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Failure of Embankment Slope of Silty Sand "Masa"

Glissement de Talus d'un Remblayage de "Masa"

I.UCHIDA Professor,
T.HIRATA Assistant, Kyushu University, Fukuoka, Japan

SYNOPSIS By the large scale model tests, it is cleared that a failure of the embankment slope made of "Masa" is a successive one. Usually the analysis is based on the rigid-plastic theory that seems to have some limits in the case of embankment having a steep slope angle. So the authors tried to solve such problems in different conditions. For the purpose of it, the authors cautiously observed the order of slip surface occurrence and classified the failure of the embankment slope into two types. Each of them should be treated with the proper coefficient of soil strength. The authors described the behavior of the embankment slope failure due to dynamic load and seepage. For very steep slope, traditional calculation gives fairly larger value than experimental one. The paper guides to solve such a problem, too.

INTRODUCTION

The so-called "Masa" which is widely distributed in the south west part of Japan, is a residual soil of weathered granite. Fig. 1 shows geographical distribution of "Masa" in Japan. "Masa" has not only a special merit to be compacted easily but also a weakpoint of losing the strength sensitively by water. Therefore it still has some problems to use "Masa" as the material of embankment slope in construction and stability. For studying the mechanism of slope failure, the authors carried out large scale model tests, mainly on the following three cases as shown below.

- (1) Bearing capacity when it is loaded statically at the top near the embankment slope.
- (2) Bearing capacity when it is loaded dynamically at the top near the embankment slope.
- (3) Failure mechanism of embankment slope due to seepage.

Total number of tests carried out is about sixty.

MATERIAL AND METHOD OF TESTING

"Masa" produced in the suburbs of Fukuoka-city (Fig. 1) were used for the embankment model and its properties are as follow. Specific gravity of particle $G_s = 2.64 - 2.69$, liquid limit $w_L = 39\%$, and plasticity index $I_p = 8\%$.

The concrete pit having the depth, width of 100 cm and the length of 235 cm is used for the tests. Its one side was made with safety glass to see the development of slip surface. 10 cm of "Masa" layer was taken as one layer for compaction. The process of compaction was repeated under constant compacting energy until the designed height was achieved. Then the embankment slope was made properly for various tests.

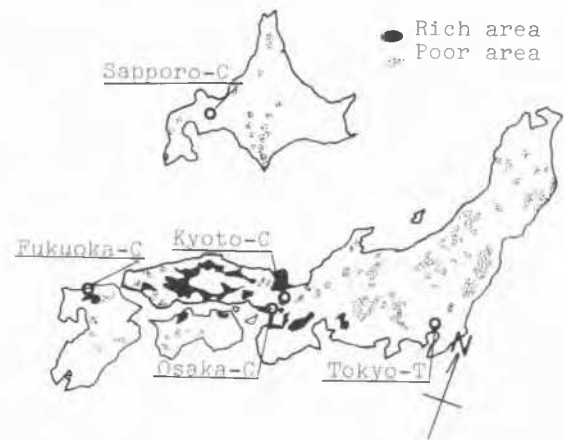


Fig. 1 Distribution of "Masa"

Rigid loading plate is reformed in order to remove the influence of side friction between soil and glass or concrete. Investigations on settlement of loading plate, bearing capacity, slip surface and behavior of soil were done. Size of embankment slope is as follow, its height of 80, 60 cm, loading plate width of 7.5, 12.5, 15.0 cm, the distance from loading plate edge to the top of slope 0, 10, 20 cm and so on. Loading was done using stress and strain control methods.

TEST RESULTS AND CONSIDERATION

- (1) Bearing capacity under static loading. The authors precisely observed slip surface when it occurred and how it grew up. And the values of the bearing capacity are compared with calculated ones. The calculated values of bearing capacity were taken from Meyerhof's formula. The relation between the ratio of calculated value (q) to experimental value (q_u) and slope angle (β) is shown in Fig. 2.

At the steep slope, (in this case, greater than about 60 degree) the calculated value is greater than the experimental one which shows the unsafety. Besides the value of q/qu take 0.5 - 2.5 when the slope angle is between 30 and 90 degree. The accuracy is still not satisfactory. The accuracy of slip surface is still not satisfactory even when Sokolovskii's method is used. (Fig. 3)

So the authors precisely looked for the occurrence of slip surface, the rate of its growth and divided the slip surface as shown in Fig. 4. According to this division, the order of slip surface occurring can be classified as shown in Table I.

