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DYNAMIC DEFORMATION AND FAILURE CHARACTERISTICS OF ROCKFILL MATERIAL SUBJECTED TO CYCLIC SHEAR LOADING UNDER VERTICAL VIBRATION

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ABSTRACT

In recent years, strong random earthquake motions have become a matter of great interest among earthquake engineers concerned with the design earthquake of important civil engineering structures such as fill dams. The influence of a strong random earthquake on fill dams is characterized by large accelerations (body forces) in both vertical and horizontal directions.

The following four kinds of experiments are conducted in order to investigate the dynamic strength and deformation characteristics of rockfill material: (1) static shear tests; (2) static shear tests with vertical vibration; (3) dynamic shear tests and (4) dynamic shear tests with vertical vibration.

The following results are obtained;

(1) The body force affects significantly the dynamic deformation behaviours, but only slightly the static deformation and strength.

(2) If the ultimate dynamic strength is defined as the maximum shear stress by which the specimen fails in the same way by the static shear stress, the ultimate dynamic strength of a dense rockfill material becomes larger than the static strength in a range of lower sustained stress.

(3) The rockfill material becomes unstable with a disturbed grain structure and a large deformation when it is subjected to the dynamic loads exceeding the yield stress ratio, $(\tau_s/\sigma_n)_y$.

Thus, the yield stress ratio becomes an important criterion for the dynamic stability of earth-structures, and the yield stress ratio can be obtained from the static shear test.

Key words: dam, deformation, dynamic, earthquake, failure, granular material, shear strength, special shear test, vibration, yield

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INTRODUCTION

Recently, there has been a trend to take into account the influence of strong random

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earthquake in selecting the design earthquake for the seismic stability analysis of important civil engineering structures such as fill dams. This trend seems to have become remarkable after the San Fernando earthquake which occurred near the Soledad fault in California in February of 1972. The hypocenter of the earthquake was found at the shallow depth of about 13 km below the ground surface. As a result, the maximum accelerations recorded near the Pacoima Dam, 14 km away from the epicenter, reached 1 000 gal or more in the vertical and horizontal directions (Committee on Earthquake Engineering, JSCE, 1972).

Typical strong random earthquakes such as the Izu-Hanto-oki earthquake of May, 1974 and the Oh-ita earthquake of April, 1975 were recorded also in Japan. The design earthquake criteria for the Proposed Los Angeles Dam (the earth fill dam with a height of 52 meters) is based on the experiences gained through the San Fernando earthquake, and is quite severe. The accelerations setforth (body forces) are very large as shown in Table 1 (Dames & Moore Consulting Engineers, 1973 a).

Considering this trend of assigning very large accelerations for the design earthquake, the evaluation of the dynamic strength of rockfill materials seems to have a significant influence on the aseismatic design of fill dams. However, the dynamic failure characteristics of granular materials such as sand or crushed rock materials have not been clarified sufficiently so far, and several different definitions of dynamic strength have been used for different circumstances.

At the time of an earthquake, the forces or stresses acting on an infinitesimal soil element in the earth or in the earth structures may be summarized as follows [see Fig. 1 (Tokue, 1976)]:

(1) static stresses: overburden soil stresses of constant amplitude act macroscopically on the surfaces of the element,

(2) dynamic stresses: fluctuating stresses act macroscopically on the surfaces of the element due to the inertia forces and

(3) body forces: inertia forces of earthquake acceleration act microscopically and directly on each soil grain within the element. They are considered to cause a quiver of grains at the points of contact and are considered qualitatively different in their

Table 1. The design earthquake criteria for the proposed Los Angeles Dam at the time of the local event (Dames & Moore Consulting Engineers, 1973 a)

Duration	The duration of the earthquake, defined as the time between the first acceleration peak exceeding 0.05 g and the last peak exceeding 0.05 g, should be approximately 40 seconds.
Acceleration	1 peak $\geq$ 1.15 g 1 peak $\geq$ 1.00 g Fifth highest peak $\geq$ 0.85 g Tenth highest peak $\geq$ 0.65 g All of these peaks should be at frequencies between 5 and 8 hertz.
Velocity	First peak $\geq$ 135 cm/s Second peak $\geq$ 115 cm/s Third peak $\geq$ 100 cm/s
Displacement	Maximum displacement should be equal to or greater than 70 cm.

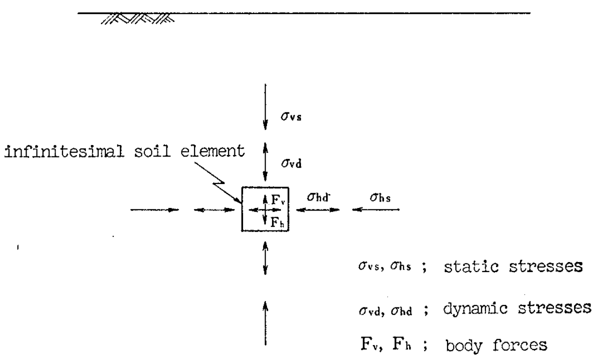

Fig. 1. Forces and stresses acting on a soil element in the earth at the time of earthquake (Tokue, 1976)

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effects from the dynamic stresses acting macroscopically on the surfaces of the element.

Main mechanical factors which were taken into account in the previous studies seem to be the following two factors based on the concept described above. The effect of water is excluded here.

One is the acceleration of earthquake correspondent to the above third factor which is an important factor to be clarified as indicated above. Mogami and Kubo (1953), Barkan (1962), Youd (1970) and others investigated the effect of acceleration on the shear strength of dry sand. In these studies, direct shear tests were performed on the shaking tables vibrating in the horizontal or vertical directions.

It was shown that the shear strength or the internal friction angle of sand decreased with an increase of acceleration. On the other hand, Takeshita and Futaba (1966) concluded from similar experiments with dry sand under horizontal vibration that the shear strengths were the same both with and without vibrations. The discrepancy between the above two results about the effect of acceleration on the shear strength needs be investigated further, along with the types of the experimental apparatus used and the testing conditions adopted.

In the studies described above, the dynamic stresses applied were negligibly small compared to the applied static stresses because of the use of small specimens, although the static stresses and the body forces (acceleration itself) are applied adequately to the smaple in comparison with the actual condition given to the soil element.

Therefore, it may be reasonable to consider that if the decrease in the shear strength occurs during vibration, it results from the body force defined above and does not result from the dynamic stress. In the vibratory compaction of sand, it has already been reported that the effect of the body force besides the effect of the dynamic stress should be taken into account (Tokue, 1976).

The other mechanical factor is the dynamic stresses corresponding the second factor defined above.

Seed (1960), Lee and Seed (1967), Toki and Kitago (1974) and others investigated the effect of dynamic stresses on the shear strengths of clay and sand. In these studies, the dynamic principal stresses with constant amplitudes were applied to the specimens after the application of the initial anisotropic confining pressures (the sustained stress) by using a triaxial shearing apparatus.

It was shown for dry sand that the dynamic strength became larger than the static shear strength under the application of a small sustained stress, but the dynamic strength became less than the static strength when the sustained stress was increased (Toki and Kitago, 1974).

It should be noted that a cumulative axial strain of 25 percent was adopted as a criterion of dynamic failure for compacted silty clay by Seed (1960). Toki and Kitago (1974) adopted a cumulative axial strain of 15 percent for dry sand as a criterion for "dynamic strength". In discussing the aseismatic stability of actual earth-structures, however, it seems quetionable in many cases to consider that the cumulative strains of soil elements within earth-structures approach 15 to 25 percent. In this sense, the dynamic criteria adopted by Seed or Toki and Kitago are close to "the ultimate dynamic strength of soils", which is defined as the maximum resultant value of dynamic and sustained stresses that a specimen can support in the same way as the static shear strength.

