fig. 20 that the values of damping seem to increase slightly with the decrease in the value of B. Although significant scatter can be pointed out among these data, it is likely that the effects of grain size distribution on damping properties are not so great as the effects on shear moduli.

CONCLUSIONS

From the results of the comprehensive laboratory tests, it is concluded that shear moduli of normally consolidated disturbed sands are greatly affected by grain size distribution characteristics. Furthermore, the following would be pointed out.

- 1) Shear moduli of clean sands at strain levels of $\gamma = 10^{-6}$, 10^{-5} and 10^{-4} are expressed approximately by equations (4), (5) and (6), respectively.
- 2) In general, shear moduli of sands decrease with the increase in uniformity coefficient and also decrease with the increase in the content of fine soils for the same values of p , e and γ .
- 3) In estimating shear moduli of ordinary natural sands which are not so uniform and contain fine soils, it is preferable to conduct resonant-column tests on the sand of interest. When tests are not conducted, the effects of grain size distribution characteristics on shear moduli should be taken into account.
- 4) The effects of grain size distribution on damping properties of the sands tested are not so great as the effects on shear moduli.

ACKNOWLEDGMENTS

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NOTATION

B = a parameter representing the relative largeness in shear moduli

C = the content of fine soils

D_r = relative density

e = void ratio

G = shear modulus

p = mean principal stress = $\frac{1}{3} (\sigma_1 + 2\sigma_3)$

U_c = uniformity coefficient

Δt = logarithmic decrement

γ = shear strain in single amplitude

η = damping

σ₁, σ₃ = major and minor principal stresses

σ_a, σ_r = axial and radial stresses (σ_a=σ₁, σ_r=σ₃)

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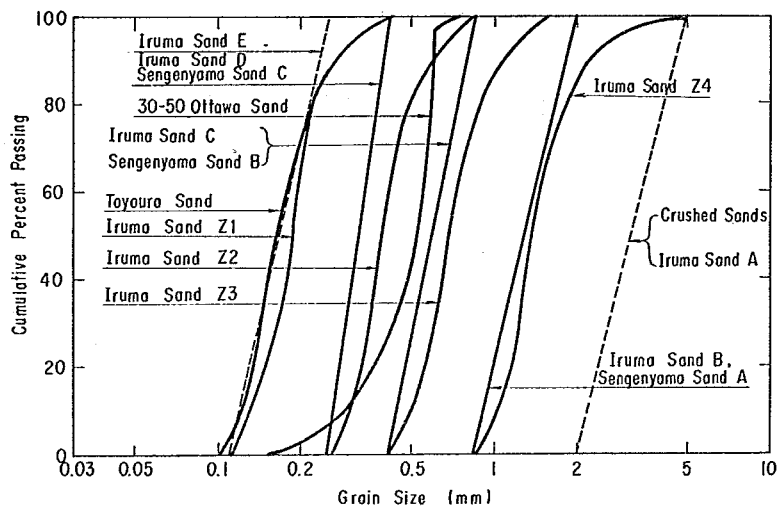


Fig. 1 Grain Size Distribution Curves of Sands in Group 1

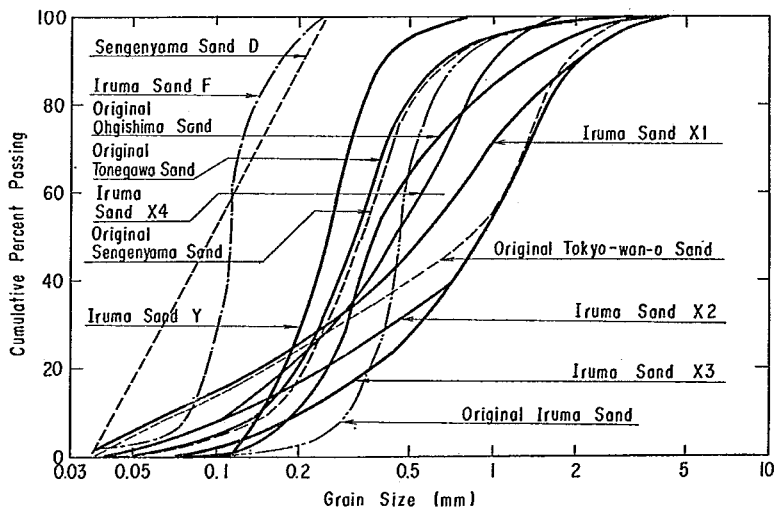


Fig. 2 Grain Size Distribution Curves of a Part of Sands in Groups 2 and 3

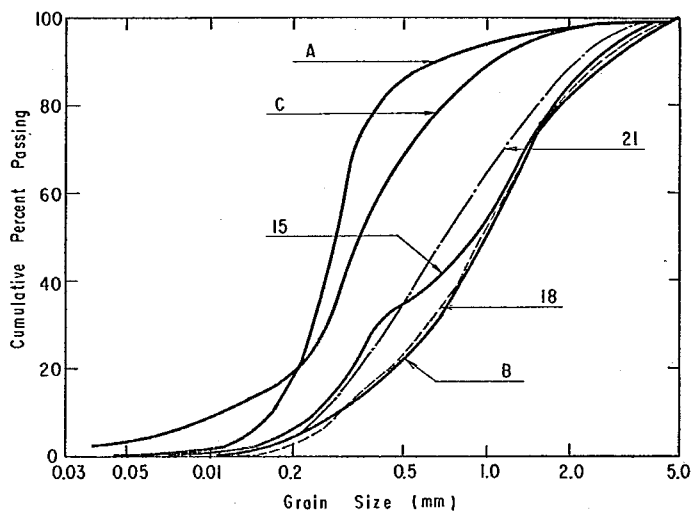


Fig. 3 Grain Size Distribution Curves of Ban-nosu Sands (Group 2)

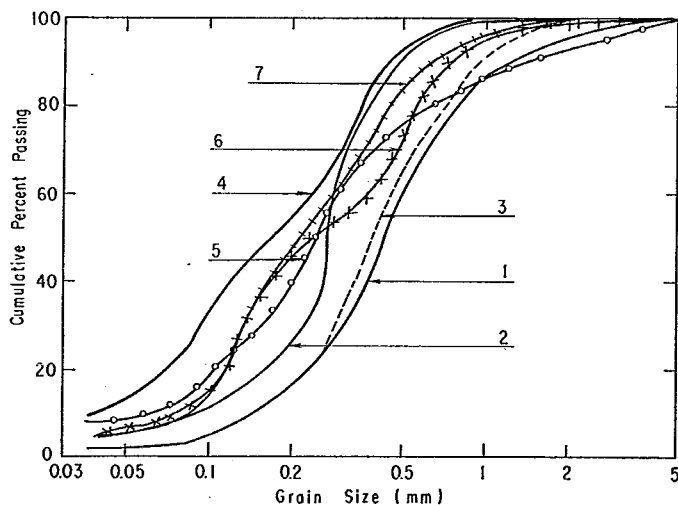


Fig. 4 Grain Size Distribution Curves of Kinjo Sands (Group 2)

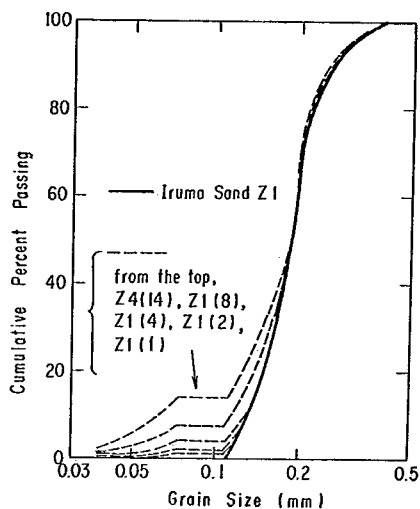


Fig. 5 Grain Size Distribution Curves of Iruma Sands Z1(1) through Z1(14)
(Group 3)

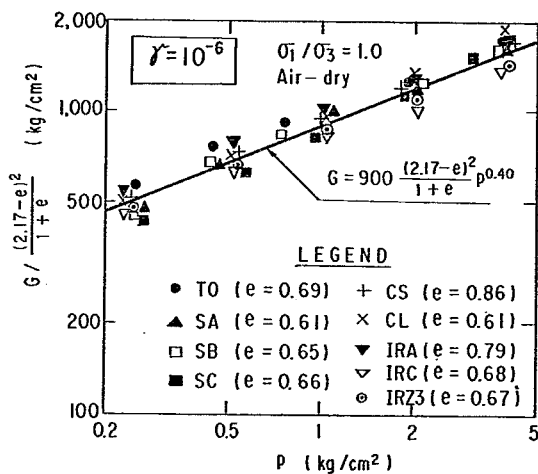


Fig. 6 Shear Moduli at $\gamma = 10^{-6}$ versus p for Clean Sands
(See Table 5 for Legend)

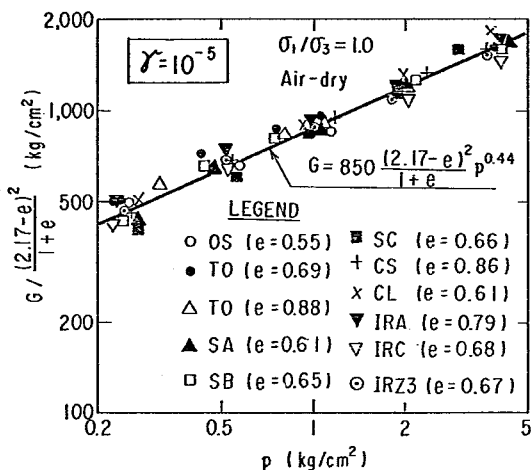


Fig. 7 Shear Moduli at $\gamma = 10^{-5}$ versus p for Clean Sands
(See Table 5 for Legend)

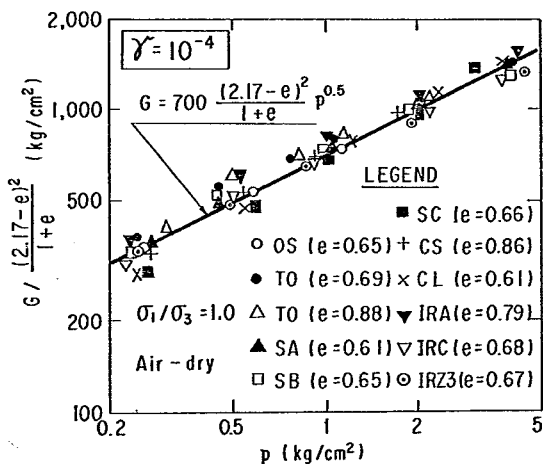


Fig. 8 Shear Moduli at $\gamma = 10^{-4}$ versus p for Clean Sands
(See Table 5 for Legend)