Consequently, judging from many tests, the authors can be concluded the embankment failure is a successive one.

From Table I, loading at the top near the embankment slope, slip surface A occurs earlier than B. (Fig. 4) This reason is illustrated in Fig. 5. Loading at the top near the embankment slope, the value of horizontal stress of point P on ac-line is smaller than P' on bc-line, because of influence of slope. Then as shown in Fig. 5, there are two Mohr's circle. One shows the stress of point P on ac-line and the other point P' on bc-line where the depth from soil surface is the same. The radius of former's Mohr's circle is always larger than the latter's. For that, two Mohr's circles do not reach to the failure criterion at the same time. Mohr's circle of point P touches the failure criterion earlier than one of point P' does. So there is some difference of time between two.

When cde-line is short, for example when slope angle is steep and loading plate is

Table I Order of slip surface occurrence

Test No.	Slope angle(β) (degree)	Distance (b) (cm)	Order of occurrence of slip surface			Remark
			A	B	C(D)	
I -1	90	0	2	-	1	Fall down
	75	0	2	3	1	
	60	0	2	3	1	
	45	0	1	3	2	
	30	0	1	-	2	
II -1	90	10	2	-	1	Fall down
	75	10	2	3	1	
	60	10	1	3	2	
	45	10	1	3	2	
	30	10	1	-	2	
III -1	90	20	-	-	1	Fall down
	75	20	2	3	1	
	60	20	1	3	2	
	45	20	1	3	2	
	30	20	1	-	2	

closely near the top of slope, slip surface C occurs earlier than A. But the longer the length of cde-line become, the later slip surface C occurs. By the authors tests, when the slope angle is between 30 and 90 degree and the distance from the loading plate edge to the top of slope (b) is the same to loading plate width or slightly larger, slip line C seems to occur earlier than B does.

On the other hand, when the loading plate is set far from the top of slope or the slope angle is close to zero, cde-line becomes more longer.

Then the slip surface C occurs later than B does. Consequently, as mentioned above, the slip surfaces do not occur at once but occur successively.

