SHEAR CHARACTERISTICS OF COMPACTED PARTIALLY SATURATED SOILS

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SYNOPSIS

Earth structures such as an embankment made artificially by compaction are in partial saturation. The shear strength of the embankment which is constructed with a constant dry density is dependent mainly on the water content. Therefore, it is necessary to investigate by experiments the influence of void ratio and degree of saturation on the shear strength of compacted soils.

Three series of triaxial compression tests were performed with different drainage conditions, and the pore pressure of partially saturated soils was measured by an earth pressure gauge designed by the authors. Samples used were two kinds of "Masa"-soil (decomposed granite, sandy loam). Specimens with various void ratios and degrees of saturation were made by compaction.

Test results indicate that the shearing resistance decreases and pore pressure increases with increase in the degree of saturation and void ratio. Therefore, when an embankment is constructed with the sandy soils, protection of slope and sufficient compaction are necessary.

TEST PROCEDURE

Samples used are two types of "Masa"-soil which exist in Fukuoka city, Kyushu, Japan. These samples are named tentatively "Kanakuma Masa"-soil and "Kanayama Masa"-soil. The grain size accumulation curves and the soil properties are shown in Fig. 1 and Table 1, respectively. Since these two kinds of samples indicate similar nature, the test results which had been obtained with different drainage conditions using these samples could be compared.

Distilled water was added to completely air dried samples (less than 4.76 mm) to obtain predetermined water contents. Each sample was cured for 24 hours, and put in a steel cylinder of 5.00 cm in diameter and 12.35 cm in height, and compacted to a specified void ratio and degree of saturation. The specimen was extracted from the steel cylinder, and its weight and

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Table 1. Soil Properties of Sample

Sample		Kanakuma Masa-Soil	Kanayama Masa-Soil
Specific Gravity		2.65	2.63
w_L	(%)	39	36
w_P	(%)	29	31
I_P	(%)	10	5
w_{opt}	(%)	16.0	15.0
$\gamma_{d\ max}$	(g/cm³)	1.730	1.780
Grain Size Distribution (%)	Gravel	8.4	8.1
	Sand	57.6	63.1
	Silt	26.0	21.3
	Clay	8.0	7.5
Classification by the Triangular Diagram		Sandy Loam	Sandy Loam

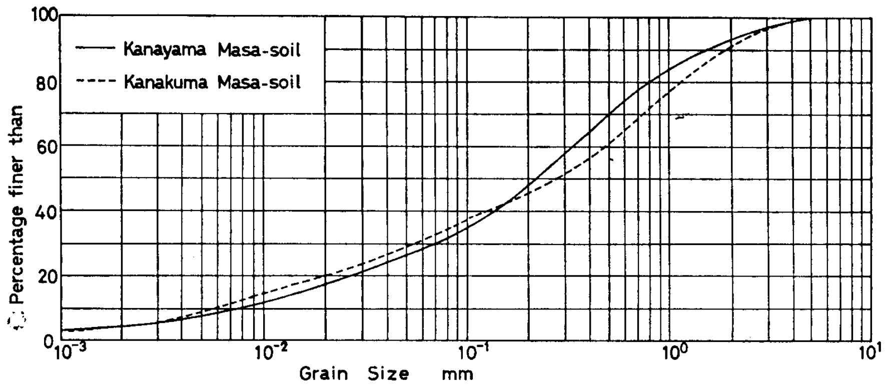

Fig. 1. Grain Size Accumulation Curve

volume were determined. The triaxial compression tests performed were of three types hereinafter denoted Tests 1 to 3. In Test 2 and Test 3, the average of the water contents before and after the triaxial compression test was used, and in Test 1 the water content was measured after the test. In all the tests, the rate of axial strain was 0.8 to 0.9 per cent per minute.

Test 1:—This test series was undrained triaxial compression test, and the sample was "Kanakuma Masa"-soil. The tests were carried out at several sets of void ratios and degrees of saturation. The desired void ratios were 1.140, 0.870, 0.660 and 0.530. These values correspond to dry densities of 72, 82, 93, and 100% of the maximum dry density in the standard compactoin test. For these four void ratios, specimens which had degrees of saturation from 40% to 90% were prepared. Specimens of larger void ratios and degrees of saturation could not be prepared.

After the specimens were set in the triaxial apparatus, the shear tests were conducted. Since a volume shrinkage of the specimen was noticed instantly after application of chamber pressure, this chamber pressure was decreased to zero after five minutes. The diameter and the height of the specimen were measured again from outside of the rubber sleeve. From these dimensions, the void ratio and the degree of saturation were obtained, and were considered initial values. The method was used because there was no suitable apparatus to measure volume change in undrained triaxial compression tests on partially saturated soils at that time. The increase in volume of the specimen due to decrease in the chamber pressure to zero was neglected. Though thist est was an undrained triaxial test, as the specimen was partially saturated there would be some change in volume with an increase in strain during the shear test. However, calculations were done with these test results assuming that there was no change in the volume of the specimen. Chamber pressures (σ_3) of 0.2, 0.5, 0.8, 1.1, 1.5, 2.0, and 2.5 kg/cm^2 were used in this test series.