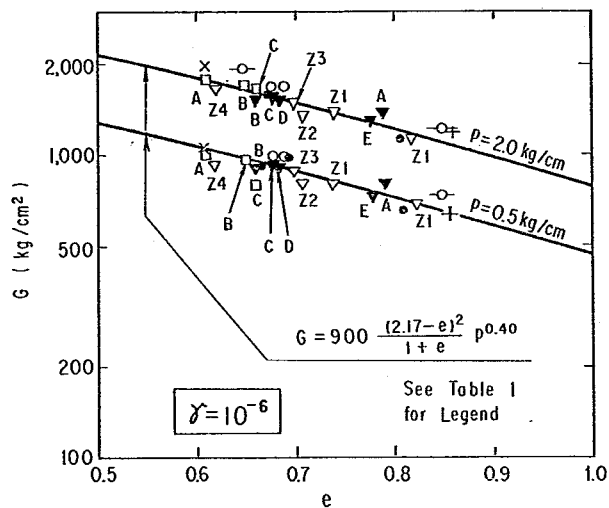


Fig. 9 Shear Moduli at $\gamma = 10^{-6}$ versus e for Clean Sands

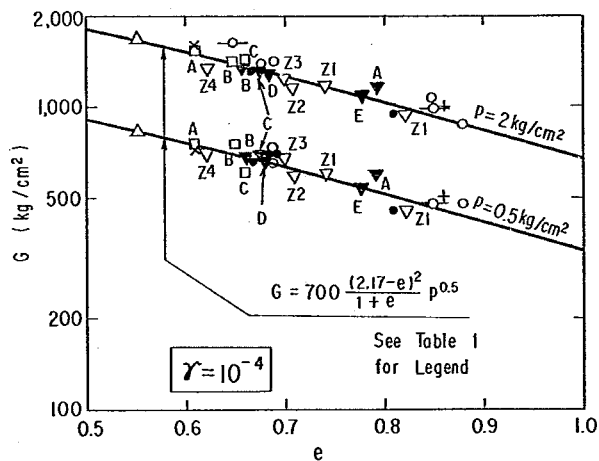


Fig. 10 Shear Moduli at $\gamma = 10^{-4}$ versus e for Clean Sands

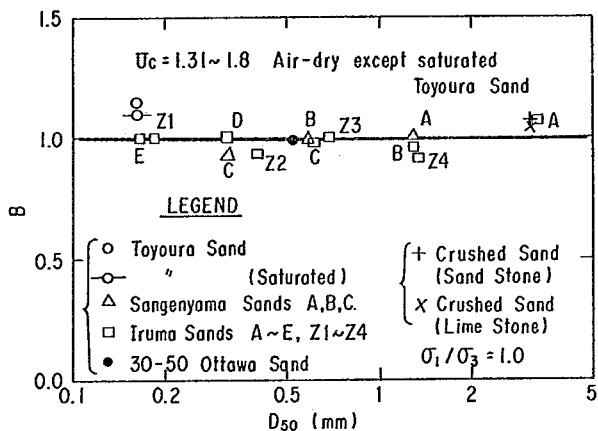


Fig. 11 Relationship between B-value and D_{50} for clean sands

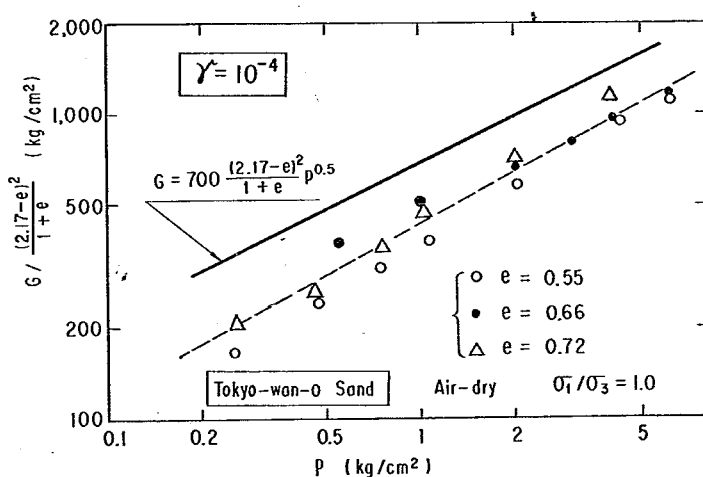


Fig. 12 Shear Modulus at $\gamma = 10^{-4}$ of Tokyo-wan-o Sand

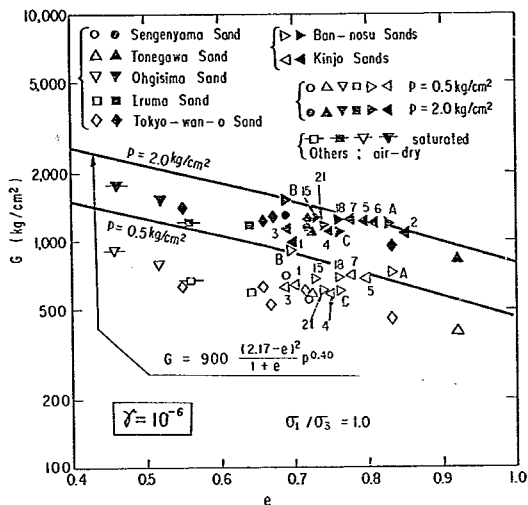


Fig. 13 Shear Moduli at $\gamma = 10^{-6}$ versus e for Original National Sands

(Group 2)

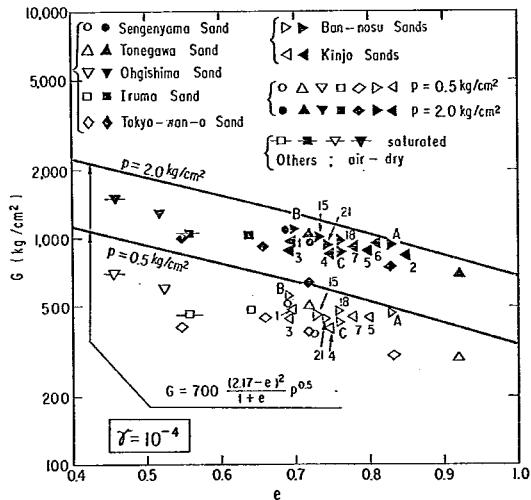


Fig. 14 Shear Moduli at $\gamma = 10^{-4}$ versus e for Original Natural Sands

(Group 2)

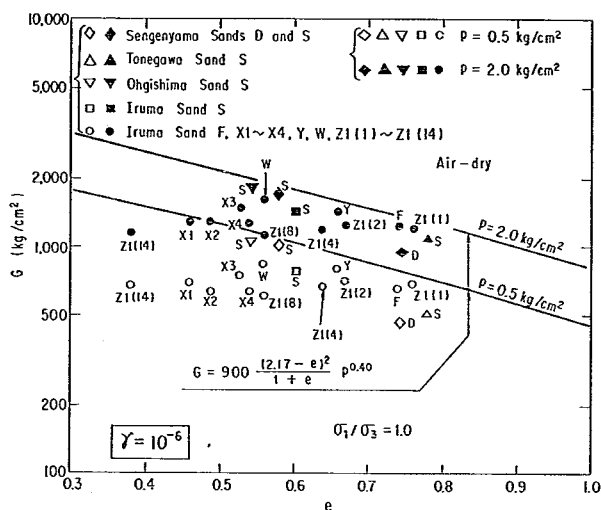


Fig. 15 Shear Moduli at $\gamma = 10^{-6}$ versus e for Sands in Group 3

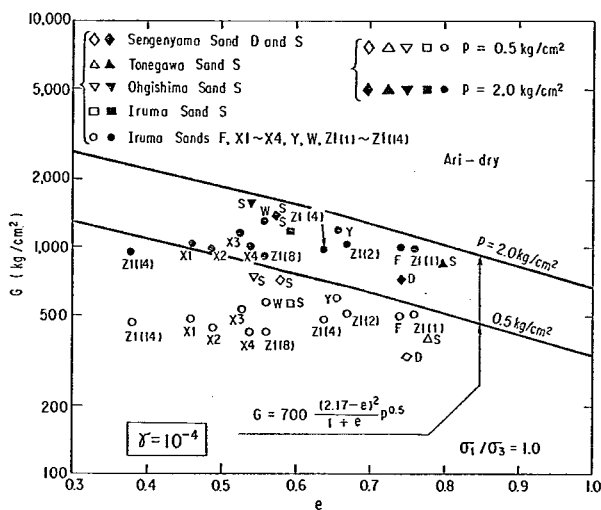


Fig. 16 Shear Moduli at $\gamma = 10^{-4}$ versus e for Sands in Group 3

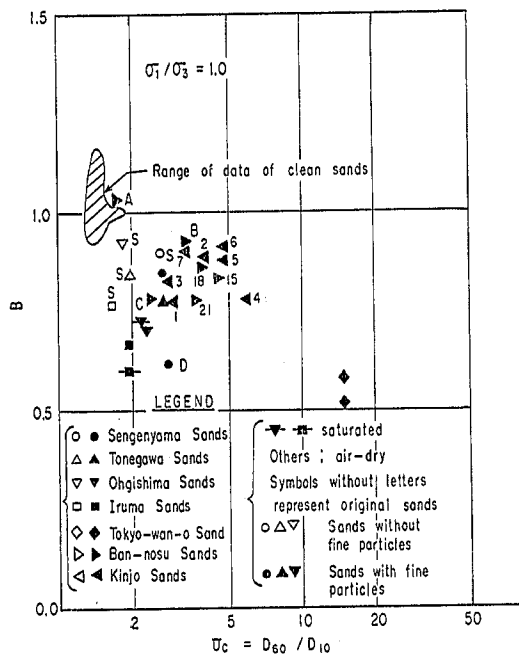


Fig. 17 Relationship between B-value
and U_c for the Sands Tested

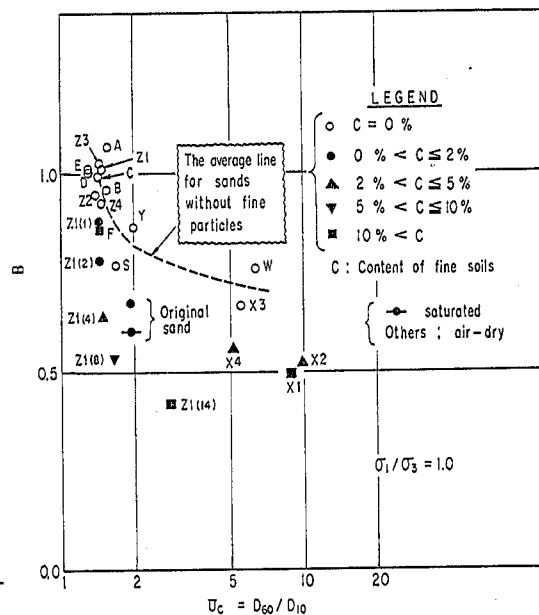


Fig. 18 Relationship between B-value
and U_c for Iruma Sands

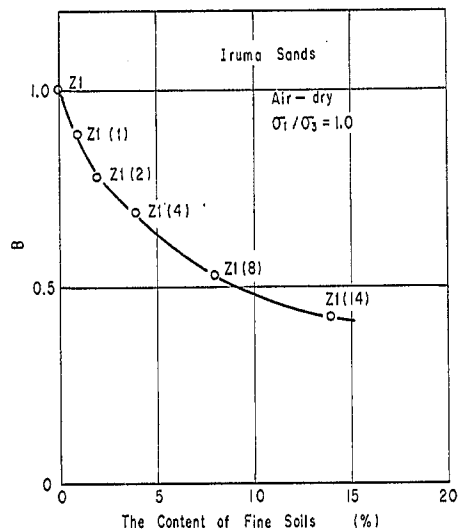


Fig. 19 Relationship between B-value and the Content of Fine Particles

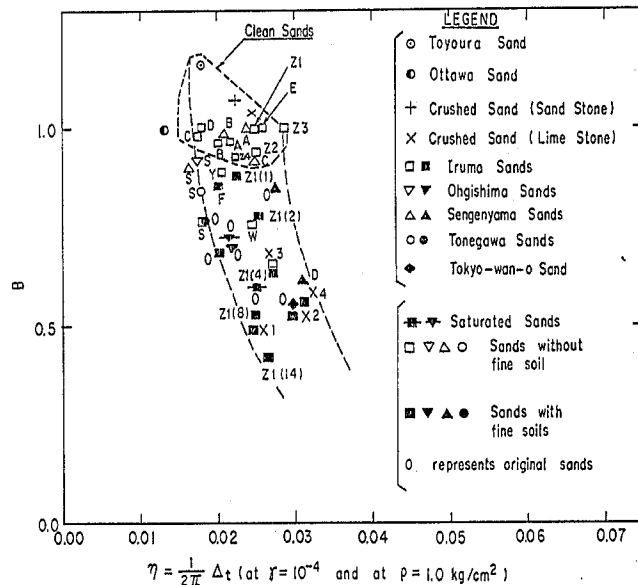


Fig. 20 Relationship between B-value and the values of Damping of the Sands Tested

Table 1 Physical Properties of Materials in Group 1

Material	Grain Shape	G _s	D ₁₀ (mm)	D ₅₀ (mm)	D ₉₀ (mm)	U _c	e _{max} ^{a)}	e _{min} ^{a)}	<75μ (%)	<37μ (%)	Symbols in Figs. 10 and 11 b) ○ σ ₁ /σ ₃ =1.0 air dry ○ σ ₁ /σ ₃ =2.0 " ● σ ₁ /σ ₃ =1.0 saturated
Toyoura Sand	subangular	2.64	0.12	0.162	0.175	1.46	0.96	0.64	0.0	0.0	
Crushed sand of sand stone	angular	2.73	2.2	3.2	3.5	1.59	1.09	0.80	0.0	0.0	+
Crushed sand of ilse stone	subangular	2.74	2.2	3.2	3.5	1.59	0.93	0.64	0.0	0.0	X
Sengeniyama Sand A	"	2.65	0.90	1.3	1.4	1.56	0.82	0.61	0.0	0.0	□ A
Sengeniyama Sand B	"	2.72	0.44	0.59	0.64	1.45	0.92	0.67	0.0	0.0	□ B
Sengeniyama Sand C	"	2.68	0.26	0.32	0.34	1.31	0.98	0.65	0.0	0.0	□ C
Iruma Sand A	angular	2.67	2.2	3.2	3.5	1.59	1.06	0.79	0.0	0.0	▼ A
Iruma Sand B	"	2.68	0.90	1.3	1.4	1.56	0.92	0.66	0.0	0.0	▼ B
Iruma Sand C	"	2.83	0.44	0.59	0.64	1.45	0.98	0.66	0.0	0.0	▼ C
Iruma Sand D	"	2.93	0.26	0.32	0.34	1.31	0.99	0.68	0.0	0.0	▼ D
Iruma Sand E	"	3.29	0.12	0.165	0.182	1.52	1.06	0.76	0.0	0.0	▼ E
Iruma Sand Z1	"	3.22	0.13	0.184	0.192	1.48	1.02	0.69	0.0	0.0	▽ Z1
Iruma Sand Z2	"	2.89	0.30	0.40	0.42	1.40	0.96	0.68	0.0	0.0	▽ Z2
Iruma Sand Z3	"	2.79	0.50	0.69	0.74	1.48	0.94	0.68	0.0	0.0	▽ Z3
Iruma Sand Z4	"	2.68	1.02	1.37	1.48	1.45	0.88	0.62	0.0	0.0	▽ Z4
30-50 Ottawa Sand ^{c)}	round	2.66	0.30	0.52	0.54	1.80	0.80	0.48	0.0	0.0	△

a) Obtained with air-dry sands by the Yoshimi-Tohno method.

b) Tests were performed on the condition of $\sigma_1/\sigma_3 = 1.0$ on air-dry samples except Toyoura Sand.

c) After Drnevich and Richart (1970).