A cumulative axial strain of 5 percent was adopted as the failure criterion of the Proposed Los Angeles Dam by Dames & Moore and Seed (Dames & Moore Consulting Engineers, 1973 b). This criterion was determined by considering the failure due to

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liquefaction. It has not been confirmed sufficiently so far whether the criteria mentioned above are appropriate or not for actual aseismatic problems. In the studies described above, unlike the actual earthquakes, the body forces are characteristically not applied to the specimens while the static and dynamic stresses are applied adequately. Based on all the above indications, the objectives of this study are following:

(1) to clarify the effect of body force on the shear strength of rockfill material by conducting static shear tests with the vartical vibration which is considered to occur during a strong random earthquake.

(2) to investigate the combined effect of the body force and the dynamic stress on the dynamic deformation characteristic of rockfill material by conducting dynamic shear tests with vertical vibration, and

(3) to investigate a proper criterion replacing the ultimate dynamic strength for the aseismatic design of earth-structures such that the dynamic deformation behavior of rockfill material determined by dynamic shear tests are taken into consideration.

In order to achieve these objectives, a new large-scaled simple shear apparatus for testing rockfill material has been developed (Tokue, Kitahara and Fujiwara, 1976). The characteristic of the apparatus is such that the dynamic shear stress and the vertical vibration can be applied to a large-scaled specimen simultaneously or separately.

TESTING MATERIAL, APPARATUS AND CONDITION

The parent rock of the rockfill material used is an unweathered granite with a specific gravity $G_s=2.61$. The rockfill material has a mean diameter $D_{50}=26.5$ mm and a uniformity coefficient $U_c=10.0$. The distribution curve of grain size is shown in Fig. 2. The initial void ratios, e_0, range from 0.41 to 0.47, when the crushing of grains is prevented, and the material is tested in a fairly dense state. The specimens are compacted by vibration. The material is tested dry in order to simulate the condition at a crest and a downstream rockfill zone of a dam.

The schematic diagram of the apparatus

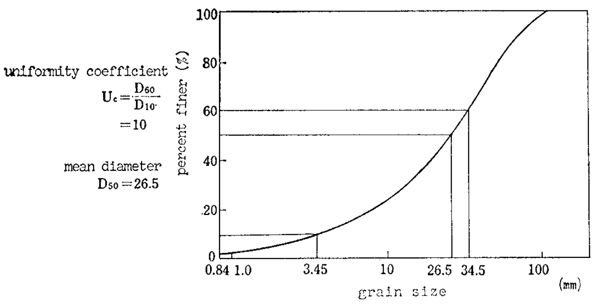
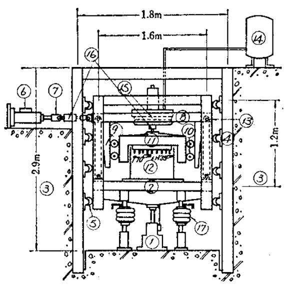


Fig. 2. Grain size distribution curve of the rockfill material



1. actuator for vertical vibration (full capacity: 5×10^4N)
2. inner frame work
3. concrete mass
4. outer frame work
5. rollers
6. actuator for shear (full capacity: 10^5N)
7. universal joint
8. pin-jointed shearing frame
9. vertical plate
10. rollers
11. loading plate
12. specimen(Φ70cm H25cm)
13. pin- joint
14. air tank
15. air spring for overburden pressure (full capacity: 10^5N)
16. load transducer
17. air spring for support of dead weight

Fig. 3. Schematic diagram of simple shear apparatus

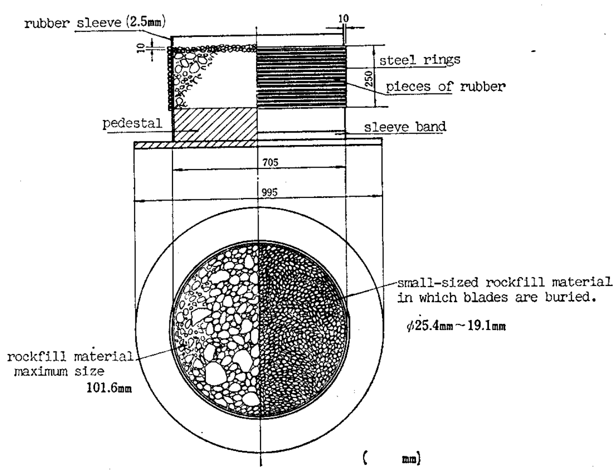


Fig. 4. Schematic diagram of a specimen

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is shown in Fig. 3.

a) Shearing apparatus

The shearing apparatus used in this study has three different functions: vertical vibrating, shear loading and overburden pressure loading. Their mechanisms are shown in Fig. 3. Vertical vibration is generated by an actuator, 1, having a full capacity of 5×10^4N and transmitted to a specimen, 12, by vibrating an inner frame work, 2, on which the specimen rests. Horizontal displacement of the frame work is constrained by rollers, 5, attached on an outer frame work, 4, and only the vertical displacement is allowed. Shear stress is applied to the specimen up to 250 kN/m^2 by an actuator, 6, having a full capacity of 10^5N. The actuator delivers its load to the specimen through a universal joint, 7, a load transducer, 16, a pin-jointed shearing frame, 8, a vertical plate, 9, rollers, 10, and a loading plate, 11.

The vertical plate, 9, is fixed rigidly to the pin-jointed shearing frame. By this mechanism, the volume change of the specimen due to dilatancy is allowed during shearing and the upper surface of the specimen is always kept parallel to the bottom of the specimen. The effect of friction on the shear stress is almost removed by the adoption of the pin-jointed structure as described above.

Blades with the height of 2 cm and the width of 1 cm are attached on the lower surface of the loading plate, 11, at an interval of 5 cm. Therefore, the slip between the loading plate and the specimen is effectively inhibited.

Overburden pressure is applied to the specimen up to 250 kN/m^2 by an air spring, 15, which is fixed on the pin-jointed shearing frame, 8.

b) Specimen

A schematic diagram of the specimen is shown in Fig. 4. The specimen is 70 cm in diameter and 5 to 50 cm in height. In the present experiments, the height is 25 cm tall to take the maximum size of grains into account. The mechanism used for constraining lateral expansion of the specimen is a Norweigian Geotechnical Institute type, and is formed by stacking up steel rings with a square section of 1 cm$\times$1 cm outside a rubber sleeve with the thickness of 2.5 mm.

The radial strains of the rings measured are less than 10^{-5} under the application of the maximum overburden pressure; therefore, the lateral expansion of the specimen is considered practically inhibited. The rubber pieces are placed between rings at an intervel of 120 degrees. As a result, the rings can deform easily in the vertical direction under the application of overburden pressure; thus allowing the application of the applied overburden pressure to the specimen without loss.

c) Control system

The vertical vibrating motion and dynamic shear load are controlled electrically. An input signal for both vibration and dynamic shear load is generated by the same variable phase function generator. Thus, the operating periods of the actuators, 1 and 6, are the same, and their phase differences can be varied continuously from 0 to 360 degrees. The amplitude of dynamic shear stress is kept constant at a set value by an electric controlling device. Dynamic shear load can be added on to static shear load, and the ratio between the two can be set freely so long as their total load does not exceed the full capacity of the actuator, 10^5 N. The overburden pressure is controlled by a regulator.