Test 2:—This test series was drained triaxial compression test, and the sample was "Kanakuma Masa"-soil. The desired void ratios and degrees of saturation were the same as for Test 1. Chamber pressures (σ_3) of 0, 0.25, 0.50, 1.00, and 2.00 kg/cm^2 were used.

For investigating the volume shrinkage and the volume expansion of a specimen by application and reduction of chamber pressure respectively, the specimen was consolidated for 1 hour under a constant chamber pressure and was allowed to expand for 30 minutes by decreasing the chamber pressure to zero. The diameter and height of the specimen were then measured. Void ratio and degree of saturation which were obtained from this measurement were considered initial values. After applying the chamber pressure again for 1 hour, shear tests were conducted. The method used for measuring the change in volume of the specimen was that proposed by Hoshino. The apparatus used is shown in Fig. 2. If the volume of a specimen changes, the air in the specimen influences the water level in the burette that is connected to the specimen by a tube through a bronze base. The change in volume of the specimen could be determined by measuring the distance which the burette has to be raised or lowered to bring the water level to $a-a'$, and multiplying it by the cross sectional area of the burette.

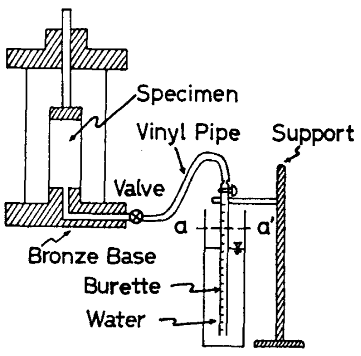

Fig. 2. Apparatus for Measuring Volume Change

Test 3:—This test series was undrained triaxial compression test, and the sample was "Kanayama Masa"-soil. The desired void ratios were 0.850, 0.640, and 0.480 which corresponded to the dry densities of 80, 90, and 100% of the maximum dry density in the standard compaction test. For these void ratios, degrees of saturation were 0, 40, 60, 80, and 100% with a few exceptions. Chamber pressures (σ_3) of 0.5, 1.0, 1.5, and 2.0 kg/cm^2 were used. The volume of the specimen was measured from outside the rubber sleeve after setting it in the cell. The void ratio and the degree of saturation which were calculated from this volume were considered initial values.

The change in volume was measured by the apparatus which was designed by A. W. Bishop and D. J. Henkel (1957). Generally, it is said that pore air pressure and pore water pressure must be measured separately in relation to the pore pressure of partially saturated soils. But the pore pressure was measured with an earth pressure gauge without considering the difference between the pore air pressure and the pore water pressure. The earth pressure gauge was a wire strain gauge type of 50 mm in outside diameter, 27 mm in height, the diameter of its diaphragm was 40 mm, and the capacity of the gauge was $-1\sim+3$ kg/cm^2. This gauge was fixed in the bronze base of the triaxial cell, and the pore pressure was measured through a porous metal under the bottom of the specimen. In this paper, detailed explanations about the apparatus for measuring the change in volume and the earth pressure gauge (I. Uchida, R. Matsumoto, K. Onitsuka and T. Tanaka, 1967) are omitted. After applying the chamber pressure for 30$\sim$60 minutes, the normal axial load was applied.

VOLUME CHANGE

The change in volume was measured in both Test 2 and Test 3. Since Test 2 is drained triaxial test, correction was made for the dilatancy effect as follows:

$$(\sigma_1-\sigma_3)_f = (\sigma_1-\sigma_3)_{rf}+\sigma_3\cdot\frac{d\left(\frac{\Delta V}{V}\right)}{d\varepsilon}$$

where $(\sigma_1-\sigma_3)_f$: measured maximum deviator stress

$(\sigma_1-\sigma_3)_{rf}$: deviator stress required to overcome the cohesive resistance and the frictional resistance of the soil

σ_3: chamber pressure

$d\left(\frac{\Delta V}{V}\right)\Big/d\varepsilon$: increase in the rate of change of volume divided by the change in unit shear strain

Akai has given a tentative name of Dilatancy Index (D.I.) (Akai, K., 1958) to the expression $d\left(\frac{\Delta V}{V}\right)\Big/d\varepsilon$ as a means of indicating the magnitude of dilatancy. The relationship between the dilatancy index at the maximum deviator stress and the degree of saturation is plotted in Figs. 3 to 5. Fig. 3 and Fig. 4 represent the results of Test 2, and Fig. 5 represents that of