Table 2 Physical Properties of Materials in Group 2

Material	Grain Shape	G _s	D ₁₀ (mm)	D ₅₀ (mm)	D ₆₀ (mm)	U _c	e _{max} ^{a)}	e _{min} ^{a)}	<74 μ (%)	<37 μ (%)	Symbols in Figs. 14 and 15	b)
Sengeniyama Sand	subangular	2.70	0.16	0.33	0.38	2.73	0.91	0.59	3.3	0.3	O ●	
Tonegawa Sand	"	2.71	0.135	0.325	0.37	2.74	1.00	0.65	3.5	0.7	△ ▲	
Ogishima Sand	"	2.69	0.20	0.37	0.45	2.25	0.72	0.49	0.36	0.2	▽ ▼ air-dry ◇ ▼ saturated	
Iruma Sand	angular	2.75	0.28	0.50	0.55	1.98	0.84	0.57	0.6	0.1	□ ■ air-dry ▢ ■ saturated	
Tokyo-wan-o Sand	subangular	2.74	0.07	0.80	1.1	15.7	0.96	0.43	11.1	0.2	◇ ◆	
Ban-nosu Sand A	"	2.67	0.168	0.26	0.285	1.70	1.17	0.82	1.1	0.5	▷ A ▶ A	
Ban-nosu Sand B	"	2.66	0.31	0.92	1.04	3.35	1.01	0.68	0.5	0.4	▷ B ▶ B	
Ban-nosu Sand C	"	2.68	0.15	0.34	0.41	2.73	1.03	0.69	5.9	2.5	▷ C ▶ C	
Ban-nosu Sand 15	"	2.67	0.24	0.88	1.10	4.58	0.91	0.69	0.26	0.1	▷ 15 ▶ 15	
Ban-nosu Sand 18	"	2.67	0.31	0.99	1.20	5.87	0.94	0.68	0.5	0.2	▷ 18 ▶ 18	
Ban-nosu Sand 21	"	2.67	0.25	0.72	0.90	3.60	0.92	0.67	0.8	0.2	▷ 21 ▶ 21	
Kinjo Sand 1	"	2.66	0.16	0.42	0.46	2.88	1.11	0.71	3.0	2.2	◁ 1 ◀ 1	
Kinjo Sand 2	"	2.67	0.09	0.32	0.35	3.89	1.42	0.92	8.8	5.1	◁ 2 ◀ 2	
Kinjo Sand 3	"	2.62	0.16	0.39	0.46	2.88	1.08	0.68	3.1	2.2	◁ 3 ◀ 3	
Kinjo Sand 4	"	2.64	0.039	0.17	0.24	6.02	1.43	0.85	23.8	9.0	◁ 4 ◀ 4	
Kinjo Sand 5	"	2.66	0.064	0.24	0.31	4.84	1.38	0.84	12.5	8.2	◁ 5 ◀ 5	
Kinjo Sand 6	"	2.68	0.079	0.25	0.40	5.06	1.37	0.86	11.2	5.3	◁ 6 ◀ 6	
Kinjo Sand 7	"	2.66	0.08	0.23	0.27	3.38	1.37	0.85	8.8	5.4	◁ 7 ◀ 7	

a) Obtained with air-dry sands by the Yoshimi-Tokno method.

b) Tests were performed on the condition of $\sigma_1/\sigma_2 = 1.0$ on only air-dry samples except Ogishima Sand and Iruma Sand.

Table 3 Physical Properties of Materials Tested in Group 3 a)

Material	Grain Shape	C_s	D_{10} (mm)	D_{50} (mm)	D_{90} (mm)	U_c	e_{max} b)	e_{min} b)	<74 μ (%)	<37 μ (%)	Symbols in Figs. 16 and 17
Sengenyama Sand S ^{c)}	subangular	2.70	0.153	0.34	0.41	2.68	0.88	0.60	0.0	0.0	◇ S ◆ S
Sengenyama Sand D	"	2.70	0.05	0.103	0.124	24.8	1.22	0.81	35.0	3.8	◇ D ◆ D
Tonegawa Sand S ^{c)}	"	2.71	0.17	0.31	0.345	2.03	0.98	0.70	0.0	0.0	△ S ▲ S
Ohgishima Sand S ^{c)}	"	2.69	0.21	0.35	0.39	1.85	0.81	0.55	0.0	0.0	▽ S ▼ S
Iruma Sand S ^{c)}	angular	2.75	0.31	0.48	0.518	1.67	0.85	0.58	0.0	0.0	□ S ■ S
Iruma Sand F	"	3.26	0.083	0.114	0.118	1.42	1.05	0.71	12.0	2.0	○ F ● F
Iruma Sand W ^{d)}	"	2.91	0.13	0.63	0.80	6.15	0.87	0.56	0.0	0.0	○ W ● W
Iruma Sand X1	"	2.91	0.08	0.54	0.72	9.0	0.70	0.46	11.1	1.8	○ X1 ● X1
Iruma Sand X2	"	2.81	0.11	0.92	1.1	10.0	0.65	0.48	5.0	0.8	○ X2 ● X2
Iruma Sand X3	"	2.80	0.20	0.92	1.1	5.5	0.69	0.52	0.0	0.0	○ X3 ● X3
Iruma Sand X4	"	2.91	0.11	0.45	0.56	5.09	0.78	0.51	5.0	0.8	○ X4 ● X4
Iruma Sand Y	"	3.08	0.145	0.26	0.285	1.97	0.92	0.64	0.0	0.0	○ Y ● Y
Iruma Sand Z1(1)	"	3.21	0.136	0.185	0.195	1.43	1.00	0.70	1.0	0.2	○ Z1(1) ● Z1(1)
Iruma Sand Z1(2)	"	3.18	0.135	0.185	0.195	1.44	0.98	0.69	2.0	0.3	○ Z1(2) ● Z1(2)
Iruma Sand Z1(4)	"	3.12	0.132	0.185	0.195	1.48	0.92	0.63	4.0	0.7	○ Z1(4) ● Z1(4)
Iruma Sand Z1(8)	"	2.98	0.115	0.185	0.195	1.70	0.83	0.47	8.0	1.3	○ Z1(8) ● Z1(8)
Iruma Sand Z1(14)	"	2.77	0.068	0.185	0.195	2.87	0.71	0.38	14.0	2.3	○ Z1(14) ● Z1(14)

a) All tests were performed on isotropically consolidated air-dry samples.

b) Obtained with air-dry sands by the Yoshimi-Tohno method.

c) Obtained by removing fine soils from original sands.

d) Obtained by removing fine soils from Iruma Sand X1.

Table 4 Legend in Figs. 7, 8 and 9

Symbol	Material
○ OS	30-50 Ottawa Sand
● TO	Toyoura Sand
△ TO	"
▲ SA	Sengenyama Sand A
□ SB	Sengenyama Sand B
■ SC	Sengenyama Sand C
+ CS	Crushed Sand (Sand Stone)
× CL	Crushed Sand (Lime Stone)
▼ IRA	Iruma Sand A
▽ IRC	Iruma Sand C
● IRZ3	Iruma Sand Z3

Shear Moduli of Sands under
Cyclic Torsional Shear Loading

by

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ABSTRACT

To evaluate degree of reduction in shear moduli of sands with an increase in shear strain amplitude, dynamic soil tests were performed with use of hollow cylindrical samples applying torsional shearing forces. Two types of soil testing equipments, a resonant-column apparatus and a torsional shear apparatus, were employed. It is pointed out that shear moduli at shear strain amplitude of 10^{-4} obtained from these two different equipments agree satisfactorily. Furthermore, extents of reduction in shear moduli with an increase in shear strain amplitude are shown for various sands. These data are compared with those presented by other investigators. On the basis of test results, a simplified procedure for predicting reduction in shear modulus with an increase in shear strain amplitude is proposed.

Key Word: Dynamic, Sands, Special Shear Test, Torsion, Vibration

IGC; D6/D7

INTRODUCTION

It is essential to properly estimate strain-dependent Shear moduli and damping properties of soils in analyzing soil-structure interactions and earthquake responses of grounds and soils structures. Shear moduli and damping properties of soils at a shear strain level to be induced in grounds during strong earthquakes may be evaluated as follows. First, shear moduli at shear strain γ of around 10^{-6} are obtained from in situ seismic surveys. Then, shear strains which are induced in grounds during strong earthquakes are estimated by computation and found to be around 10^{-3} or around 10^{-2} at the largest, which are much larger than 10^{-6} . Reduction in shear moduli with increasing shear strain can be obtained by laboratory soil testings. Internal dampings of soils can also be obtained by laboratory testings. To determine a reliable relationship between shear moduli and shear strains, it firstly becomes necessary to know shear moduli at shear strain of around 10^{-6} by laboratory soil testings. For this purpose, the resonant-column method is one of the best laboratory soil testing procedures. For alluvial-origin sands, Stokoe and Richart (1973), Cuny and Fry (1973) Iwasaki and Tatsuoka (1977) and Richart, Anderson and Stokoe (1977) have found good agreement between resonant-column results and in situ seismic survey values. This is due to the fact that the secondary time effect is negligible for sands. At the shear strain level larger than 10^{-4} , sands behave as non-elastic materials and have rather large dampings. For this range of strain, the resonant-column method is not a proper one in evaluating soil properties. To obtain shear moduli and dampings of soils for the shear strain level larger than 10^{-4} , several devices have been developed. Typical examples are dynamic triaxial test devices (Park and Silver (1975)), box type simple shear devices (Kovacs, Seed and Chan (1971), Sugimoto, Suzuki, Terada and Watanabe (1974)), NGI type simple shear devices (Theirs and Seed (1968), Silver and Seed (1971), Hara and Kiyota (1976)), hollow cylinder static torsional shear devices (Hardin and Drnerich (1972a), (1972b), Sherif and Ishibashi (1976)) and a ring torsion shear apparatus (Yoshimi and Oh-Oka (1973)).

In the present study, two devices, a resonant-column apparatus

applicable for the strain of $\gamma = 10^{-6}$ to 10^{-4} and a torsional shear device for the strain of $\gamma = 10^{-4}$ to 10^{-2} , were employed. For the two types of testings, hollow cylindrical samples with identical cross-sections were used, because nearly uniform shear strain are induced throughout hollow cylindrical specimens. Furthermore, the identical consolidation stress condition was adopted for the both tests.

Presented herein are shear moduli of various kinds of sands for shear strain of 10^{-6} to 10^{-2} which were obtained by utilizing a Drnevich-type resonant-column apparatus and a torsional shear apparatus. Sands tested involved clean ones and other natural sands which include fine soils to some extent. Consecutive values of shear moduli for the strain range of $\gamma = 10^{-6}$ to 10^{-2} were obtained from the two testing procedures. Test results are compared with those by other investigators. A simplified relationship between shear moduli and shear strains is proposed to provide a design engineer with a convenient tool to estimate shear moduli of sands subjected to large strains.

APPARATUS

Resonant-Column Apparatus

A hollow cylindrical specimen in resonant-column tests is 25 cm in height, 10 cm in outside diameter and 6 cm in inside diameter. A sample is fixed at the bottom and torsional forces are applied at the top. The total rotational inertia of the top mass is 0.1926 kg-cm-sec². The spring constant is 1,000 kg-cm/radian at the amplitude of 0.00043 radians and the spring constant varies slightly with the amplitude. Isotropic confining stress condition is adopted in this study. The consolidation time for each confining pressure is taken about 2 hours. Loosely packed samples were made by different two methods; the spooning method and the raining method. In the spooning method, air-dry sands were poured with a spoon into a mold and a loosely packed saturated sample was made by pouring de-aired saturated sands with a spoon into a mold which was full-filled with de-aired water in advance. In the raining method, air-dry sands were poured into a mold through a small hole with the diameter of 3 mm from a constant height of 10 cm above the sand surface. Densification of samples made by the spooning method was achieved by tapping the mold with a wooden hammer (the spooning and tapping method). In each test, fresh sands were used.

By using the resonant-column apparatus, shear moduli and dampings of sands for the shear strain ranging from about 10^{-6} to about 10^{-4} were measured. Details of the apparatus and the testing procedures are described in the previous papers (Kuribayashi, Iwasaki and Tatsuoka (1975), and also Iwasaki and Tatsuoka (1977)).