In order to investigate the effect of vertical vibration on static and dynamic deformation and failure behaviours of the rockfill material, the following four experiments are conducted: ① static shear tests; ② static shear tests with vertical vibration; ③ dynamic shear tests; and, ④ dynamic shear tests with vertical vibration.

Testing and loading conditions are shown in Table 2 and in Fig. 5, respectively. The

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Table 2. Testing conditions

Test type	Loading condition	Loading method	Vibrating condition	Measurements	Measuring instruments	Initial void ratios
1) Static shear test	σ_n=0.2, 1.0, 1.7, 2.5 (kg f/cm^2)	Shear stress, τ_s, is increased until failure by stages with an increment of about 10% of overburden pressure, σ_n, after displacements are finished in each stage	—	Horizontal displacement (at 3 points). Vertical displacement (at 2 points). Overburden load. Shear load	Displacement-transducers Load-transducers	0.43 ~ 0.47
2) Static shear test with vertical vibration	σ_n=0.5, 1.5, 2.5 (kg f/cm^2)	Shear loading is performed in the same way as 1) under the application of vertical vibration	α_v=300~800 (gal) f=4 and 7 (Hz)	In addition to 1), vertical acceleration (at 2 points)	In addition to 1), accelerometers	0.42 ~ 0.44
3) Dynamic shear test	σ_n=1.5 (kg f/cm^2) τ_s=0.75 (kg f/cm^2) $\frac{\tau_s+\tau_d}{\tau_{sf}}$= 0.8~1.22	Dynamic shear stress, τ_d, is applied after the application of sustained stress, τ_s, for 5000 cycles	f=4 (Hz)	In addition to 1), dynamic shear load and loading time	The same as 1)	0.42 ~ 0.43
4) Dynamic shear test with vertical vibration	〃	Shear loading is performed in the same way as 3) under the application of vertical vibration	α_v=330 (gal) f=4 (Hz) Phase difference : 0 and 180 (deg.)	In addition to 3), vertical acceleration	The same as 2)	0.41 ~ 0.43

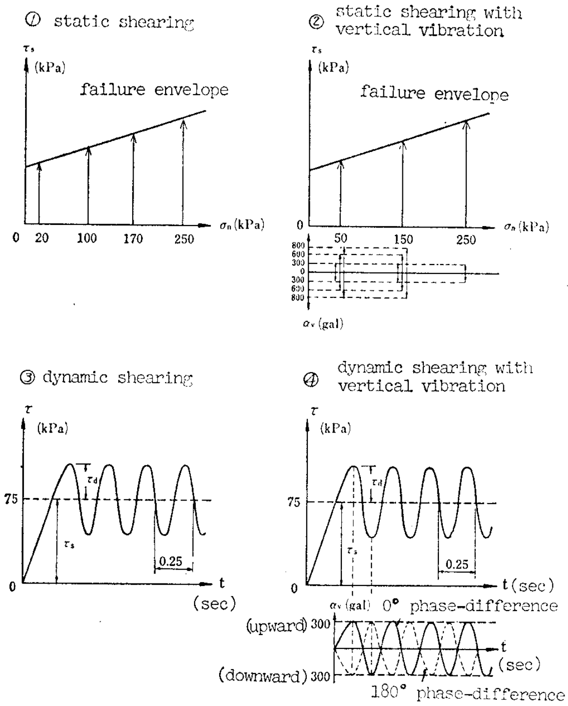

Fig. 5 Loading conditions

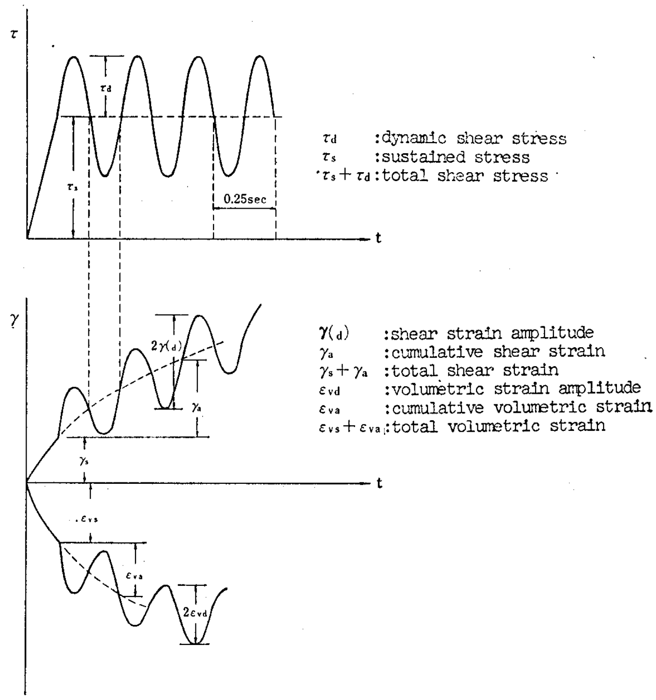

Fig. 6. Definitions of dynamic stresses and strains

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terms used in Table 2 are illustrated in Fig. 6. In order to investigate the effect of body force, the dynamic shear tests with vertical vibration were conducted at two different phase angles of 0° and 180° between the dynamic shear stress and the vertical vibration as shown in Fig. 5. In the case of 0° phase difference, the maximum shear stress is applied to the specimen at the lowest vertical position of the inner frame work where the maximum downward inertia force due to the maximum upward acceleration increases the interparticle confinement. On the other hand, in the case of 180° phase defference, the maximum shear stress is applied to the specimen at the highest vertical position where the maximum upward inertia force due to the maximum downward acceleration decreases the interparticle confinement.

TEST RESULTS

Results of Static Shear Tests with Vertical Vibration

Fig. 7 shows the comparison between the shear strengths with and without vertical vibrations. It is noted that the shear strengths are not decreased at least by the accelerations with amplitudes smaller than 800 gal. Fig. 8 shows the comparison between the stress-strain curves with and without vertical vibration. The shear tests with vibration were conducted under the overburden pressure of 150 kN/m², slightly smaller than 170 kN/m² used for the static shear tests. It is not seen that the deformation under vibration becomes larger than that under no vibration. It is also seen that the shear strain is somewhat smaller in the case of the static shear tests with vibration than that in the case of the static shear tests without vibration to a certain value of τ_s/σ_n. This is considered to be due to the difference of the overburden pressures and the initial void ratios in both cases.