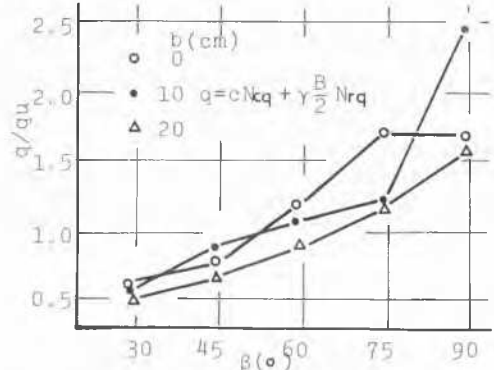


Fig. 2 Relation between q/qu and β

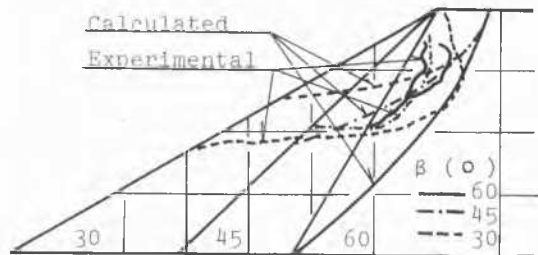


Fig. 3 Comparison of calculated slip surface with experimental one

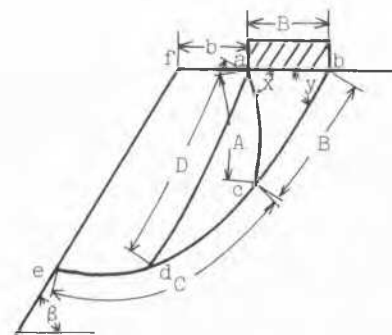


Fig. 4 Division of slip surface

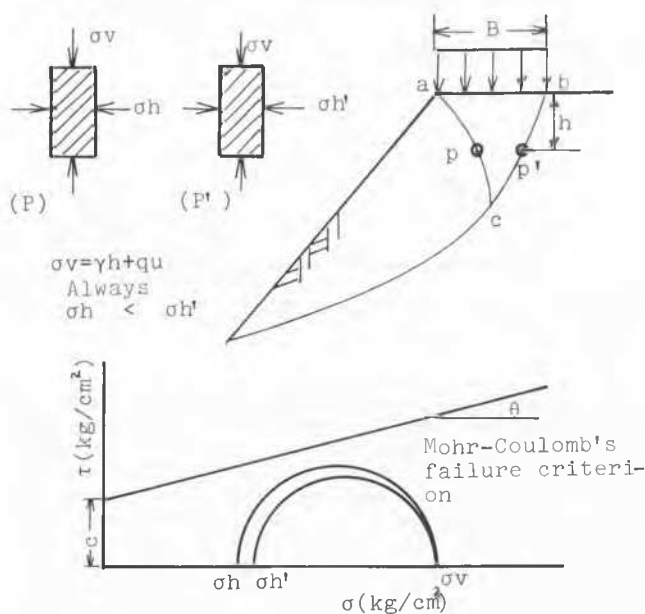


Fig. 5 Explanation of difference between occurrence of slip surface A & B

One of the most important causes of error between calculated value of bearing capacity and experimental one seems to be the calculation which is based on rigid-plastic theory. (Fig. 3)

On the other hand, when slope angle is steep, soil of mass acdef will fall down as soon as slip surfaces A and C are both complete. (Fig. 4) It seems to be the main cause of having the lower experimental. The authors observed the order of slip surface occurrence and noticed the difference between growth speeds of slip surfaces. It took 16.5 minutes from occurrence of slip surface to failure of slope when $b = 0$, $\beta = 30^\circ$ and $e = 0.77$ that is the longest one. And it took 1/6 minute for $b = 0$, $\beta = 90^\circ$ and $e = 0.74$ which is the shortest one.

Then the authors surveyed the slip surface's growth speed in detail with taking picture and found that slip surface C is the fastest of all.

For the difference of slip growth speed, the authors classified the failure mechanism into progressive failure and creep failure as shown in Table II.

Here, progressive failure obeys the Morgenstern's definition and creep failure is a failure due to constant loading. Progressive failure seems to occur near the slope where the soil is more or less in the state of over consolidation due to over compaction or cutting off in construction of embankment.

Where stress-strain curve shows a peak for over consolidated soil. (Fig. 6)

Before the peak, strain increment accompanies stress increment. Under this condition the

embankment slope is still stable. But after passing the peak, strain increment accompanies stress decrement. Then it cannot get necessary stress increment to keep it stable, at all. Therefore, the slip surface grow up fairly fast until the stress increment to keep the embankment slope stable is given by continuing the stress redistribution at the edge of slip surface.

On the other hand, creep failure has a tendency to occur at zone abc which seems to be normally consolidated one. In this state, stress-strain curve does not have a peak and strain increment always give corresponding stress increment. Hence, the zone abc can still keep itself stable. As a result, the growth speed of slip surface is low at zone abc.

In the tests, the growth speed of slip surface at zone ed was found to be higher and it developed toward point c. The more closely it approached, the lower the growth speed became and finally it seemed to be stopped. After that, at abc zone slip surface B occurred and was completed. That is the failure of the embankment slope.

Fig. 6 shows the stress-strain curves of "Masa" which were taken by the direct shear test. These curves are classified into two groups. One has a peak: over consolidation and the other does not: normally consolidation. Its difference is due to the magnitude of vertical stress when initial void ratio are the same. Previously referred Meyerhof's formula is based on the rigid-plastic theory and used one coefficient of soil strength in

Table II Relation of failure type and various index properties

Failure type		Progressive failure	Creep failure
Embankment condition		O. C. (C-zone)	N. C. (A·B-zone)
Stress-strain Relation	Peak	Exist	Non-exist
	Stress increment ($\Delta\sigma$) for strain increment ($\Delta\epsilon$)	Before peak $\Delta\sigma$ After peak $-\Delta\sigma$	Always
Slip surface growth speed		$(\Delta\sigma) \rightarrow$ Decrease \rightarrow Stable $(-\Delta\sigma) \rightarrow$ Increase \rightarrow Unstable	$(\Delta\sigma) \rightarrow$ Decrease \rightarrow Stable
Spent time for failure		Short	Long
Slope angle		Steep	Gentle
Length of C-zone		Short	Long
Coefficient of soil strength to be used		Coefficiencie of residual strength	A certain % of coefficient of u. u. strength