Torsional Shear Device

For obtaining soil properties for strain range of $\gamma = 10^{-4}$ to 10^{-2} , a torsional shear device was developed. In the device torsional loads and torsional displacements can be measured directly. Fig. 1 shows the schematic drawing of the apparatus. Fig. 2 shows a close view of the triaxial cell of the torsional shear device without a lucid cylinder. Soil samples with dimensions of 6 cm in inside diameter, 10 cm in outside

diameter and 10 cm in height are tested. The samples are fixed at bottom and torsional loads are applied at the top of the samples. Equal lateral confining pressures are applied to both the outside and the inside of the hollow samples. Axial loads exerted by air pressures are applied through a loading shaft as shown in Fig. 1. The initial stresses in a sample prior to application of dynamic shear stress are under isotropical consolidating conditions where no initial torsional stresses exist. As described in the above, the both resonant-column tests and torsional shear tests shapes and cross-sectional dimensions of samples, confining stress conditions and the procedures for applying torsional loads are identical. Heights of samples and strain rates in cyclic loadings, however, are different. Accordingly, it is seen that deformation properties for the shear strain of $\gamma = 10^{-6}$ to 10^{-2} evaluated by the two types of devices in this study can be compared under the identical conditions of sample shapes, confining stresses and loading manners. Torsional loads are measured by a torque pickup which is located in the loading shaft just above a sample to eliminate errors due to the friction. Cyclic torsional forces were applied by controlling air-pressure in a cylinder as schematically shown in Fig. 1. The change in air-pressures in the cylinder is rendered into the torsional forces in the loading shaft through gears as shown in Fig. 1. The air-pressures are controlled by handoperations using pressure-regulators. Pseudo-sinusoidal torsional forces with the period of about 10 seconds are produced. Torsional displacements at the top of a sample were monitored by a potentiometer which is set up just above the sample as shown in Figs. 1 and 2. With use of a wheel, cyclic torsional displacements of the sample are first amplified mechanically five times. The rotation of this wheel is transformed into the change in electric current by the potentiometer. Then, the current is amplified and the amplified current is recorded. In this arrangement, the change in the electric current is linearly proportional to the rotational displacement of the wheel, from infinitely small to infinitely large displacements. This arrangement permits a measurement of shear moduli and dampings for a wide range of shear strains. However, from the limitation of mechanical

systems, the smallest shear to be measured reliably will be about $\dot{\gamma} = 5 \times 10^{-5}$.

The relationship between cyclic torsional forces and cyclic torsional displacements can be recorded in two forms, time histories and hysteresis loops.

TEST PROCEDURES IN TORSIONAL SHEAR TESTS

In torsional shear tests, air-dry and saturated samples were prepared by the procedures identical to those in resonant-column tests. All tests were performed under drained conditions. In each test, fresh sands were used. After isotropically consolidated, a specimen was sheared with a constant shear stress amplitude. Fig. 5 shows one of the typical test results. In this test, the confining lateral pressure σ_r was 2.0 kg/cm^2 . On the other hand, the measured initial static axial stress σ_a was 2.078 kg/cm^2 while the axial load was arranged so that $\sigma_a = 2.0 \text{ kg/cm}^2$. A small difference between σ_a and σ_r is indispensable because σ_a and σ_r are controlled independently in this apparatus. The differences were small in all the tests conducted. The mean principal stress $p = (\sigma_a + 2\sigma_r)/3$ was adopted to represent a confining stress. In the test shown in Fig. 5, p becomes $(2.078 + 2 \times 2.0)/3 = 2.026 \text{ (kg/cm}^2\text{)}$. First, ten times of shear tests were conducted for a sample at shear strain of about 7×10^{-5} . This first stage is represented by the Greek letter I in Fig. 5. The hysteresis loop at the number of cyclic loading $N = 10$ at the Stage I is presented at the extremely left in Fig. 4. Then with a large cyclic shear stress amplitude, the second stage (Stage II) with ten times of shear tests was done. In Fig. 5, seven stages of tests were illustrated. It is seen from Fig. 5 that shear strain amplitudes decrease with the increase in cycles of loadings for stages with larger shear strains. It is also seen from Fig. 5 that as the numbers of cyclic loadings increase, shear moduli increase and dampings decrease. Shear moduli at the tenth cyclic loading are taken as the representative values in the following. Variations of shear moduli with respect to the number of cyclic loadings will also be discussed.

Material used and test programs are listed in Table 1. A wide variation of testings were performed on Toyoura Sand. For the other sands, only one case of testing, for air-dry samples prepared by the spooning and tapping method and with the confining pressure of 1.0 kg/cm^2 , was conducted.

DEFINITION OF SHEAR MODULUS AND SHEAR
STRAIN IN THE TORSIONAL SHEAR DEVICE

In the torsional shear tests, it is necessary to define shear stress and shear strain, because the distributions of torsional shear stresses and shear strains on a cross section of a hollow cylindrical sample are not uniform when torsionally sheared (Ishihara and Li (1972)). Therefore, in this study, the torsional shear stress τ was defined as

$$\tau = \tau_{av} = S/A \quad (1)$$

where τ_{av} means the average shear stress on a cross section of a sample. S in Eq. (1) is the total magnitude of shear stress defined as

$$S = \int_{r_1}^{r_2} \tau_r \cdot 2\pi r \cdot dr \quad (2)$$

in which τ_r is the shear stress at the radius r from the axis of the sample and r_1 and r_2 denote the inside radius and the outside radius of the sample, respectively. Furthermore, A in Eq. (1) is the net area of the cross section of the sample, given by

$$A = \pi (r_2^2 - r_1^2) \quad (3)$$

On the other hand, the torque T which is applied to the axis given by

$$T = \int_{r_1}^{r_2} \tau_r \cdot 2\pi r \cdot r \cdot dr \quad (4)$$

is measured. There could be two extreme states in the respect of deformation properties of soils, one is the perfectly linear elastic state and the other is the perfectly plastic state. In the perfectly linear elastic state, shear stress τ_r is proportional to the radius r . In this case, from Eqs. (2) and (4) the value of S ; S_e is given by

$$S = S_e = \frac{4}{3} \frac{r_2^3 - r_1^3}{r_2^4 - r_1^4} T \quad (5)$$

From Eqs. (1), (3) and (5), the average shear stress at the perfectly linear elastic state is obtained as

$$\tau_{ave} = \frac{S}{A} = \frac{4}{3\pi} \frac{r_2^3 - r_1^3}{(r_2^2 - r_1^2)(r_2 - r_1)} T \quad (6)$$

In the perfect plastic case, shear stress is identical over the cross section of the sample and given by

$$\tau_r = \tau_{avp} \quad (7)$$

in which τ_{avp} represents the shear stress in the perfectly plastic case. From Eqs. (4), (5) and (7), the value of S is obtained as

$$S = \frac{3}{2} \frac{r_2^2 - r_1^2}{r_2^3 - r_1^3} T \quad (8)$$

Eventually, the average shear stress in the perfectly plastic state is obtained as

$$\tau_{avp} = \frac{S}{A} = \frac{3}{2\pi} \frac{1}{r_2^3 - r_1^3} T \quad (9)$$

In this study, the approximate values of r_1 and r_2 are 3.0 cm and 5.0 cm, respectively. With these values, τ_{ave} and τ_{avp} are given by

$$\begin{aligned} \tau_{ave} &= 0.00478 T \\ \tau_{avp} &= 0.00487 T \end{aligned} \quad (10)$$

Since the difference between these two values is small, the mean value of the two values was defined as shear stress for the entire range of shear strain, i.e. $\gamma = 10^{-4} \sim 10^{-2}$ as

$$\tau = (\tau_{ave} + \tau_{avp}) / 2 \quad (11)$$

The shear strain may be proportional to the distance from the center in a given plane section perpendicular to the axis. Therefore, in this study, the shear strain γ can be defined as

$$\gamma = \gamma_{av} = \frac{\theta}{H} \frac{r_1 + r_2}{2} \quad (12)$$

In equation (12), γ_{av} is the average shear strain, θ is the an angle of distorsion in radian and H is the height of the sample.

In determining values of τ and γ from Eqs. (11) and (12), values of

T and θ were read off from their recorded time histories. Secant shear moduli G were defined as

$$G = |\tau| / |\gamma| \quad (13)$$

in which $|\tau|$ and $|\gamma|$ means the amplitudes of shear stress τ and shear strain γ in each loading cycle, respectively.

Dampings were obtained from hysteresis loops of shear stress and shear strain as shown in Fig. 3. Fig. 4 shows typical recorded hysteresis loops. In this figure, N denotes the number of cyclic loadings for a given cyclic shear stress amplitude.

SHEAR MODULUS OF TOYOURA SAND

G/G* ~ γ Relationship

From resonant-column tests on fifteen clean sands, Iwasaki and Tatsuoka (1977) found that shear moduli of clean sands at shear strains of $\gamma = 10^{-6}$, 10^{-5} and 10^{-4} can be approximately represented by

$$G = 900 \frac{(2.17-e)^2}{1+e} p^{0.40} \quad (\text{at } \gamma = 10^{-6}), \quad (14)$$

$$G = 850 \frac{(2.17-e)^2}{1+e} p^{0.44} \quad (\text{at } \gamma = 10^{-5}) \quad (15)$$

$$\text{and } G = 700 \frac{(2.17-e)^2}{1+e} p^{0.50} \quad (\text{at } \gamma = 10^{-4}) \quad (16)$$

where G is the shear modulus in kg/cm^2 , e is the void ratio and p is the mean principal stress in kg/cm^2 . Eq. (16) was originally established by Hardin and Richart (1963) for round-grained Ottawa sands ($e < 0.80$) and for shear strains of 10^{-4} or less. It seems important to compare shear moduli by resonant-column tests with those by torsional shear tests under the identical conditions of void ratios, confining pressures and shear strains. As both of resonant-column tests and torsional shear tests can give shear moduli at shear strain of 10^{-4} , both results from these two tests were compared with the shear moduli estimated by Eq. (16). Data from the two tests were plotted in Figs. 6 through 9 where G/G^* versus γ relationships are presented. In the figures, G is the measured shear modulus and G^* is obtained by substituting void ratio e and confining pressure p into Eq. (16). The values of e and p were measured when measuring shear modulus G for each cyclic loading at each stage. In this procedure, effects of changes in void ratios and confining pressures on shear moduli can be eliminated in each of Figs. 6 through 9. It is seen from Figs. 6 through 9 that at shear strains of around 10^{-4} , shear moduli by resonant-column tests agree well with those by torsional shear tests evaluated at the 10th cyclic loading for confining pressures of 0.25 kg/cm^2 to 2.0 kg/cm^2 . It is also seen that consecutive values of shear moduli between $\gamma = 10^{-6}$ and 10^{-2} can be obtained using both of the

resonant-column apparatus and the torsional shear device.

It may be induced from the test results shown in Figs. 6 through 9 that for shear strain of 10^{-4} or less the effects of strain rate and repeated loading on shear modulus of clean sands are rather small, because both of strain rate and number of repetitions of loading in resonant-column tests are about 500 to 1,000 times as many as those in torsional shear tests. However, with increasing shear strain amplitude the effects of repeated loadings on shear modulus get larger as discussed later. It is also seen from these figures that the G/G^* -values of saturated samples are slightly less than those of air-dry samples in the case of resonant-column tests. The difference is rather small.

Shear Modulus and Void Ratio

It is noteworthy that in Figs. 6 through 9 the effect of void ratio on $G/G^* \sim \gamma$ relationship is not significant. Fig. 10 shows the relationship between shear modulus for $p = 1.0 \text{ kg/cm}^2$ and void ratio at different shear strain levels. In obtaining shear moduli for $p = 1.0 \text{ kg/cm}^2$ from the data of tests where confining pressure p was slightly different from 1.0 kg/cm^2 , measured shear moduli were divided by $p^{0.5}$. This correction was 4% at the largest. Solid curves in Fig. 10 represent shear moduli which are proportional to a function $(2.17-e)^2/(1+e)$. It is seen from Fig. 10 that the change in shear moduli due to the change in void ratio can be evaluated by the function $(2.17-e)^2/(1+e)$ for a wide range of shear strain ($\gamma = 10^{-6}$ to 3×10^{-3}).