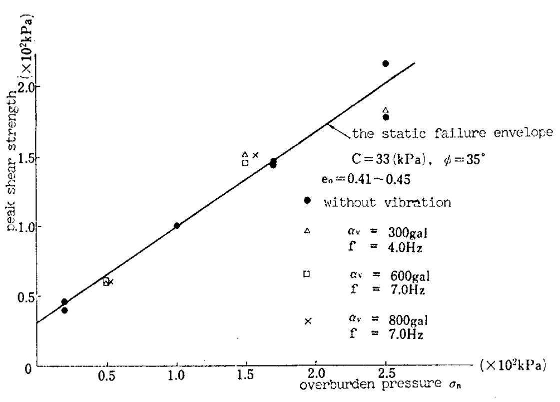

Fig. 7. Shear strengths with and without vertical vibration

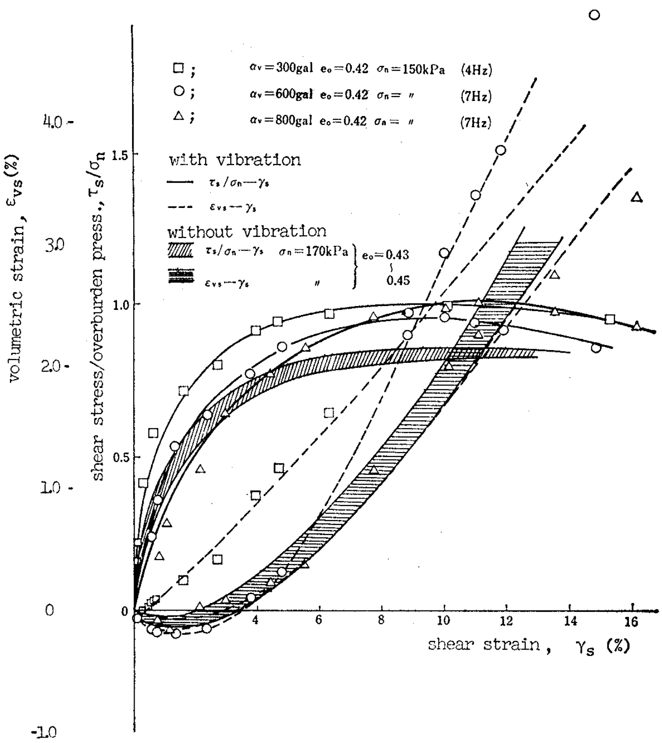

Fig. 8. Stress-strain curves with and without vertical vibration

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Results of Dynamic Shear Tests

a) Dynamic deformation behaviours and ultimate dynamic strength

Hereon, the ultimate dynamic failure is defined as the limiting state in which a specimen can not bear a further increase of shear stress in the same way as the static failure. Therefore, according to the above definition, it can be supposed that however large the shear deformation of a specimen may be, the specimen does not reach the failure so long as the shear stress continues to increase further.

Fig. 9 shows the relationship among the total shear strain, $(\gamma_s+\gamma_a)$, the total volumetric strain, $(\varepsilon_{vs}+\varepsilon_{va})$, and the stress ratio, $(\tau_s+\tau_d)/\sigma_n$, at N=2 000 cycles.

The definitions of these parameters are given in Fig. 6.

The followings are noted from Fig. 9.

(1) The specimens do not fail even if the stress ratio, $(\tau_s+\tau_d)/\sigma_n$, exceeds the static stress ratio at failure, τ_{sf}/σ_n.

(2) The total volumetric strains, $(\varepsilon_{vs}+\varepsilon_{va})$, show only contraction at any level of the stress ratio, $(\tau_s+\tau_d)/\sigma_n$.

(3) An amount of the total shear strain, $(\gamma_s+\gamma_a)$, increses with an increase of the stress ratio, $(\tau_s+\tau_d)/\sigma_n$.

(4) The total volumetric strains show a larger contraction with an increase of the total shear strains.

Fig. 10 shows the relationship between the shear strain ratio, $(\gamma_s+\gamma_a)/\gamma_{sf}$, and the shear stress ratio, $(\tau_s+\tau_d)/\tau_{sf}$, where γ_{sf} and τ_{sf} show the shear strain and the shear stress at failure in a static shear test, respectively. It is seen that the shear deformation increases remarkably under dynamic loading compared to that under static loading, and at the same time, the resistance of the material increases as indicated above. In fact, the specimen does not fail at $(\tau_s+\tau_d)/\tau_{sf}>1.2$ in Fig. 10.

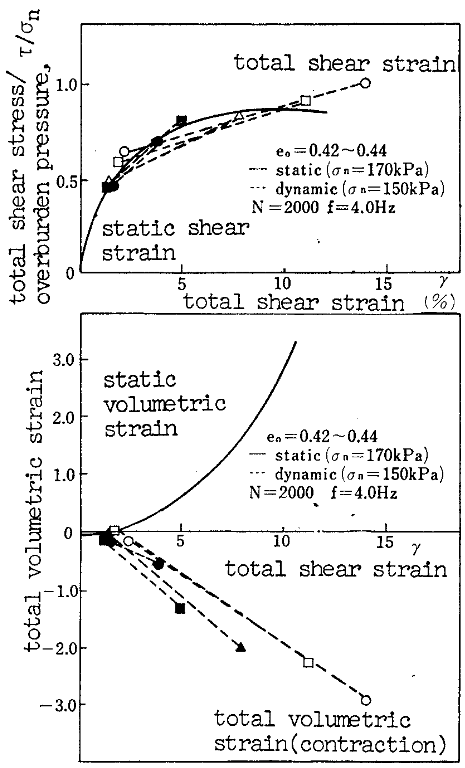

Fig. 9. Dynamic deformation behaviours in the dynamic shear tests

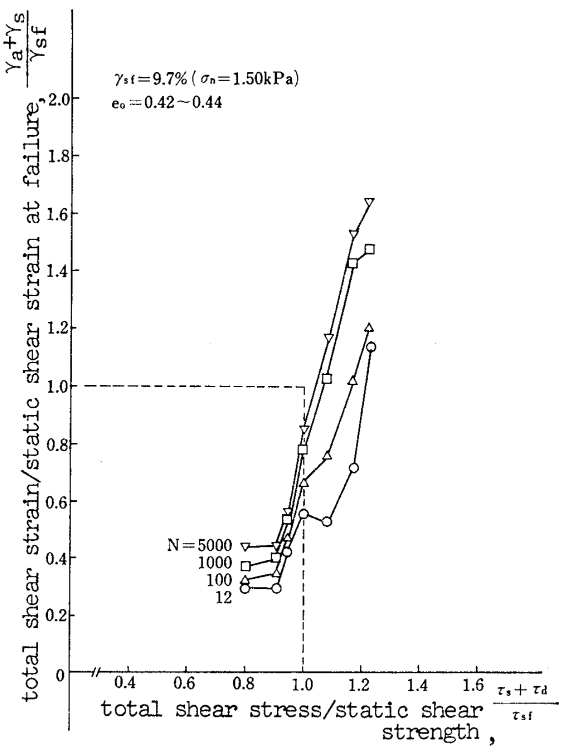

Fig. 10. Dynamic shear deformation behaviours

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In the present dynamic shear tests, the ultimate dynamic failure was not recognized, because the capacity of the shear loading actuator was not adequate to overcome the ultimate dynamic strength at $\sigma_n=150\,\mathrm{kN/m^2}$.

However, it is seen from the above results that the ultmate dynamic strength is at least larger than 1.2 times the shear strength under the intitial stress condition given, and the total shear strain becomes larger than 1.6 times the static shear strain at failure near the ultimate dynamic strength.

b) Yield stress ratio and dynamic deformation behaviors

Fig. 11 shows the relation between $(\gamma_s+\gamma_a)$ and $(\tau_s+\tau_d)/\sigma_n$, and Fig. 12 shows the relationship between $(\varepsilon_{vs}+\varepsilon_{va})$ and $(\tau_s+\tau_d)/\sigma_n$.

It is noted from the figures that the both total shear strain and the total volumetric strain increase abruptly at a critical value of $(\tau_s+\tau_d)/\sigma_n \fallingdotseq 0.75$.