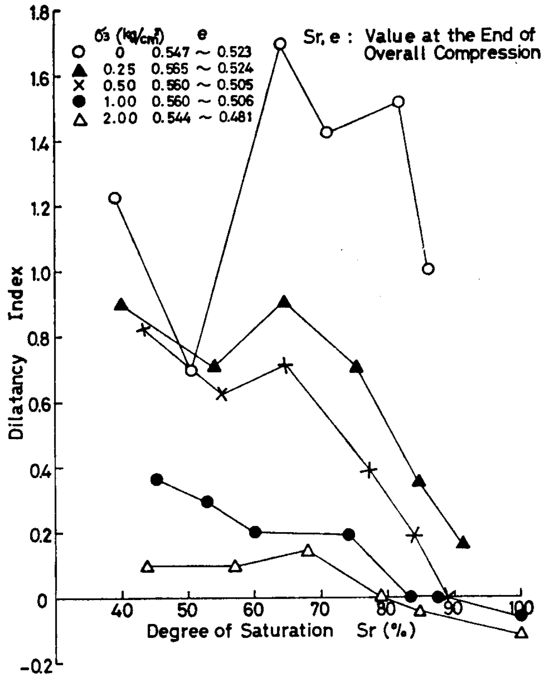

Fig. 3. Relation between Dilatancy Index at the Maximum Deviator Stress and Degree of Saturation in Test 2

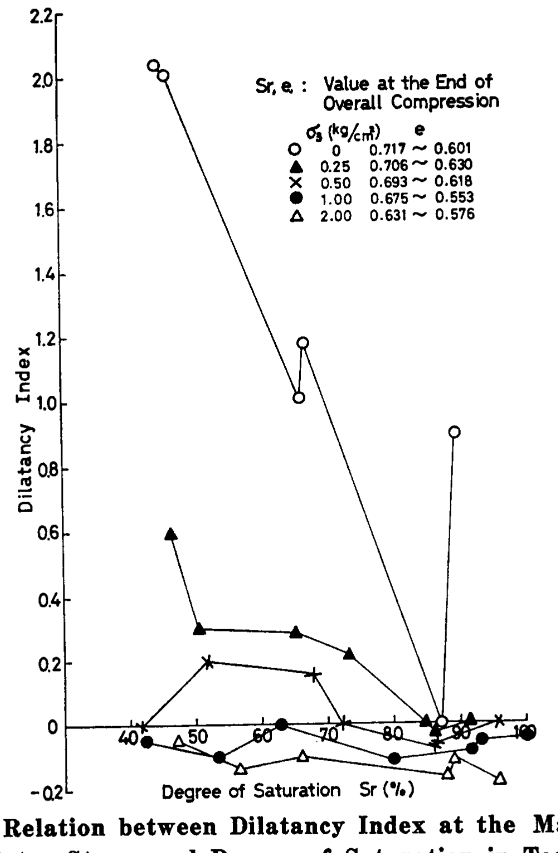

Fig. 4. Relation between Dilatancy Index at the Maximum Deviator Stress and Degree of Saturation in Test 2

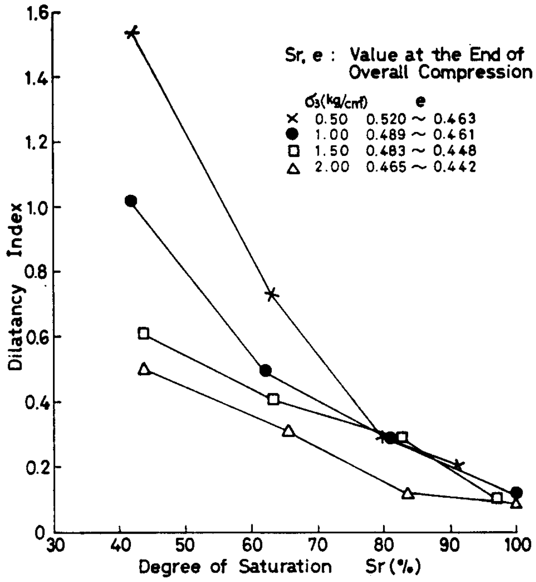

Fig. 5. Relation between Dilatancy Index at the Maximum Deviator Stress and Degree of Saturation in Test 3

Test 3.

The following characteristics are noted. When the void ratio of a specimen is small, the dilatancy index increases with the decrease in chamber pressure as already known for saturated sandy soils. However, when the void ratio increases, such tendencies become obscure. In specimens of small void ratio, the dilatancy index becomes smaller with the increase in degree of saturation. For large void ratios, the graphs in the figure become parallel to the base line, D.I. = 0, and there are no clear tendencies as above. With an increase in the chamber pressure, the dilatancy index becomes negative. When the dilatancy indices at the maximum deviator stress of samples in both tests that have the same void ratio and degree of saturation at the end of overall compression are compared, it is noted that the value in Test 2 (drained test) is slightly larger than that in Test 3 (undrained test).