Shear Moduli and Mean Principal Stresses

To evaluate the effect of confining pressures on shear moduli, shear moduli G divided by $(2.17-e)^2/(1+e)$ were plotted as functions of p as shown in Figs. 11 and 12. This is to eliminate the effects of void ratios on shear moduli. The values of G in these figures were read off from $G \sim \gamma$ curves as shown in Fig. 5. It may be seen in Figs. 11 and 12 that for a wide range of shear strain, there are linear relationships between $\log G$ and $\log p$ for both of air-dry and saturated Toyoura Sands. This means

that shear moduli for γ and p of interest can be represented by

$$\{G\}_{\gamma, p} = K(\gamma) \frac{(217 - e)^2}{1 + e} p^{m(\gamma)} \quad (17)$$

in which $K(\gamma)$ is a function of shear strain and $m(\gamma)$ is the exponent of p which is also a function of shear strain. Exponents $m(\gamma)$ of Toyoura Sands derived from Figs. 11 and 12 are shown in Fig. 13 as a function of shear strains. A similar representation was firstly indicated by Silver and Seed (1971). The values of exponents $m(\gamma)$ obtained by other investigators are also indicated by hollow circles in Fig. 13. Numbers beside hollow circles correspond to those in Table 2 in which shear tests on clean sands utilizing torsional resonant-column devices, torsional shear devices, ring torsion devices, and simple shear devices are listed. It is seen from Fig. 13 that with increasing shear strain amplitude the exponent $m(\gamma)$ increases from about 0.4 at the shear strain of around 10^{-6} or less to 0.9 at the shear strain of around 5×10^{-3} .

A SIMPLIFIED REPRESENTATION OF STRAIN-DEPENDENCY
IN SHEAR MODULI OF SANDS

$G/\{G\}_{\gamma=10^{-6} \sim \gamma}$ Relationship for Toyoura Sand

From Eq. (17), shear modulus for Toyoura Sand at the shear strain of 10^{-6} is represented by

$$\{G\}_{\gamma=10^{-6}, p} = K (\gamma = 10^{-6}) \frac{(2.17 - b)^2}{1 + e} p^{m(\gamma=10^{-6})} \quad (18)$$

From Eqs. (17) and (18), the ratio of shear modulus at an arbitrary shear strain γ to shear modulus at $\gamma = 10^{-6}$ is derived as

$$\left[\frac{G}{\{G\}_{\gamma=10^{-6}}} \right]_p = \frac{K(\gamma)}{K(\gamma=10^{-6})} p^{m(\gamma) - m(\gamma=10^{-6})} \quad (19)$$

For $p = 1.0 \text{ kg/cm}^2$, Eq. (19) becomes

$$\left[\frac{G}{\{G\}_{\gamma=10^{-6}}} \right]_{p=1.0 \text{ kg/cm}^2} = \frac{K(\gamma)}{K(\gamma=10^{-6})} \quad (20)$$

From Eqs. (19) and (20), a simple equation is derived between $p = 0.25 \text{ kg/cm}^2$ and 2.0 kg/cm^2 as

$$\left[\frac{G}{\{G\}_{\gamma=10^{-6}}} \right]_p = \left[\frac{G}{\{G\}_{\gamma=10^{-6}}} \right]_{p=1.0 \text{ kg/cm}^2} \cdot p^{m(\gamma) - m(\gamma=10^{-6})} \quad (21)$$

The value of $\left[\frac{G}{\{G\}_{\gamma=10^{-6}}} \right]_{p=1.0 \text{ kg/cm}^2}$ at a shear strain of interest was obtained from Fig. 8 by dividing G/G^* -values at the shear strain by G/G^* -value at the shear strain of 10^{-6} . Thick solid curve in Fig. 14 is the measured relationship between $G/\{G\}_{\gamma=10^{-6}}$ for $p = 1.0 \text{ kg/cm}^2$ and shear strain amplitude γ . Dotted curves in Fig. 14 represent $G/\{G\}_{\gamma=10^{-6} \sim \gamma}$ relationships for $p = 0.25, 0.5$ and 2.0 kg/cm^2 which were obtained from solid curves shown in Figs. 6, 7 and 9 by dividing G/G^* -values at arbitrary shear strains γ by G/G^* -values at $\gamma = 10^{-6}$. Thin solid curves in Fig. 14 represent $G/\{G\}_{\gamma=10^{-6} \sim \gamma}$ relationships which were obtained from Eq. (21) by substituting measured

exponents $m(\gamma)$ of Toyoura Sand (Fig. 13) and the measured value of $[G/\{G\}_{\gamma=10^{-6}}]_{p=1.0 \text{ kg/cm}^2}$ represented by a thick solid curve in Fig. 14 into Eq. (21). As understood from Fig. 15, when $G/\{G\}_{\gamma=10^{-6}} \sim \gamma$ relationship for $p = 1.0 \text{ kg/cm}^2$ and $(m(\gamma) - m(\gamma=10^{-6})) \sim \gamma$ relationship are established, Eq. (21) can be a convenient equation to evaluate the $G/\{G\}_{\gamma=10^{-6}} \sim \gamma$ relationship for a confining pressure p other than 1.0 kg/cm^2 . Fig. 15 shows the effects of the number of cyclic loadings on $G/\{G\}_{\gamma=10^{-6}} \sim \gamma$ relationships of Toyoura Sand in the case of $p = 1.0 \text{ kg/cm}^2$. The curves for the 2nd and the 5th cyclic loadings in Fig. 15 were obtained from $G/G^* \sim \gamma$ plottings of measured shear moduli as shown in Figs. 6 through 9. Using these curves and Eq. (21), the value of $G/\{G\}_{\gamma=10^{-6}}$ for values of p and γ of interest may be evaluated for an arbitrary number of cyclic loading between two and ten.

$G/\{G\}_{\gamma=10^{-6}} \sim \gamma$ Relationship for Various Sands

Before establishing $G/\{G\}_{\gamma=10^{-6}} \sim \gamma$ relationship for various sands, measured strain-dependency in clean sands were summarized. In Fig. 16 shown are $G/G^* \sim \gamma$ relationships for p of around 1.0 kg/cm^2 which were obtained by various investigators. The reason why measured shear moduli were divided by $G^* = 700 \frac{(217 - e)^2}{1 + e} p^{0.5}$ in Fig. 16 is that in this way shear moduli by different methods can be compared for the identical values of void ratio and confining pressure. In Fig. 16 also presented is the $G/G^* \sim \gamma$ relationship for $p = 1.0 \text{ kg/cm}^2$ for Bannosu Sand A which was obtained by the procedures as described in Fig. 8. A result for Bannosu Sand A which includes a small amount of fine soils is shown in Fig. 16 for a comparison. It may be seen from Fig. 16 that when limited to the data by torsional resonant-column tests and torsional shear tests (No. 3, 6, 7, 8, 9, 10, 11, 12, 13, 14, 16, 17, 18 and 24), the scatterings among the data might be considered to be rather small in viewing of differences of materials, strain rate, number of cyclic loadings, sample shapes and the other inevitable factors. On the other hand, the difference between these data and those by simple shear tests (No. 21, 22 and 23) may be due to the differences in the fashion of shear deformations in samples and consolidation stress conditions. Accordingly, it may be concluded that resonant-column tests and torsional shear tests can give

consistent results.

In Fig. 17 shown are measured relationships between $G/\{G\}_{\gamma=10^{-6}}$ and γ for $p = 1.0 \text{ kg/cm}^2$ of medium dense Banno-su Sands. These curves were plotted by the procedures similar to those shown in Figs. 8 and 14. As indicated in Table 1, these sands have nonuniform gradings and include fine soils to some extent. The $G/\{G\}_{\gamma=10^{-6}}$ versus γ relationships of the whole sands tested are shown in Fig. 18. As seen in Fig. 18, no significant differences were found among $G/\{G\}_{\gamma=10^{-6}}$ relationships of these sands. It should be noted, however, that shear moduli of these sands are not identical for the identical values of shear strain amplitude γ , void ratio e and confining pressure p . Iwasaki and Tatsuoka (1977) found that shear moduli of disturbed sands for the identical values of γ , e and p , in general, decrease with the increases in uniformity coefficients and contents of fine soils. Furthermore, the index B was defined to represent the relative largeness of shear moduli for the identical values of γ , e and p . Using this index, shear moduli of sands may be represented by

$$G = A(\gamma) \cdot B \cdot \frac{(217 - e)^2}{1 + e} p^{m(\gamma)} \quad (22)$$

where $A(\gamma)$ and $m(\gamma)$ are functions only of shear strain and B is not a function of shear strain but is a characteristic value for each sand. A way to obtain the value of B for each sand is reported in the previous paper (Iwasaki and Tatsuoka (1978)). Fig. 19 shows the relationship between the values of $G/\{G\}_{\gamma=10^{-6}}$ at $\gamma = 10^{-4}$, 5×10^{-4} , 10^{-3} and 3×10^{-3} and the index B for sands tested for $p = 1 \text{ kg/cm}^2$. The values of $G/\{G\}_{\gamma=10^{-6}}$ were obtained from the measured relationship between $G/\{G\}_{\gamma=10^{-6}}$ and γ for each sand. Fig. 19 suggests that the value of $G/\{G\}_{\gamma=10^{-6}}$ for each value of γ is almost constant for a wide range of B . It may be derived from the result shown in Fig. 19 that an average curve of $G/\{G\}_{\gamma=10^{-6}} \sim \gamma$ relationship shown in Fig. 18 can be established for a wide variety of sands for practical engineering purposes.

Seed and Idriss (1970) established an average $G/\{G\}_{\gamma=10^{-6}} \sim \gamma$ relationship for sand on the basis of various data presented by other

investigators. Furthermore, it was shown by Hardin and Drnevich (1972a, 1972b) that the shape of the strain-dependency of shear modulus for a particular soil can be adequately represented by a hyperbolic curve. This curve is defined by the equation

$$G/G_{\max} = \frac{1}{1 + \gamma/\gamma_r} \quad (23)$$

in which G is shear modulus at any shear strain γ , $G_{\max}$ is the value of G at $\gamma = 0$, and γ_r is the reference shearing strain ($\gamma_r = \tau_{\max}/G_{\max}$). For clean sands, $\tau_{\max}$ is given by

$$\tau_{\max} = \left[\left(\frac{1+K_0}{2} \right) \cdot \sin \phi \right]^2 - \left(\frac{1-K_0}{2} \right)^2 \right]^{\frac{1}{2}} \cdot \bar{\sigma}_v \quad (24)$$

in which K_0 is the coefficient of earth pressure at rest and $\bar{\sigma}_v$ is the effective vertical stress. $G_{\max}$ can be obtained from in situ seismic surveys or laboratory soil tests. For round-grained clean sands $G_{\max}$ is given by Eq. (16). In the following Eq. (16) is used to obtain $G_{\max}$ in Eq. (23). Shibata and Soelarmo (1975) proposed a more simplified equation than Eq. (23) as

$$G/G_{\max} = \frac{1}{1 + 1000 \cdot \frac{\gamma}{p^{0.5}}} \quad (25)$$

Furthermore, Sherif and Ishibashi (1976) proposed a different equation on the basis of results by their own torsional shear tests with hollow cylindrical samples as

$$G = 2.8 \phi \left(\frac{p}{0.070307} \right)^{1.167\gamma^{0.05} \cdot 4.0 \cdot (0.205)^{\gamma/3}} \quad (\text{for } \gamma < 3 \times 10^{-4})$$

$$G = 2.8 \phi \left(\frac{p}{0.070307} \right)^{0.85} \cdot (100\gamma)^{-0.6} \quad (\text{for } \gamma \geq 3 \times 10^{-4}) \quad (26)$$

In Eq. (26) p is represented in kg/cm^2 and γ is represented in rad./rad. Note that the notations used in Eq. (26) are different from those in the original paper. Fig. 20 shows a comparison of strain-dependency of shear modulus for $p = 1.0 \text{ kg/cm}^2$ among the average curve in Fig. 18, the average curve by Seed and Idriss and the other proposed relationships, Eqs. (23),

(25) and (26). In Eq. (23), the values of e , ϕ and K_0 should be determined to obtain a particular relationship between $G/G_{\max}$ and γ for $p = 1.0 \text{ kg/cm}^2$. To depict the curve in Fig. 20 from Eq. (23), $e = 0.65$, $\phi = 30^\circ$ and $K_0 = 0.6$ were assumed as standard values. It may be seen that the differences among four curves is rather small except the curve by Shibata and Soelarno.

Exponent $m(r) \sim \gamma$ Relationship

In evaluating the reduction in G with increasing shear strain amplitude with use of Eq. (21), another coefficient $m(r)$ is necessitated. In addition to the data shown in Fig. 13, the values of $m(r)$ at $\gamma = 10^{-6}$, 10^{-5} and 10^{-4} which were obtained for fifteen clean sands are also plotted in Fig. 21. Properties of these clean sands are reported elsewhere (Iwasaki and Tatsuoka (1978)). A solid curve in Fig. 21 represents the average curve of $m(r)$ for the whole data shown in Fig. 21. Furthermore, the values of $m(r) - m(10^{-6})$ are shown in Fig. 22. Note that in deriving the $G/\{G\}_{\gamma=10^{-6}}$, the values of $m(r) - m(10^{-6})$ are needed. The value of $m(r) - m(10^{-6})$ which is obtained from the average curve in Fig. 21 was compared with those which are derived from Eqs. (23), (25) and (26). Two black circles in the figure represent the average values of $m(r) - m(10^{-6})$ at $\gamma = 10^{-5}$ and $\gamma = 10^{-4}$ which were obtained by resonant - column tests on 33 non-clean sands. Properties of these sands were also presented in the previous paper (Iwasaki and Tatsuoka (1978)). Fig. 22 indicates that the difference among black circles and these curves is rather small and that differences among other curves except the curve by Sherif and Ishibashi is also rather small.