At this critical stress ratio, $(\tau_s+\tau_d)$ is almost equal to $0.9\,\tau_{sf}$ as shown in Figs. 11 and 12. In order to clarify the mechanical meaning of this critical stress ratio, the static shear tests on the specimens prepared at similar initial void ratios, e_0, were conducted under $\sigma_n=170\,\mathrm{kN/m^2}$. Shear stress, τ_s, was increased by stages with a constant increment of $\Delta\tau_s=7.5\,\mathrm{kN/m^2}$. The shear strain, γ_s, and its increment, $\Delta\gamma_s$, which corresponds to $\Delta\tau_s$, are shown for the stress ratio, τ_s/σ_n, in Fig. 13. The incremental shear strain, $\Delta\gamma_s$ increases abruptly at about $\tau_s/\sigma_n=0.75$. This value agrees with the critical stress ratio indicated above. Furthermore, the stress-strain curves begin to bend remarkably at about this stress ratio as shown in Fig. 13. Accordingly, it may be possible to say that a kind of yielding of the material occurs at the critical stress ratio. In this sense, this critical stress ratio will be called as the yield stress ratio, $(\tau_s/\sigma_n)_y$, henceforth.

Results of Dynamic Shear Tests with Vertical Vibration

The dynamic shear tests with vertical vibration were conducted under two phase

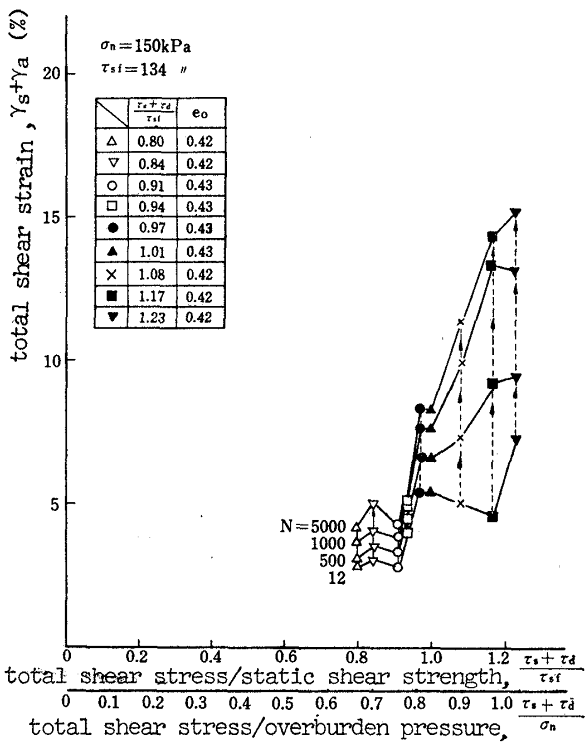

Fig. 11. Cumulative shear strains and dynamic stress ratio

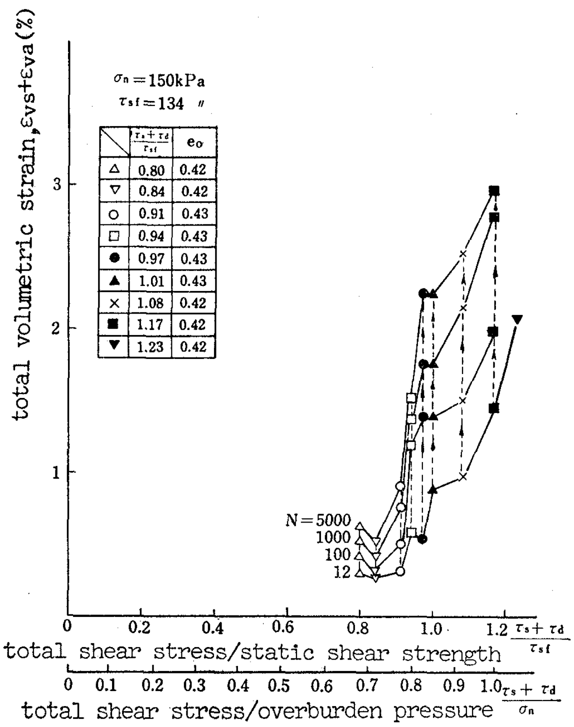

Fig. 12. Cumulative volumetric strains and dynamic stress ratio

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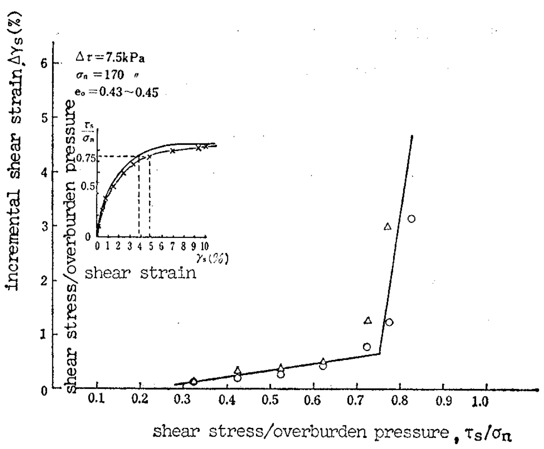

Fig. 13. **Yield stress ratio**

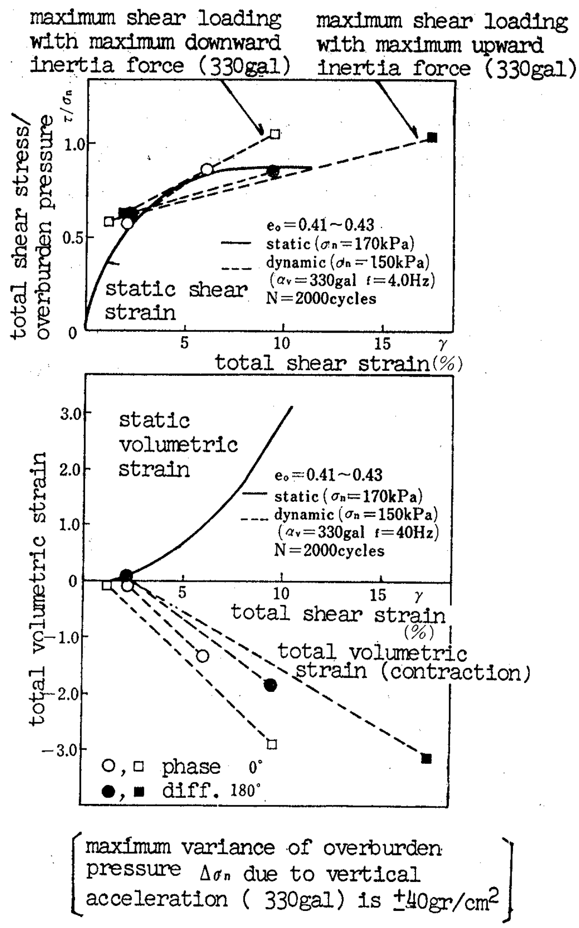

Fig. 14. **Dynamic deformation behaviours in the dynamic shear tests during vertical vibration**

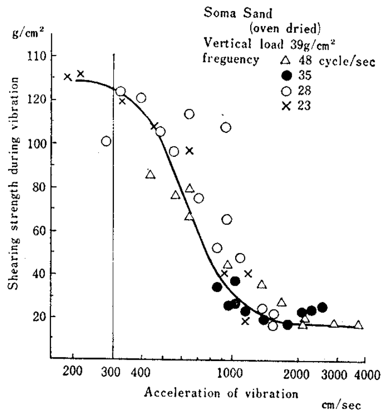

Fig. 15. **Shear strength during vertical vibration (Mogami and Kubo, 1953)**

differences between the dynamic shear loading and the vertical vibration: 0° and 180°.