STRESS AND STRAIN

In Fig. 6 and Fig. 7, the relationship between maximum deviator stress and strain is shown. Both the maximum deviator stress and the strain become larger with an increase in the chamber pressure regardless of the void ratio. In Test 1 and Test 3, though graphs are not drawn, the gradient of the stress-strain curve approaches zero with an increase in the degree of saturation in a specimen which has a large void ratio. The strain value at the maximum deviator stress does not seem to be affected by the value of chamber pressure. For small void ratios, however, the stress-strain curve inclines considerably towards the maximum deviator stress axis even if the

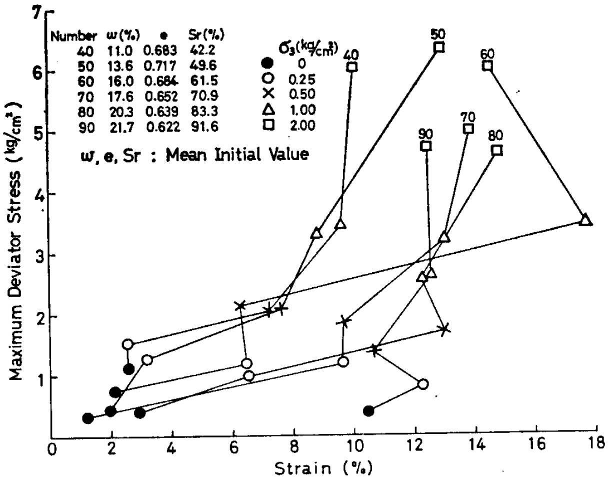

Fig. 6. Relation between Maximum Deviator Stress and Strain in Test 2

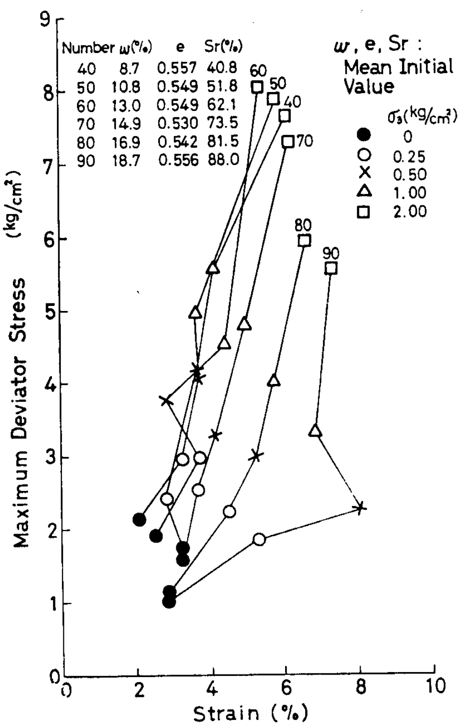

Fig. 7. Relation between Maximum Deviator Stress and Strain in Test 2

degree of saturation is about 90%.

In drained and undrained tests, the following observations were made. For a constant degree of saturation, the slope of the stress-strain curve becomes steeper and the strain decreases with a decrease in the void ratio. For the constant void ratio, the strain of a sample of high degree of saturation is larger than that of low degree of saturation.

PORE PRESSURE

Generally it is said that the pore air pressure and the pore water pressure must be measured separately, since the pore water pressure is smaller than the pore air pressure because of surface tension. However, the values which were measured by an earth pressure gauge at the bottom of a specimen were treated as the pore pressures. Regarding the values which were measured with the earth pressure gauge at the end of overall compression and at the time of maximum deviator stress, the following observation could be made. When the chamber pressure is low and the specimen has a small void ratio, the measured value is approximately equal to the pore air pressure which is calculated from the volume change of the specimen. With an increase in the chamber pressure and the degree of saturation, the measured and the claculated values become inconsistent. Errors in the measurement of change of volume and the existence of air between the specimen and the rubber sleeve, etc., are pointed out as causes for this inconsistency. Since a paper regarding the measurement of pore pressure in partially saturated soil (Uchida, et al., 1967.) was published by the authors earlier, detailed explanation is omitted here.

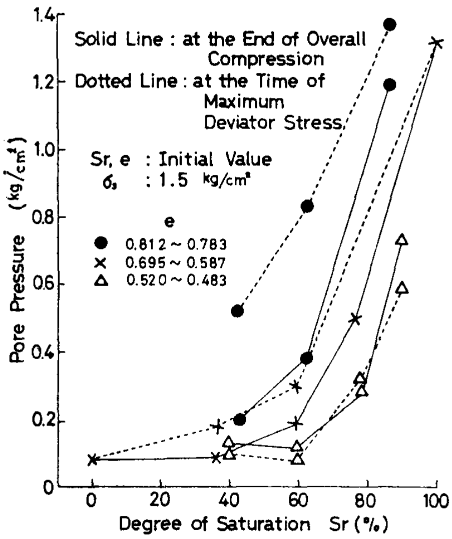

Fig. 8. Relation between Pore Pressure and Degree of Saturation

In Fig. 8, the relationship between the pore pressure and the degree of saturation is plotted. When the void ratio is large, the volume shrinkage of a specimen by overall compression and deviator stress is large. For a constant void ratio, when the degree of saturation is high, the volume of air is small. Consequently, in a sample of partially saturated soil, the pore pressure due to the compression of air increases with an increase in the void ratio and the degree of saturation.