When earthquake response analyses are performed for a particular ground, it is desirable to conduct laboratory tests to evaluate $G/\{G\}_{\gamma=10^{-6}}$ or $G/G_{\max} \sim \gamma$ relationship for each soil layer. However, this procedure is, in general, rather laborious and sometimes difficult. When such laboratory tests cannot be performed, Eq. (21) may be adopted for sand layers for preliminary studies in which the average curve of $G/\{G\}_{\gamma=10^{-6}}$ for $p = 1.0 \text{ kg/cm}^2$ shown in Fig. 18 and the average curve of $m(r)$ shown in Fig. 21 are available.

CONCLUSIONS

The test results and the analyses presented show that by combining resonant-column tests and torsional shear tests using hollow cylindrical samples, consecutive values of shear moduli of sands for shear strain amplitude ranging $\gamma = 10^{-6}$ to 10^{-2} can be evaluated.

On the basis of the results by resonant-column tests and torsional shear tests on various sands, a simplified equation (21) was proposed to represent reduction in G with increasing shear strain amplitude for sands. This equation with the average curves shown in Figs. 18 and 21 may be available in conducting earthquake response analyses.

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NOTATION

e = void ratio

$m(r)$ = exponent of mean principal stress in relation with shear modulus

$p = \frac{1}{3} (\sigma_a + 2\sigma_r)$; mean principal stress

$A(r)$ = a function of shear strain in relation with shear modulus

B = a parameter representing relative largeness of shear modulus

G = shear modulus

$\{G\}_{r=10^{-6}}$ = shear modulus at $r = 10^{-6}$

$G_{\max}$ = shear modulus at $r = 0$

$G^* = 700 \frac{(2.17 - e)^2}{1 + e} p^{0.5}$; the average shear modulus of clean sands
at $r = 10^{-4}$

N = number of cyclic loadings

γ = single amplitude shear strain

τ = shear stress

$\eta = \text{damping ratio} = \frac{1}{2\pi} \frac{\Delta W}{W}$

σ_a, σ_r = axial and radial stresses

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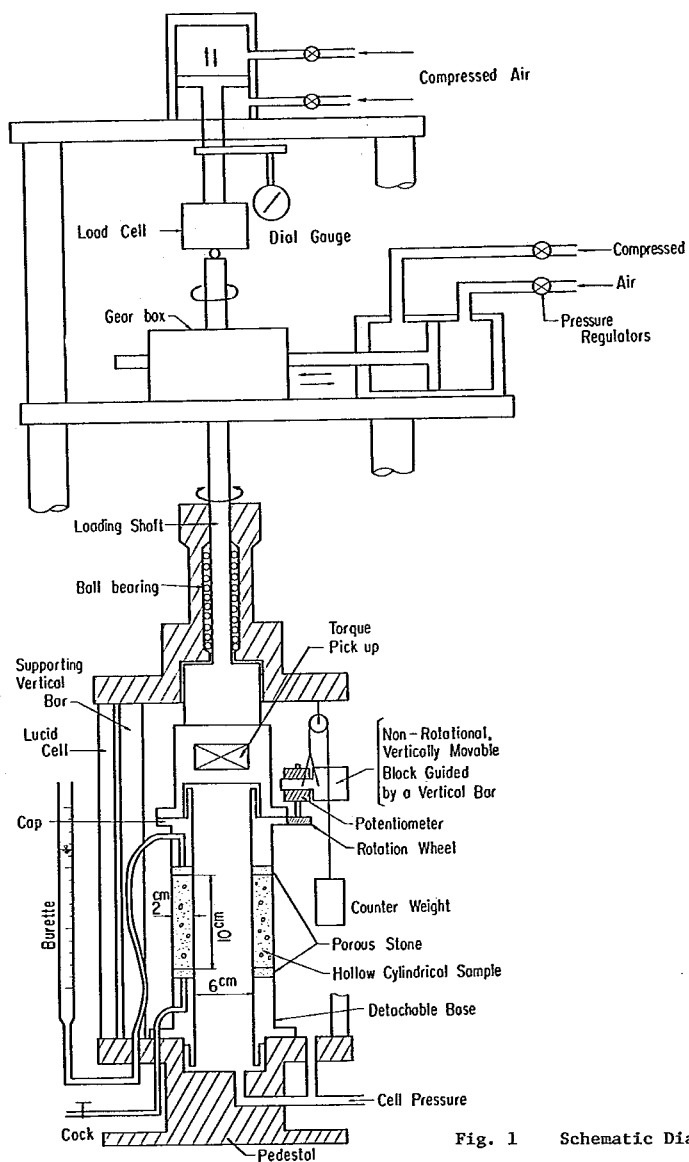


Fig. 1 Schematic Diagram
of Torsional Shear Device

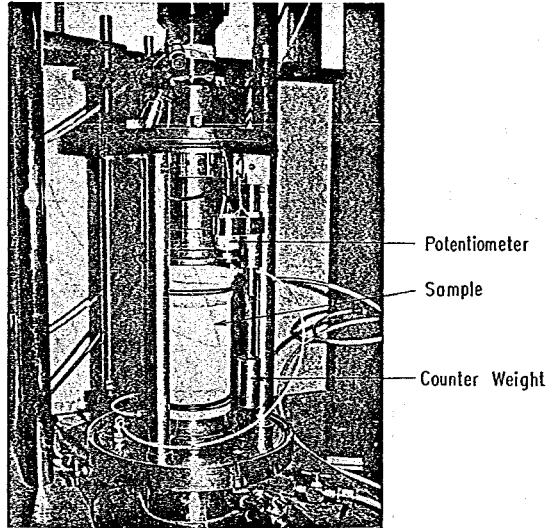


Fig. 2 Close view of Triaxial Cell of Torsional Shear Device

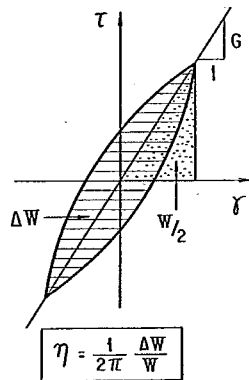


Fig. 3 Definition of Shear Modulus and Damping Ratio

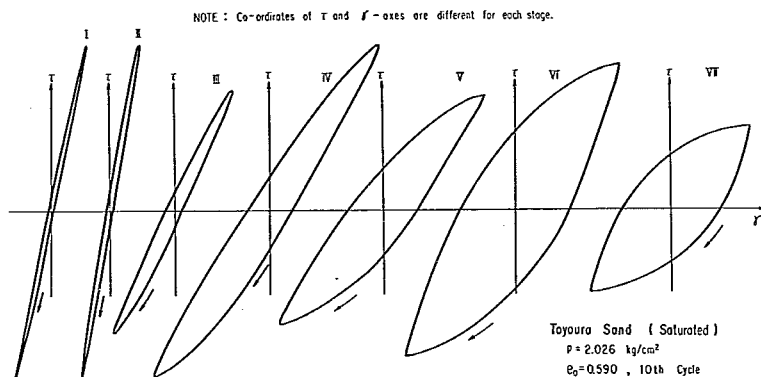


Fig. 4 Stress-Strain Records at 10th Cycle of I to VII Stages (Shear moduli and damping ratios of this test are shown in Fig. 5.)

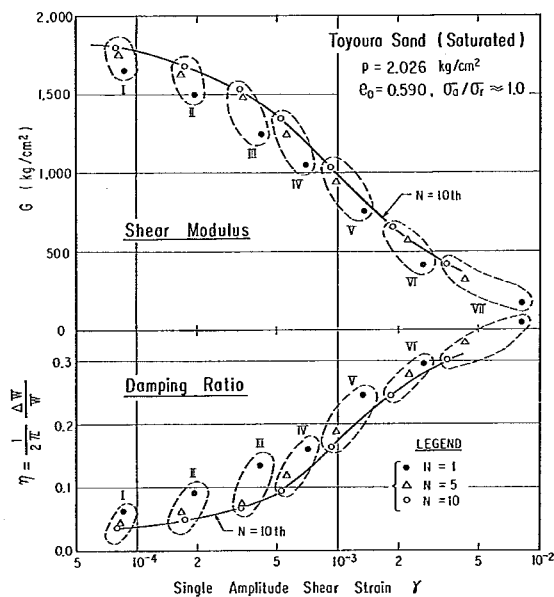


Fig. 5 Typical Test Result

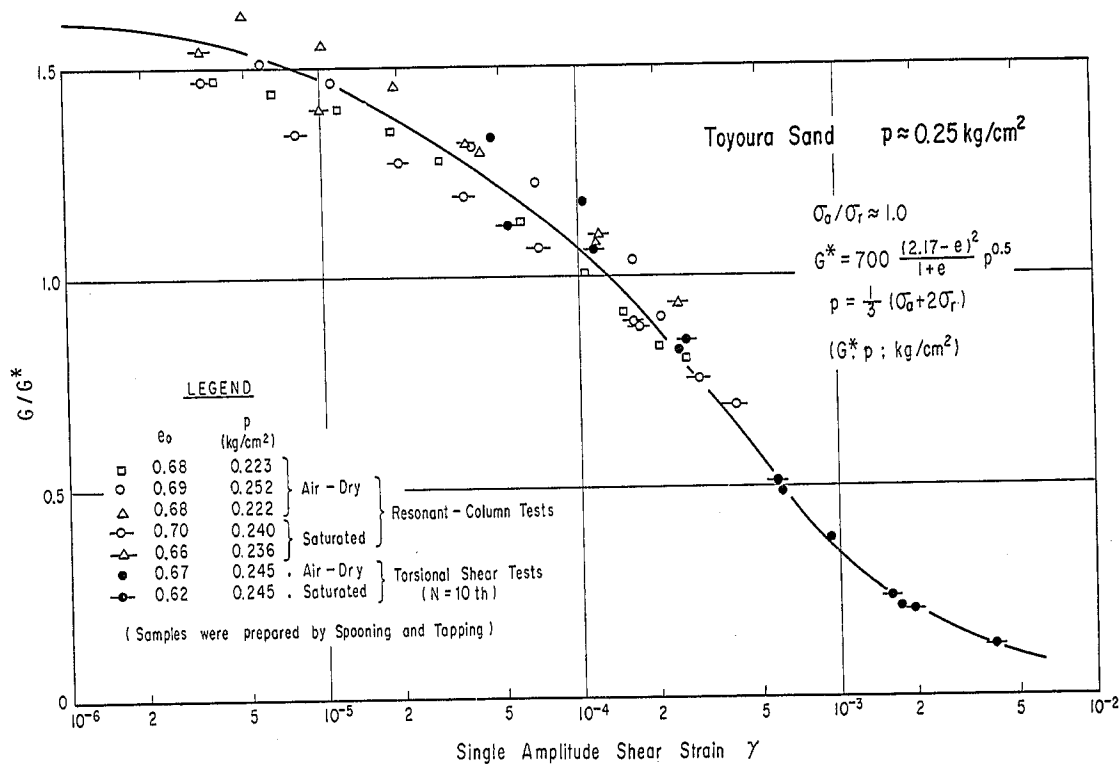


Fig. 6 G/G^* versus γ Relationship of Toyoura Sand ($p = 0.25 \text{ kg/cm}^2$)