In both cases, the vertical acceleration, α_v, and the frequency, f, were set at 330 gal and 4 Hz, and the overburden pressure, σ_n, at 150 kN/m². Fig. 14 shows the relationship among $(\gamma_s+\gamma_a)$, $(\varepsilon_{vs}+\varepsilon_{va})$ and $(\tau_s+\tau_d)/\sigma_n$ in the same way as shown in Fig. 9.

It is seen from the figure that the total shear strain $(\gamma_s+\gamma_a)$ is remarkably influenced by the phase difference. That is, the total shear strain is about two times larger in the case of 180° phase difference than in the case of 0° phase difference.

CONSIDERATIONS AND DISCUSSIONS

Effect of Body Force under Monotonously Increasing Stress Condition

Fig. 15 shows the relationship between vertical acceleration and shear strength of a dry sand tested in a box shear apparatus with vertical vibration, conducted by Mogami and Kubo (1953).

According to the figure, the shear strength decreases abruptly when the vertical acceleration exceeds 300 gal, and loses most of its strength near 1 g. However, it should be noted that these results are obtained under the small overburden pressure and very high frequencies as indicated in Fig. 15. In fact, the shear strength does not decrease and the deformation behaviours are not affected by the vertical acceleration until the

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acceleration reaches 800 gal, even if the overburden pressure exceeding 50 kN/m^2 is applied to the specimen as shown in Figs. 7 and 8.

That is, in the case of the rockfill material compacted sufficiently, its static shear strength and deformation behaviour are not significantly affected by the vertical vibration with the peak accelerations less than about 800 gal and the frequencies less than about 10 Hz.

Further considerations about the cause of the above result will be made in the following paragraph.

Effect of Body Force under Cyclic Loading Condition

In the dynamic shear tests during vertical vibration, the deformation behaviours of the rockfill material have been significantly influenced by the phase difference between the vertical vibration and the dynamic shear load as shown in Fig. 14. It seems that the deformation curve at the time of no vibration is between those with 0° and 180° phase differences. The reason for the effect of the phase difference in this way will be discussed below. The mechanical influences of an earthquake on the soil element are mainly dynamic stress and body force as shown in Fig. 1. Let us first pay attention to the dynamic stress. The dynamic overburden pressure should be considered primarily in this case because of veritical vibration. At the vertical acceleration of ± 330 gal, the maximum variance of the dynamic overburden pressure, $\Delta\sigma_n$, acting on the top and bottom surfaces of the specimen is about ± 4 kN/m^2 if the inertia force acting on both the loading plate and the specimen is taken into account. This variance is negligibly small compared to the applied constant overburden pressure of 150 kN/m^2. As a result, the same maximum values of the total shear stress, $(\tau_s+\tau_d)$, are applied to the specimen at $\sigma_n=154$ kN/m^2 in the case of the phase difference of 0°, and at $\sigma_n=146$ kN/m^2 in the case of 180°.

It can be hardly considered possible that this small variance of overburden pressure affects the deformation behaviour so significantly as shown in Fig. 14. This suggests that the mechanical factor causing the effect of phase-difference may be the body force; that is, acceleration itself as previously discussed. As the granular material is composed of a large number of grains in contact with one another, the shear resistance develops from the interparticle friction at points of contact and the interlocking between grains. Accordingly, the interparticle forces at contacts fluctuates when the body force acting on each grain changes. In fact, Okamoto and Hakuno (1964) conducted a unique vibration test with sand and showed that the average value of interparticle contact forces of sand grains fluctuated considerably during vertical and horizontal vibrations. This result shows the likelihood that the shear resistance is significantly affected by the body force. Consider the influence of vertical acceleration on the interparticle force on the basis of a simple model composed of grains as shown in Fig. 16. Interparticle force F_n is supposed to be a constant. Let us pay attention to the grain with the mass of m. If grain m is in static equilibrium under vertical confining

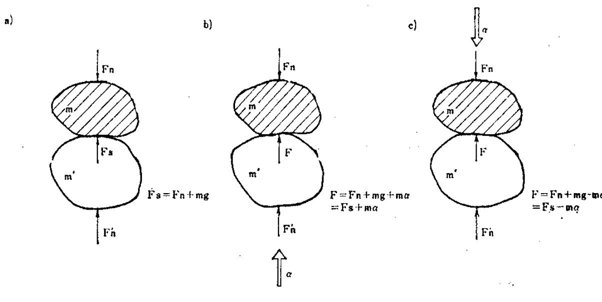

Fig. 16. Effect of body force on interparticle contact force
a) in static equilibrium, b) in motion with upward acceleration (in case of 0°), c) in motion with downward acceleration (in case of 180°)

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forces, F_n, and, F_s, as shown in Fig. 16(a), the interparticle force, F_s, between grains m and m' becomes:

$$F_s = F_n + mg$$

If the grains are in motion with upward acceleration, α, under the same confining condition as shown in Fig. 16(b), the interparticle force, F, between grains m and m' **becomes:**

$$F = F_n + mg + m\alpha = F_s + m\alpha$$

That is, the interparticle force, F, is increased by $m\alpha$ due to the downward inertia force acting in an opposite direction of α.

As a result, the confinement between the grains m and m' is strengthened. In a similar way, the interparticle force, F, becomes $F = F_n + mg - m\alpha = F_s - m\alpha$ when the grains are in motion with downward acceleration, α, as shown in Fig. 16(c).

In this case, the interparticle force, F, decreases by $m\alpha$ due to the upward inertia force, and the confinement between the grains m and m' is reduced. Considering the correspondence between the above consideration and the dynamic shear tests with vertical vibration, the specimen may be regarded as being subjected to the maximum total shear stress in the state of Fig. 16(b) in the case of the phase difference of 0°, and in the state of Fig. 16(c) in the case of the phase difference 180°. If these effects of body force is born in mind, it is recognized from Fig. 14 that the shear deformation becomes about two times larger when the upward body force loosens interparticle forces (180°) compared when the downward body force strengthens interparticle forces (0°).

On the other hand, the influence of vertical acceleration on the deformation and strength is not significant in the static shear tests with vertical vibration (Fig. 7 and 8).

This may be because the effect of body force is rather delicate. That is, monotonously increasing load increases the interparticle confining forces and strengthens the confinement between grains.

Thus, a delicate effect of body force on interparticle forces may not have been exhibited. In the dynamic shear tests, cyclic load disturbs grain-structure, and fluctuates the interparticle confining forces at points of contact. Therefore, the body force has considerably influenced on the deformation behaviours.

The effect of body force has been reported previously in the field of dynamic compaction of sand. Tokue (1976) conducted the cyclic simple shear tests of sand under two vibrating conditions: with and without horizontal vibration. It was shown that the amount of deformation caused by the test with the vibration was greater than the deformation caused without vibration under the same stress condition.

From all the above considerations, it is reasonable to state that the body force affects the influence of the effectiveness of the phase difference between vertical vibration and dynamic shear load on the deformation behaviours of the rockfill material.

Ultimate Dynamic Strength

The results of the dynamic triaxial shear tests on sand are shown in Fig. 17 in the same manner as Fig. 9 (Timmerman and Wu, 1969).

It is seen from the figure that a cumulative volumetric strain due to cyclic loading varies from contraction to dilatation when the sustained stress ratio exceeds the static stress ratio σ_1/σ_3 at the minimum volume during static shear.