SHEAR RESISTANCE

Table 2, Table 3 and Table 4 show the results of shear tests of Test 1, Test 2 and Test 3, respectively. Apparent cohesion and angle of shearing resistance in these tables were calculated from test results by means of the method of least squares. In Test 2, the results obtained with the chamber pressure $\sigma_3 = 0$ were excluded from these calculations. In Test 2, the apparent cohesion and the angle of shearing resistance were obtained from two values, namely, the measured value $(\sigma_1-\sigma_3)_f$ and the calculated value $(\sigma_1-\sigma_3)_{rf}$, in the equation for correcting the dilatancy effect. According to

Table 2. Results of Shear Test, Test 1

Test Number	Initial Value			Apparent Cohesion Cd (kg/cm²)		Angle of Shearing Resistance ϕd	
	Void Ratio e	Water Content w (%)	Degree of Saturation Sr (%)	by $(\sigma_1-\sigma_3)f$ measured	by $(\sigma_1-\sigma_3)rf$ calculated	by $(\sigma_1-\sigma_3)f$ measured	by $(\sigma_1-\sigma_3)rf$ calculated
1-40	0.890	18.2	47.6	0.019	0.020	34° 22′	35°54′
50	0.816	21.4	57.9	0.037	0.025	29° 55′	32°17′
2-40	0.844	14.3	42.4	0.148	0.112	33° 00′	35°11′
50	0.826	17.2	52.2	0.077	0.063	32° 47′	34°37′
60	0.772	20.3	63.9	0.089	0.079	29° 48′	31°30′
70	0.678	22.1	78.9	0.033	0.018	31° 41′	33°16′
3-40	0.678	11.0	42.2	0.223	0.195	34° 18′	35°47′
50	0.693	13.6	49.6	0.144	0.148	36° 07′	37°05′
60	0.638	16.0	61.5	0.180	0.140	35° 00′	36°32′
70	0.638	17.6	70.9	0.228	0.208	31° 13′	32°30′
80	0.633	20.3	83.3	0.166	0.152	30° 18′	31°43′
90	0.601	21.7	91.6	0.083	0.068	31° 52′	32°50′
4-40	0.545	8.7	40.8	0.603	0.755	35° 34′	33°06′
50	0.542	10.8	51.8	0.626	0.569	35° 09′	35°11′
60	0.544	13.0	62.1	0.547	0.504	35° 58′	35°52′
70	0.529	14.9	73.5	0.501	0.448	35° 08′	35°40′
80	0.537	16.9	81.5	0.521	0.470	30° 40′	31°33′
90	0.556	18.7	88.0	0.342	0.315	31° 10′	32°12′

Table 3. Results of Shear Test, Test 2

Test Number	Initial Value			Apparent Cohesion Cu(kg/cm²)	Angle of Shearing Resistance ϕ_u
	Void Ratio e	Water Content w(%)	Degree of Saturation Sr(%)		
1-40	1.116	16.4	38.8	0.064	17° 20′
50	1.154	21.9	50.3	0.058	7° 19′
2-40	0.844	12.8	40.3	0.219	26° 57′
50	0.889	17.1	50.9	0.238	20° 59′
60	0.884	19.9	59.8	0.161	15° 48′
70	0.849	23.0	71.9	0.079	3° 28′
3-40	0.686	10.1	38.9	0.410	30° 30′
50	0.690	12.6	48.5	0.539	26° 49′
60	0.701	15.8	59.8	0.307	27° 17′
70	0.675	16.7	65.4	0.324	24° 49′
80	0.682	20.4	79.4	0.299	8° 27′
90	0.673	21.5	84.5	0.403	4° 51′
4-40	0.562	8.8	41.4	0.798	32° 59′
50	0.557	9.9	46.8	0.621	33° 23′
60	0.557	11.7	55.6	0.679	31° 24′
70	0.571	14.2	66.1	0.749	31° 00′
80	0.561	15.7	73.9	0.638	29° 16′
90	0.557	17.2	81.8	0.442	27° 46′

these the following may be said regarding the apparent cohesion and the angle of shearing resistance.

Regarding the apparent cohesion, the values which were corrected for the dilatancy effect were smaller than the values which were not corrected. In Test 2, the angles of shearing resistance which were corrected for the dilatancy effect were larger than the uncorrected values by 1° to 2° regardless of the void ratio and the degree of saturation. Concerning the apparent cohesion and the angle of shearing resistance which were calculated from measured maximum deviator stress $(\sigma_1-\sigma_3)_f$, the following observation may be made. On the basis of the Table 2, the relationship between apparent