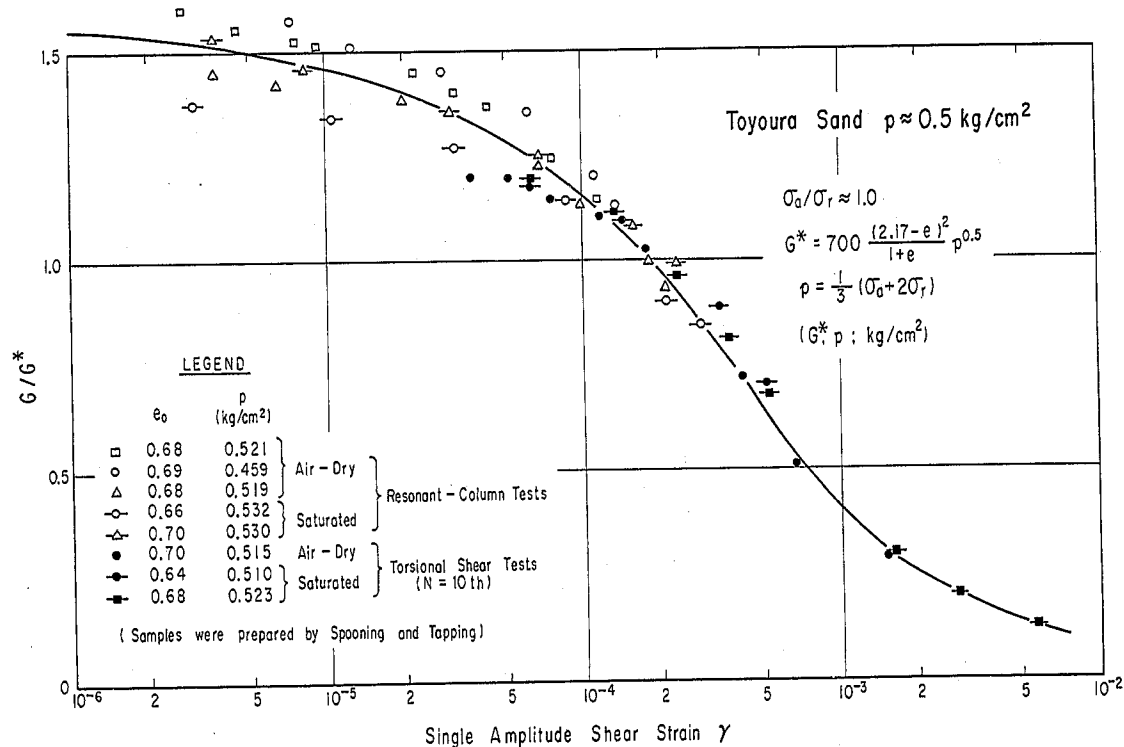


Fig. 7 G/G^* versus γ Relationship of Toyoura Sand ($p = 0.5 \text{ kg/cm}^2$)

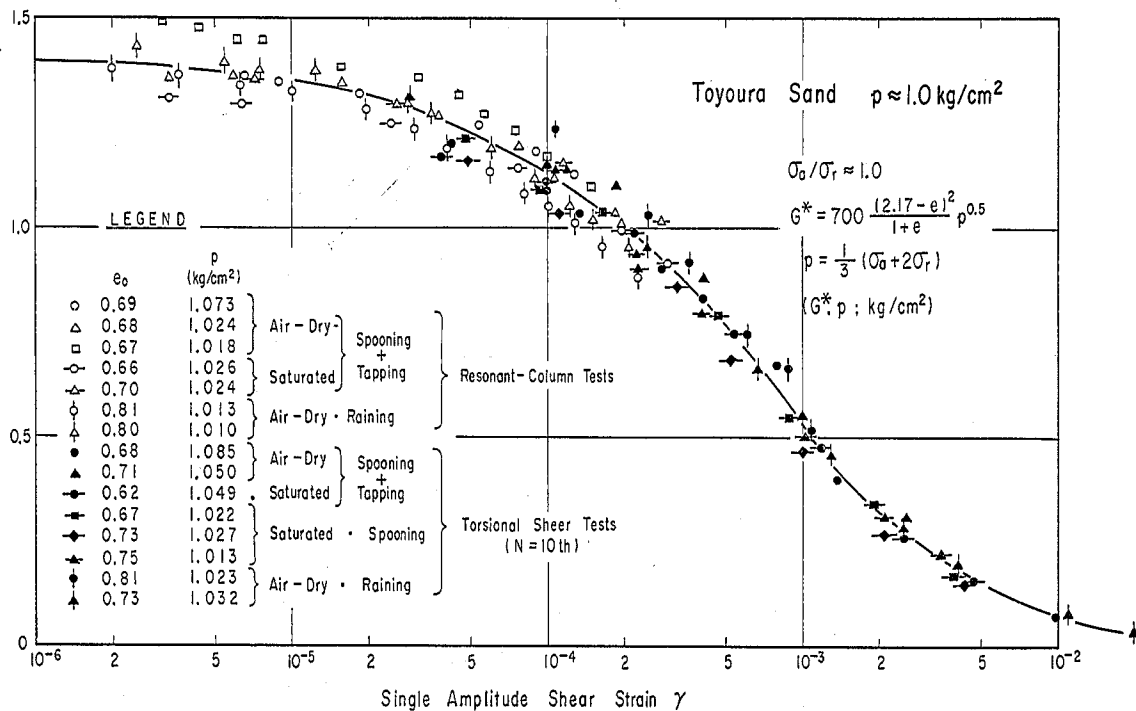


Fig. 8 G/G^* versus γ Relationship of Toyoura Sand ($p = 1.0 \text{ kg/cm}^2$)

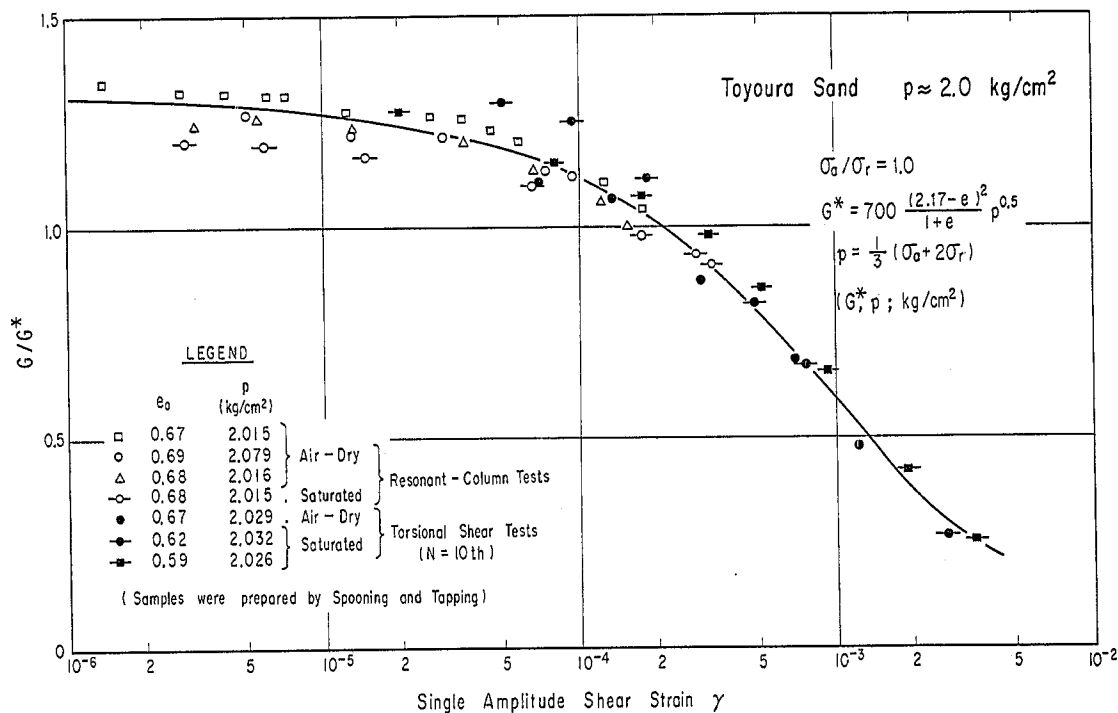


Fig. 9 G/G^* versus γ Relationship of Toyoura Sand ($p = 2.0 \text{ kg/cm}^2$)

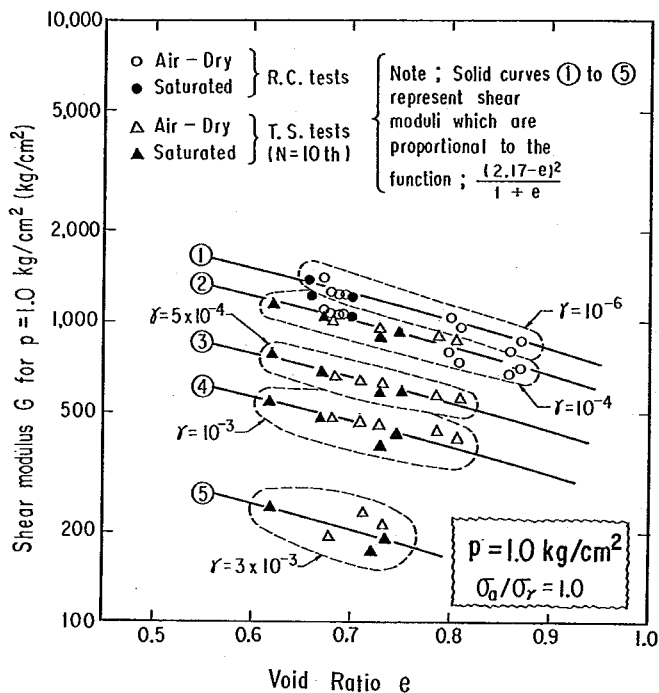


Fig. 10 G versus e Relationship of Toyoura Sand

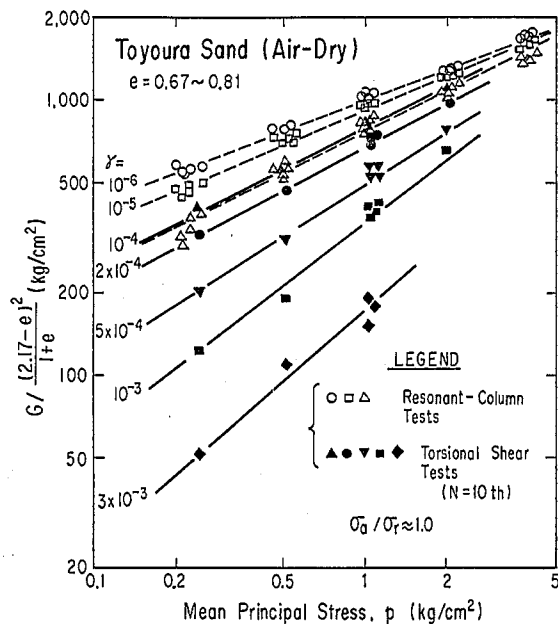


Fig. 11 $G / \frac{(2.17 - e)^2}{1 + e}$ versus p Relationship
 of Air-Dry Toyoura Sand

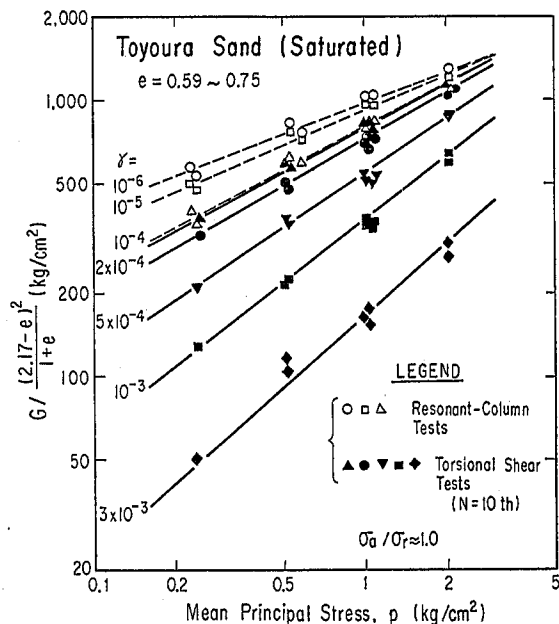


Fig. 12 $G / \frac{(2.17 - e)^2}{1 + e}$ versus p Relationship
 of Saturated Toyoura Sand

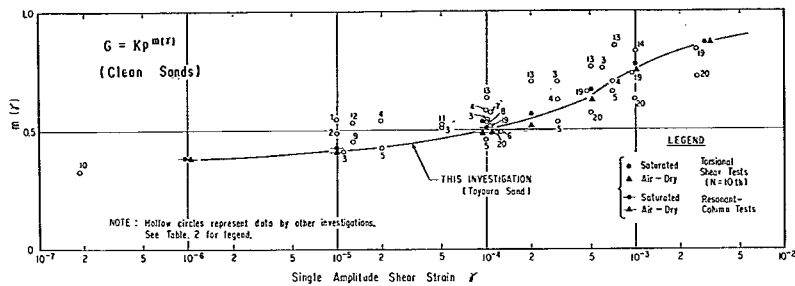


Fig. 13 $m(r)$ versus γ Relationship of Clean Sands

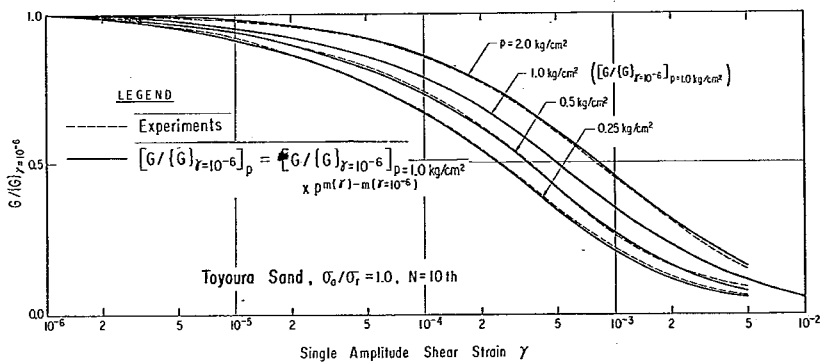


Fig. 14 $G/(G)_{\gamma=10^{-6}}$ versus γ Relationship of Toyoura Sand

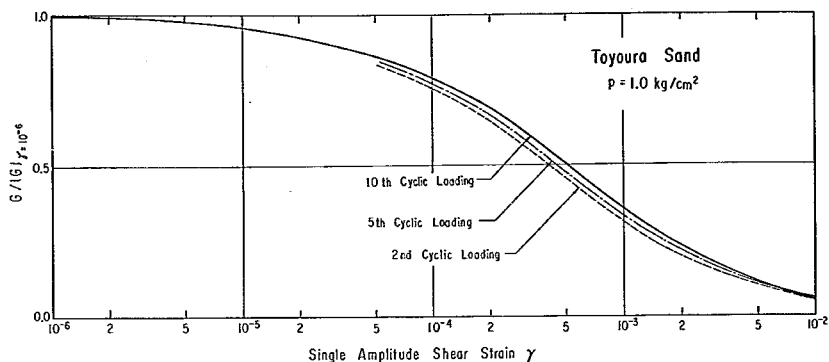


Fig. 15 Effects of Repeated Loading on G/G_r $\gamma=10^{-6} \sim \gamma$ relationship of Toyoura Sand