In the case of Fig. 9, the sustained stress ratio, τ_s/σ_n, is about 0.5, and this is close to the stress ratio at the minimum volume during static shear. Thus, it may be reasonable to state from the above result that the total volumetric strain shows only

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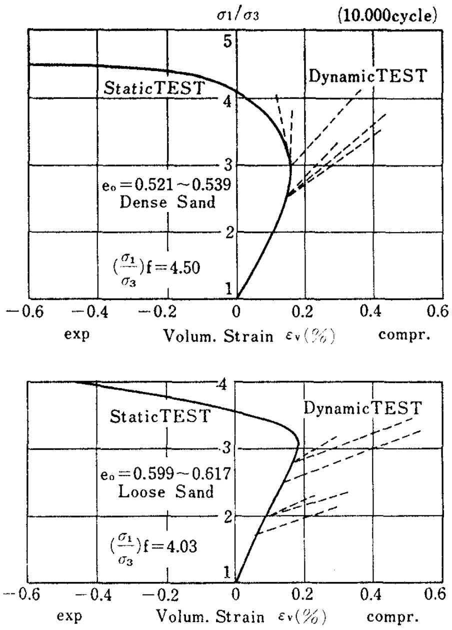

Fig. 17. Influence of sustained stress on the dynamic deformation (Timmerman and Wu, 1969)

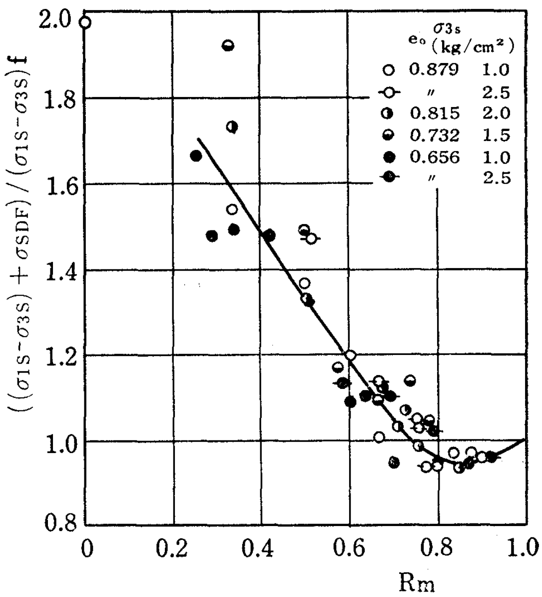

Fig. 18. "Dynamic strength" of dry at ε_1 =15% (Toki and Kitago, 1974)

contraction however large the dynamic shear stress amplitude, τ_d, may be.

If a comparison is made between Figs. 9 and 17, the cumulative volumetric strain of the rockfill material becomes about five times larger than that of loose sand, although in the case of the rockfill material the specimen becomes dense by vibratory compaction and the number of loading cycles is 2000 cycles fewer than 10000 cycles in the case of sand. This suggests that the rockfill material shows a larger amount of contraction than sand by cyclic loading. This may partly results from grain breakage. Fig. 18 shows the relationship between the stress ratio, $((\sigma_{1s}-\sigma_{3s})+\sigma_{SDF})/(\sigma_{1s}-\sigma_{3s})_f$, and the parameter $R_m=(\sigma_{1s}-\sigma_{3s})/(\sigma_{1s}-\sigma_{3s})_f$ of the cyclic triaxial tests of a dry sand conducted by Toki and Kitago (1974). $(\sigma_{1s}-\sigma_{3s})$ and $(\sigma_{1s}-\sigma_{3s})_f$ show the sustained and peak deviator stresses applied statically. Therefore, R_m corresponds to τ_s/τ_{sf} in the present experiment. σ_{SDF} shows the double amplitudes of the cyclic principal stresses, σ_1 and σ_3, with the phase difference of 180° and the same amplitude at the axial strain of 15%. Therefore, $(\sigma_{1s}-\sigma_{3s})+\sigma_{SDF}$ is the deviator stress required to reach the axial strain of 15%. It is seen from the figure that the stress, $(\sigma_{1s}-\sigma_{3s})+\sigma_{SDF}$, is much larger than the static strength, $(\sigma_{1s}-\sigma_{3s})_f$, in a small range of R_m, but decreases with an increase of R_m.

Thus, a kind of "dynamic strength", $(\sigma_{1s}-\sigma_{3s})+\sigma_{SDF}$, decreases as the sustained stress, $(\sigma_{1s}-\sigma_{3s})$, increases. This is considered to be correlated deeply with the dynamic dilatancy behaviours described above. That is, it may be possible to say that the ultimate dynamic strength of dry sand increases more than the static shear strength in the range of the sustained stress under which contraction occurs, and the ultimate dynamic strength decreases in the range of the sustained stress under which dilatation occurs. Accordingly, it is understood that the increase of strength of the rockfill material indicated in Fig. 9 consists with the trend of sand indicated above. Moreover, an amount of increase in the ultimate dynamic strength is suggested to become larger in rockfill material than in sand considering that an amount of contraction becomes larger in rockfill material than in sand.

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Yield Stress Ratio and Dynamic Deformation Behaviours

The ultimate dynamic strength represents only an ultimate state of the material, and is not always a proper criterion for the dynamic failure of actual soil structures. For example, there may be some cases problematical to the stability of earth-structures that the total shear strains in some parts of earth-structures reach 15% as shown in Fig. 10 even if the ultimate dynamic failure does not occur in these parts.

Therefore, it is requred to examine the dynamic deformation behaviour before the ultimate dynamic failure.

Paying attention to the dynamic deformation behaviour, the following findings in Figs. 9~13 are worthy of note:

(1) The total strains, $(\gamma_s+\gamma_a)$ and $(\varepsilon_{vs}+\varepsilon_{va})$, increase abruptly when the stress ratio, $(\tau_s+\tau_d)/\sigma_n$, exceeds the yield stress ratio, $(\tau_s/\sigma_n)_y$.

(2) The yield stress ratio, $(\tau_s/\sigma_n)_y$, is defined as the stress ratio at which the incremental shear strain, $\Delta\gamma_s$, increases abruptly, where the stress-strain curve begins to bend remarkably.

These findings have already been reported in the case of sand by Tokue (1979). Fig. 19 shows the relationship between the cyclic stress ratio, τ/p, and the change of void ratio, Δe, of the cyclic simple shear tests on sand under the application of a constant overburden pressure, p, and a constant shear stress amplitude, τ.

As the sustained stress was not applied, the shear stress was in a perfectly reversed state. It is seen in Fig. 19 that the change of void ratio, Δe, increases sharply about $\tau/p=0.32$, as the similar trend has been shown in Fig. 12. Fig. 20 shows the relationship between $\Delta\gamma$ and τ/p of the static shear tests of sand in a similar manner as shown in Fig. 13. The incremental shear strain, $\Delta\gamma$, increases sharply about $\tau/p=0.32$ in consistence with the cyclic stress ratio at an abrupt increase of Δe in Fig. 19. Moreover, the agreement between the stress ratios where Δe and $\Delta\gamma$ take abrupt changes is confirmed in a broad range of an initial void ratio under the overburden pressure of 50 to 200 kN/m^2 as shown in Fig. 21. It is seen from the figure that this "critical stress ratio", $(\tau/p)_c$, decreases with an increase of initial void ratio. Furthermore, it was shown with the static shear tests using aluminium rods that the structure of rods were disturbed remarkably more than the critical stress ratio, $(\tau/p)_c$. Tokue (1979) gave a mechanical definition to the critical stress ratio and named it the stress ratio of critical disturbance by paying attention to the variation of the grain structure during cyclic shear on the basis of a two-dimensional stress-dilatancy model.