Table 4. Results of Shear Test, Test 3

Test Number	Initial Value			by Total Stress		by Effective Stress	
	Void Ratio e	Water Content w (%)	Degree of Saturation Sr (%)	Apparent Cohesion Cu (kg/cm²)	Angle of Shearing Resistance ϕu	Apparent Cohesion C' (kg/cm²)	Angle of Shearing Resistance ϕ'
1-40	0.819	13.2	42.6	0.087	30°32′	0.044	38°14′
60	0.796	19.4	63.9	0.100	20°47′	0.056	35°29′
80	0.767	25.7	88.2	0.084	3°50′	0.074	13°46′
2- 0	0.631	0	0	0.047	44°45′	0.016	46°29′
40	0.639	10.5	43.7	0.226	34°28′	0.195	36°55′
60	0.670	14.8	58.2	0.280	28°52′	0.102	36°55′
80	0.668	19.6	77.2	0.216	21°10′	0.031	37°23′
100	0.622	23.5	98.7	0.307	0°10′	—	—
3-40	0.485	7.4	39.8	0.803	39°40′	0.829	40°13′
60	0.484	11.1	60.1	0.630	37°37′	0.531	39°39′
80	0.503	14.8	77.2	0.516	32°53′	0.286	39°22′
100	0.520	18.1	91.6	0.287	30°48′	—	—

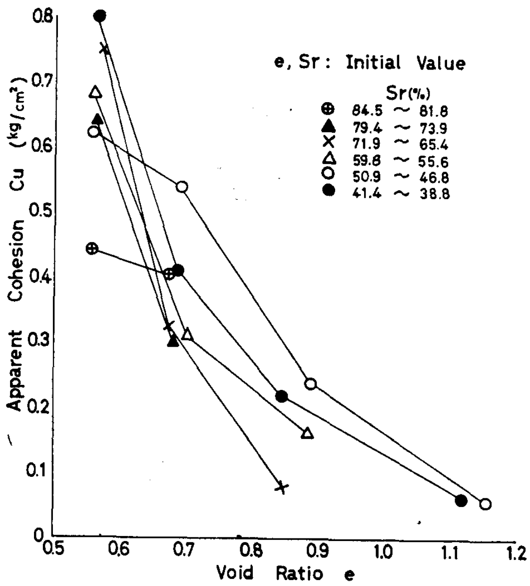

Fig. 9. Relation between Apparent Cohesion and Void Ratio in Test 1

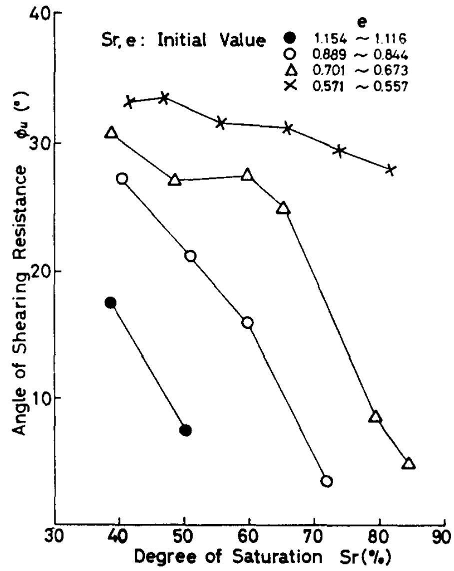

Fig. 10. Relation between Angle of Shearing Resistance and Degree of Saturation in Test 1

cohesion (C_u) and void ratio (e) and the relationship between angle of shearing resistance (ϕ_u) and degree of saturation (S_r) are plotted in Fig. 9 and Fig. 10 respectively. On the basis of Table 3, the relationship between apparent cohesion (C_d) and void ratio (e), and the relationship between angle of shearing resistance (ϕ_d) and degree of saturation (S_r) are plotted in Fig. 11 and Fig. 12, respectively. On the basis of Table 4, a Mohr's circle, the relationship between apparent cohesion (C_u, C') and void ratio (e), and the relationship between angle of shearing resistance (ϕ_u, ϕ') and degree of saturation (S_r) are plotted in Fig. 13, Fig. 14 and Fig. 15, respectively. From these graphs the following observation may be made:

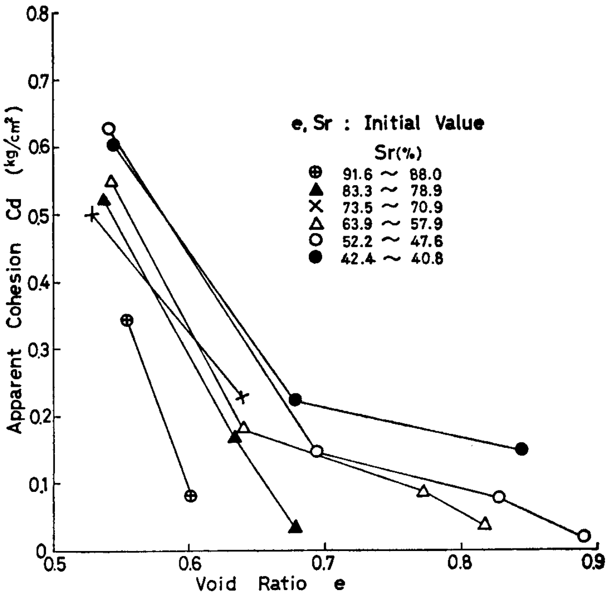

Fig. 11. Relation between Apparent Cohesion and Void Ratio in Test 2

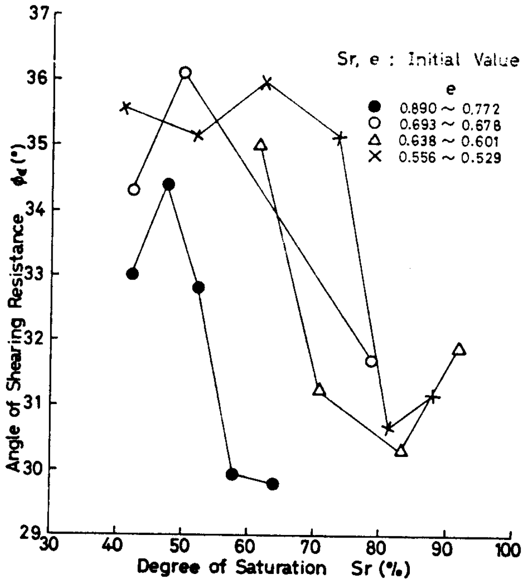

Fig. 12. Relation between Angle of Shearing Resistance and Degree of Saturation in Test 2

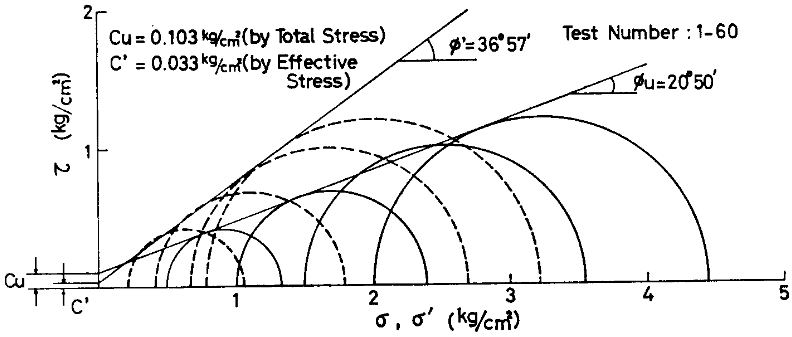

Fig. 13. Mohr's Circle in Test 3

Test 1:—The apparent cohesion decreases with an increase in void ratio regardless of the degree of saturation. When the degree of saturation is large, for a small increase in the void ratio the decrease of apparent cohesion is large. With an increase in degree of saturation, the slopes of C_u-e curves (Fig. 9) seem steep but this trend is not very clear. The angle of shearing resistance decreases sharply with an increase in void ratio, and this tendency to decrease becomes large with an increase in degree of saturation. With an increase in degree of saturation, the angle of shearing resistance decreases sharply. When the void ratio is large, for a small increase in the degree of saturation there is a large decrease in the angle of shearing resistance.

Test 2:—Regarding the apparent cohesion in this test, the same can be said as for Test 1. The angle of shearing resistance decreases somewhat

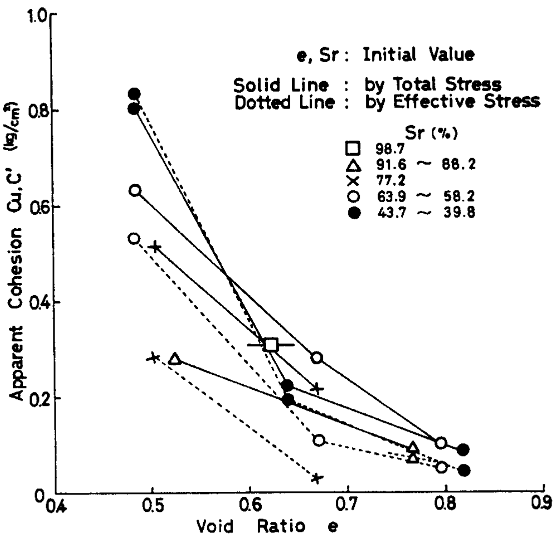

Fig. 14. Relation between Apparent Cohesion and Void Ratio in Test 3

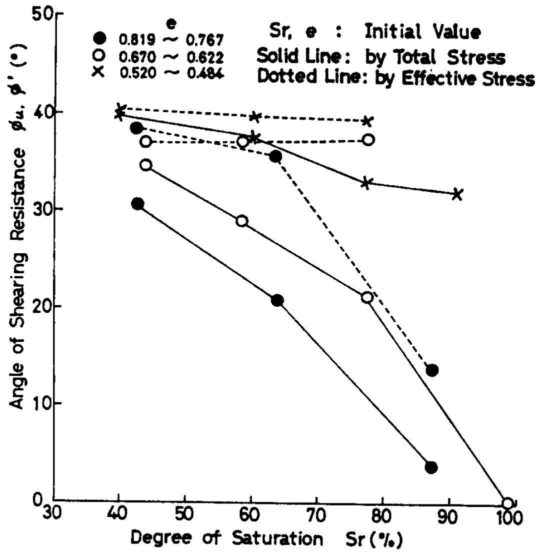

Fig. 15. Relation between Angle of Shearing Resistance and Degree of Saturation in Test 3

as the void ratio increases, but this tendency is not so clear as the apparent cohesion. This value also decreases with an increase in degree of saturation.