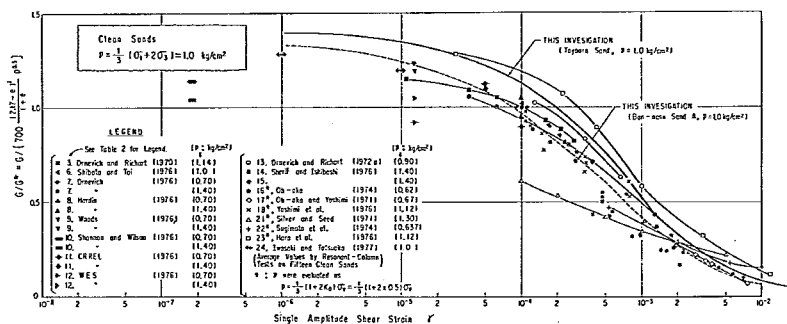


Fig. 16 G/G^* versus γ Relationship of Various Clean Sands

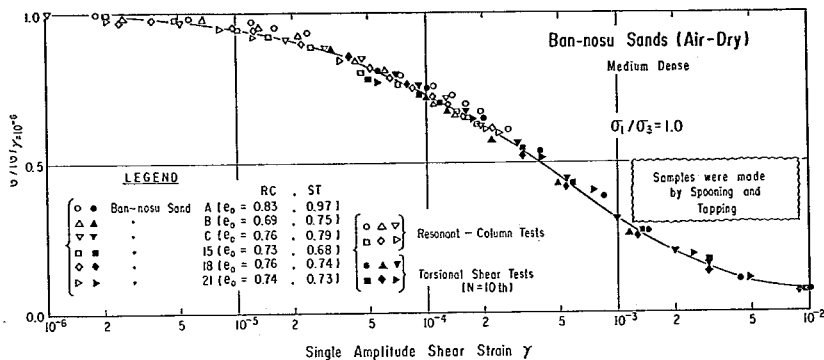


Fig. 17 $G/{G}_{\gamma=10^{-6}}$ versus γ Relationship of Ban-nosu Sands

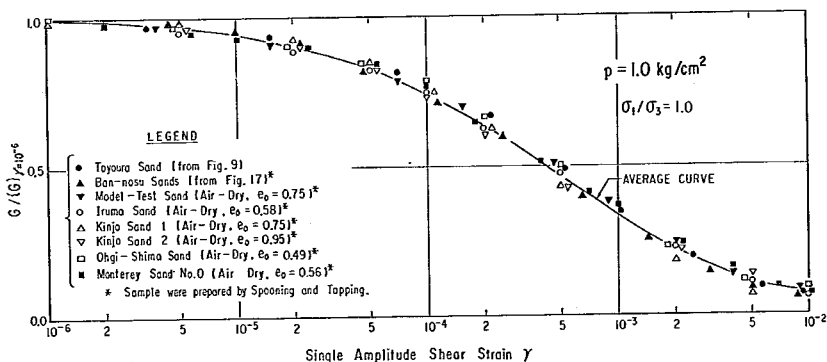


Fig. 18 Average $G/{G}_{\gamma=10^{-6}}$ versus γ Relationship of Sands Tested

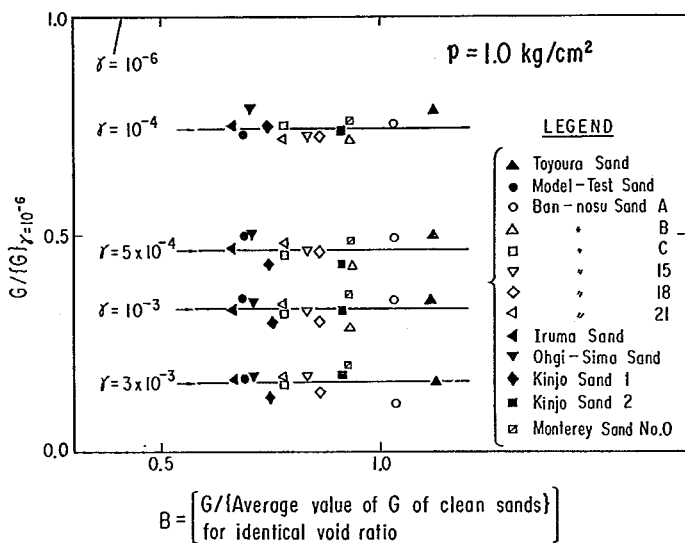


Fig. 19 $G/\{G\}_{\gamma=10^{-6}}$ versus B Relationship of Sands Tested.

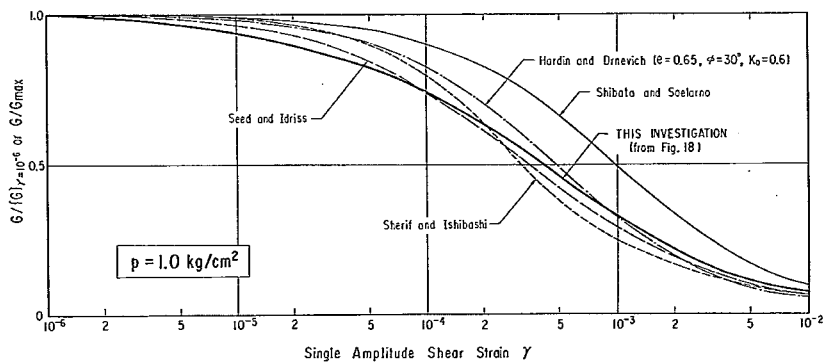


Fig. 20 Comparison among $G/\{G\}_{\gamma=10^{-6}} \sim \gamma$ Relationships

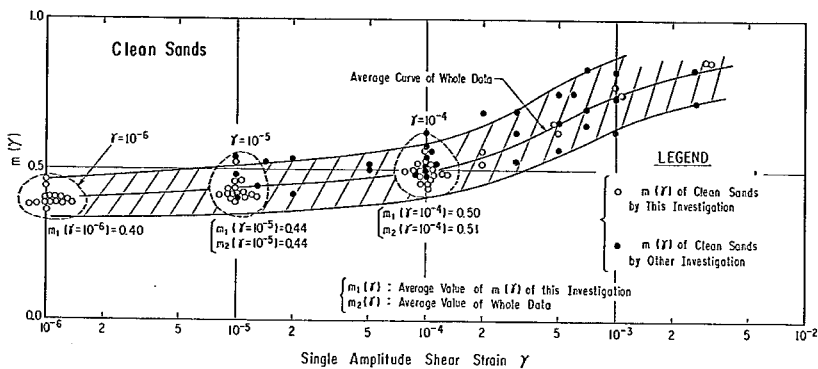


Fig. 21 $m(\gamma) \sim \gamma$ Relationship

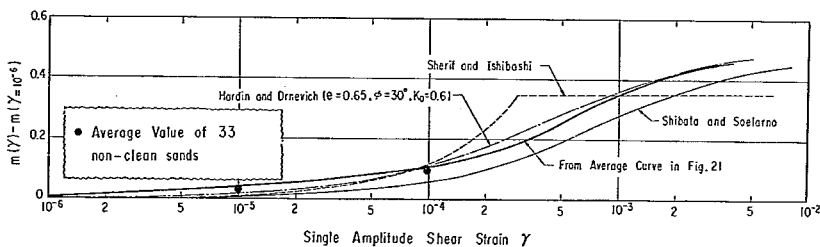


Fig. 22 Comparison among $m(\gamma) - m(\gamma = 10^{-6}) \sim \gamma$ Relationships

Table 1. Test Materials and Test Programs

Material	G _s	D ₁₀ (mm)	D ₅₀ (mm)	U _c	e _{max}	e _{min}	F.C. (%)	*)	**)			
								Air-Dry or Saturated	Sample Preparation	Density	Confining Pressures p (kg/cm ²)	
Toyoura Sand	2.64	0.12	0.162	1.46	0.96	0.64	0.0	A-D	RN	Loose	1.0	
								S	SP	Med-Loose	1.0	
								A-D, S	SP + TP	Dense, Med-Dense	0.25, 0.5, 1.0, 2.0	
Model-Test Sand***)	2.88	0.31	0.48	1.52	0.93	0.66	0.5	Air-Dry	Spoon- ing and Tapping	Dense or Medium- Dense	1.0	
Ban-nosu Sand A	2.67	0.168	0.26	1.70	1.17	0.82	1.1					
" B	2.66	0.31	0.92	3.35	1.01	0.68	0.5					
" C	2.68	0.15	0.34	2.73	1.03	0.69	5.9					
" 15	2.67	0.24	0.88	4.58	0.91	0.69	0.26					
" 18	2.67	0.31	0.99	1.20	0.94	0.68	0.5					
" 21	2.67	0.25	0.72	3.60	0.92	0.67	0.8					
Iruma Sand	2.75	0.28	0.50	1.98	0.84	0.57	0.6					
Ohgi-Sima Sand	2.69	0.20	0.37	2.25	0.72	0.49	0.36					
Kinjo Sand 1	2.66	0.16	0.42	2.88	1.11	0.71	3.0					
Kinjo Sand 2	2.67	0.09	0.32	3.89	1.42	0.92	8.8					
Montorey No. 0 Sand	2.65	0.32	0.44	1.98	0.85	0.56	0.0					

*) F.C.; Content of Fine Soils less than 0.074 mm.

**) SP ; Spooning Method, SP + TP; Spooning and Tapping Method, RN; Raining Method

***) After oven-dried several times.

Table 2. List of Other Investigations

		*)		**)			
No.	Sand	Void Ratio	Device	Sample	N	σ_a/σ_r	References
1	Dry Ottawa No. 20-30	Dense	Resonant-Column (RC)	Solid Cylinder	-	1.0	Hardin and Richart (1963)
2	Dry Crushed Quartz, No. 20-30	Dense					
3	Dry Ottawa, No. 30-50	0.55	RC (Drnerich)	Hollow Cylinder	-	1.0	Drnevich and Richart (1970)
4	Dry Ottawa No. 20-30	0.46	"	"	-	1.0	Drnevich (1967)
5		0.62					
6	Dry Toyoura	0.64	RC	Hollow Cylinder	-	1.0	Shibata and Tai (1976)
7	Dry Reid-Bedford-Model Sand	0.66	RC (Drnevich)	Solid Cylinder	-	1.0	Skoglund, Marcuson and Cunney (1976)
8		0.64	RC (Hardin)				
9		0.65	RC (Woods)				
10		0.65	RC (Shannon and Wilson)				
11		0.55	RC (CRREL)				
12		0.65	RC (WES)				
13	Clean Dry Sand	0.57	Torsional Shear	Hollow Cylinder	10th	1.0	Hardin and Drnevich (1972a)
14	Dry Ottawa	0.55	"	"	2nd	1.0	Sherif and Ishibashi (1976)
15	Dry Seward-Park Park	0.66, 77					
16	Dry Niigata	0.88	Ring Torsion	Hollow Cylinder	100th	PS	Oh-Oka (1974)
17		0.76			-		Oh-Oka and Yoshimi (1971)
18	Dry Toyoura	0.71			10th		Yoshimi et al. (1976)
19	Dry Silica Sand No. 20	0.83	NGI-Simple Shear	Circular	10th	PS	Silver and Seed (1971)
20		0.71					
21		0.78					
22	-	0.56	Simple Shear	Box	-	PS	Suginoto et al. (1974)
23	Dry Toyoura	0.71	Simple Shear	Circular	10th	PS	Hara et al. (1976)
24	15 Clean Sands	-	RC	Hollow Cylinder	-	1.0	Iwasaki and Tatsuoka (1977)

*) Number of Cyclic Loadings (- ; unknown)

**) PS; Plane Strain Condition