The dynamic deformation behaviours of sand described above are quite similar to those

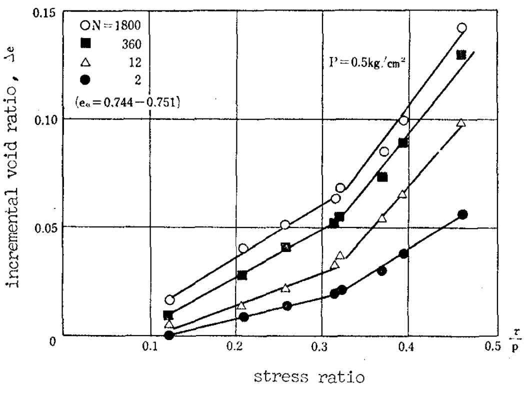

Fig. 19. Volume change of dry sand under cyclic loading in simple shear test (Tokue, 1979)

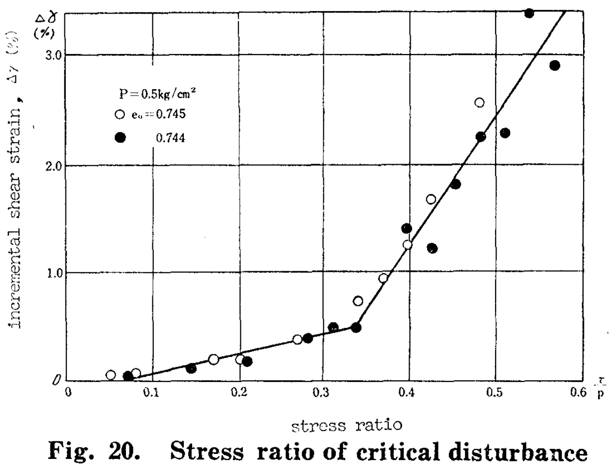

Fig. 20. Stress ratio of critical disturbance (Tokue, 1979)

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of the rockfill material. Accordingly, the yield stress ratio, $(\tau_s/\sigma_n)_y$, defined above agrees with the stress ratio of critical disturbance, $(\tau/p)_c$.

From all the above considerations, the following, summarization may be possible to the dynamic deformation characteristics of granular materials:

(1) The yield stress ratio less than the peak stress ratio exists. The granular material is in an unstable state when it is subjected to the dynamic loads exceeding the yield stress ratio, and the material undergoes and possesses a large deformation and a disturbed grain-structure.

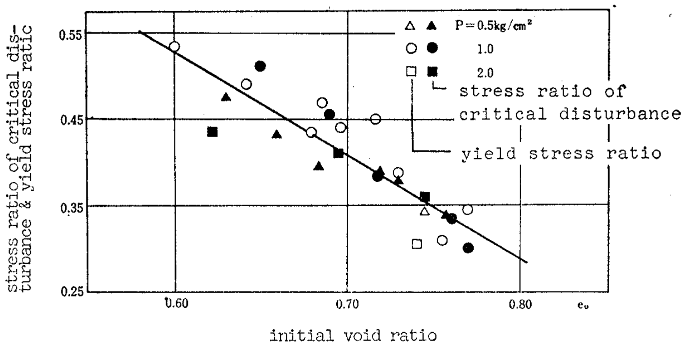

Fig. 21. Relation between yield stress ratio and stress ratio of critical disturbance (Tokue, 1979)

(2) The yield stress ratio depends on an initial void ratio. The denser the specimen becomes, the larger the yield stress ratio also becomes. In the case of sand, the following relationship is expressed about the maximum value of the yield stress ratio (Tokue, 1979);

$$\max\left(\frac{\tau_s}{\sigma_n}\right)_y \doteqdot \left(\frac{\tau_s}{\sigma_n}\right)_{dv=0} = \tan\phi_\mu$$

where max $(\tau_s/\sigma_n)_y$ is obtained at the minimum void ratio, and $(\tau_s/\sigma_n)_{dv=0}$ shows the stress ratio at the minimum volume during static shear. ϕ_μ is the interparticle friction angle of sand.

Considering the effect of body force indicated in the foregoing paragraph, the material subjected to the cyclic load more than the yield stress ratio may be much more unstable if the material receives significant body force at the same time. The above results suggest that the yield stress ratio becomes an important criterion for the dynamic stability of earth-structures.

CONCLUDING REMARKS

Considering a strong random earthquake, its mechanical effect on earth structures may be characterized by both large dynamic stress and large vertical and horizontal body forces (accelerations). The followings are concluded from the static and dynamic simple shear tests of the rockfill material under two vibrating conditions with and without vertical vibrations:

a) The influence of body force

(1) From the tests in which both vertical acceleration (body force) of 330 gal and dynamic shear stress have been applied to the rockfill material, it is noted that the shear and volumetric deformations become larger when the body force loosens the interparticle confinement compared when the body force strengthens the interparticle confinement under the same stress condition. That is, the body force has a large influence on the dynamic deformation behaviours because the cyclic loading disturbs the grain strcture and the interparticle confinement.

(2) Furthermore, the degree of influence of the body force is affected considerably by the interparticle confining condition. In the static shear tests of the rockfill material during vertical vibration, a monotonously increasing loading strengthens the interparticle confinement. As a result, the strength and deformation behaviour are not varied by the

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vibration at least until the peak acceleration becomes 800 gal and the frequency exceeds 10 Hz.

b) The ultimate dynamic strength

If the ultimate dynamic strength is defined as the maximum shear stress by which the specimen is failed in the same way as the static shear strength, the ultimate dynamic strength of the rockfill material may be dependent on a sustained stress in the same way as those of sand and clay. When the sustained stress is within the range in which zero or negative dilatancy occurs, the volume of a specimen is contracted by cyclic loadings, and as a result, the ultimate dynamic strength becomes larger than the static shear strength. An amount of contraction becomes larger with rockfill material compared with sand due to the grain breakage.

c) The yield stress ratio and the dynamic deformation

The granular materials are in an unstable state when they are subjected to the dynamic loads exceeding the yield stress ratio, and the materials undergo and possess large deformations and disturbed grain structures. The yield stress ratio can be obtained from the static shear tests. In the above sense, the yield stress ratio is an important criterion for the dynamic stability of earth structures. All these results show that when aseismatic design of earth structure is being made, the effects of body force and deformation behaviour must be taken into account in order to determine the dynamic strength of the material.

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NOTATION

c=cohesion
e_0=initial void ratio
f=frequency of cyclic load and vertical vibration
g=gravity force
N=number of cycles
t=time (second)
α_v=vertical vibratory acceleration
γ_a=cumulative shear strain due to cyclic loading
γ_d=amplitude of shear strain
γ_s=static shear strain
γ_{sf}=static shear strain at failure
$\Delta\gamma_s$=increment of static shear strain
ε_{va}=cumulative votumetric strain due to cyclic loading
ε_{vd}=amplitude of volumatric strain
ε_{vs}=static volumetric strain
θ=phase difference between dynamic shear load and vertical vibration
σ_n=overburden pressure
τ_d=amplitude of dynamic shear stress
τ_s=static shear stress (sustained shear stress)

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τ_{sf}=static shear stress at failure
$(\tau_s/\sigma_n)_y$=yield stress ratio
ϕ=internal friction angle

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