The apparent cohesion decreases abruptly to near zero, namely, about 0.02 kg/cm² when the void ratio and the degree of saturation become large. However, the angle of shearing resistance does not decrease so much, and settles at about 30°. Since this test is the drained compression test, the decrease in the angle of shearing resistance was avoided.

Test 3:—When the degree of saturation is constant, the apparent cohesion decreases with an increase in void ratio. This value does not seem to be affected by the degree of saturation when the void ratio is large. However, when the void ratio becomes small, this value decreases with an increase in degree of saturation. These tendencies are the same for the two kinds of apparent cohesions which are obtained from the total stress and the effective stress. The value of the apparent cohesion which is obtained from the effective stress is smaller than that obtained from the total stress. Comparing the two kinds of angles of shearing resistance obtained from the total stress and effective stress, the latter is larger than the former. The angle of shearing resistance obtained from the total stress shows similar tendencies as that obtained in undrained Test 1. When the degree of saturation is small, the angles of shearing resistance calculated from the effective stress are not affected so much by a change in void ratio. Only in the case of large void ratios, it is found that the angle of shearing resistance decreases with an increase in degree of saturation.

APPLICATION OF TEST RESULTS

When an embankment is constructed with sandy soils which have a consistency and a grain size distribution such as "Masa"-soil, the following considerations should be taken into account. In undrained tests, the apparent cohesion and the angle of shearing resistance decrease with an increase in degree of saturation. In drained tests, the apparent cohesion decreases with an increase in degree of saturation, but the angle of shearing resistance does not decrease so much. This means that a slope erosion and an embankment failure may occur easily due to a decrease in the apparent cohesion with an increase in the degree of saturation due to rainfall. When drainage is permitted, the influence of the angle of shearing resistance on the failure of the embankment is smaller than that of apparent cohesion.

When the soil is compacted with a larger void ratio than that corresponding to the maximum density of the standard compaction test even though the degree of saturation is low, the apparent cohesion (C_u, C_d) and the angle of shearing resistance (ϕ_u) decrease sharply. This indicates that the compaction of the embankment should be conducted with care.

SUMMARY

The shearing characteristics of partially saturated soils, especially the influence of the void ratio and the degree of saturation on change of volume, stress-strain, pore pressure, and shearing resistance have been made fairly clear. The volume change seems to be influenced primarily by the void ratio and secondly by the degree of saturation.

The apparent cohesions, obtained in the drained test and the undrained test, are influenced mostly by the void ratio. The influence of the degree of saturation on these values is not very large. The angle of shearing resistance in the drained test is not greatly influenced by the void ratio and the degree of saturation. In the undrained test, however, the influence of the void ratio and the degree of saturation on the angle of shearing resistance is fairly large. On the basis of the effective stress, as in the results of the drained test, the apparent cohesion is controlled mainly by the void ratio, but the angle of shearing resistance is not controlled so much by the void ratio and the degree of saturation. The shearing resistance of compacted sandy soils such as "Masa"-soil decreases with an increase in the degree of saturation due to rainfall. When an embankment is constructed using sandy soils, the slope should be well protected and the soil compacted with care.

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NOTATIONS

C_d	apparent cohesion in drained shear test
C_u	apparent cohesion in terms of total stress in undrained shear test
C'	apparent cohesion in terms of effective stress in undrained shear test
D.I.	dilatancy index named by Prof. Akai
$d\left(\frac{\Delta V}{V}\right)/d\varepsilon$	increase in the rate of change of volume divided by the change in unit shear strain
e	void ratio
I_p	plasticity index
S_r	degree of saturation
$\Delta V/V$	rate of change of volume
w	water content
w_L	liquid limit
w_{opt}	optimum water content
w_P	plastic limit
$\gamma_{d \cdot \max}$	maximum dry density
ε	strain
σ	total normal stress
σ'	effective normal stress
$(\sigma_1-\sigma_3)_f$	measured maximum deviator stress
$(\sigma_1-\sigma_3)_{rf}$	deviator stress required to overcome the cohesive resistance and the frictional resistance of the soil
τ	shear stress
ϕ_d	angle of shearing resistance in drained shear test
ϕ_u	angle of shearing resistance in terms of total stress in undrained shear test
ϕ'	angle of shearing resistance in terms of effective stress in undrained shear test

REFERENCES

Akai, K., 1958, The dilatancy effect during shear of sands, Transaction, Japan Soc. Civil Engrs., No. 58, July 1958, pp. 76-81.

Bishop, A. W., and D. J. Henkel, 1957, The measurement of soil properties in the triaxial test, p. 64, Fig. 38.

Uchida, I., R. Matsumoto, K. Onitsuka and T. Tanaka, 1967, A measurement of pore air pressure in partially saturated soils, Soil and Foundation (Japanese edition), Vol. 15, No. 9, Sep. 1967. pp. 7-13.