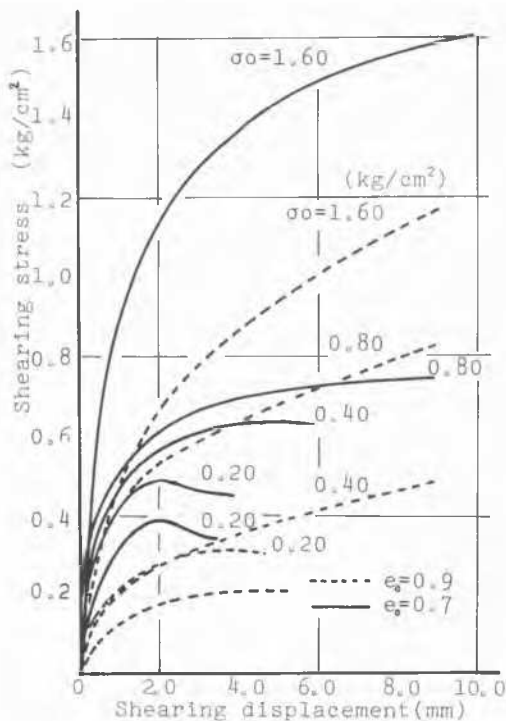


Fig. 6 Relation between stress and strain of "Masa"

calculation.

But as the authors told hitherto, the failure of the embankment slope usually contains progressive failure and creep one. So coefficient of soil strength ought to be preferred for each failure. For the former, coefficient of residual strength and for the latter, a certain percent of coefficient of soil strength gained by unconsolidated undrained shear test should be used. (Table II)

Soil mass edcaf will fall down before the embankment slope failure when the slope angle is very steep such as 75 - 90 degree. As a result, the bearing capacity can not be expected. So in such a case, the authors present to calculate the bearing capacity without considering it of mass edcaf.

The relation of bearing capacity and water content is studied by testing about 20 models' embankment slope and it is cleared that the bearing capacity has a peak where the water content is slightly smaller than the optimum water content, if the void ratio are all the same.

(2) Bearing capacity under dynamic loading. Fig. 7 shows the relation between Pre/Pus and Pure/Pus where Pure is defined as the static bearing capacity. The value of Pre shows a great influence on the increment of soil strength. After construction of the embankment slope, opening to the traffic for a period is useful. That is the recognized from the tests.

(3) Failure for seepage
Failure of slope is composed of two types,

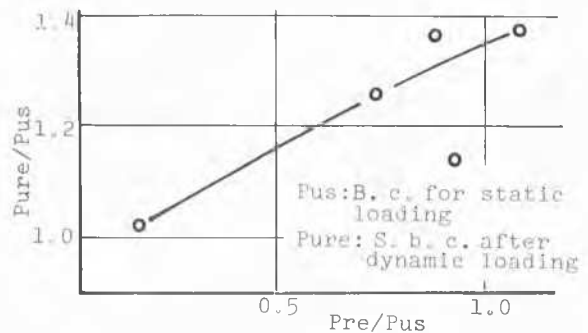


Fig. 7 Relation between Pre/Pus & Pure/Pus

pre-failure and main-failure. Pre-failure occurs first near the out let of seepage and gradually the main-failure. From the authors' model tests, it is cleared that "Masa" has a characteristic of weakening the strength by seepage. Particularly in the state of loose compaction, its characteristic strongly appears. Then, "Masa" should be treated very carefully for water under construction in practice.

CONCLUSION

From many large scale model tests, the authors get the following conclusions.

- (1) Failure of the embankment slope seems to occur successively. According to the difference of slip surface growth speed, the authors classified the failure mechanism into progressive failure and creep failure. So the authors present the two different coefficients of strength (Table II) and they should be taken into account in the analysis. In the case of steep slope ($\beta = 75 - 90$ degree), soil mass edcaf will fall down before the embankment failure, the calculation must be done without considering of its part.
- (2) When the void ratio is all the same, the bearing capacity has a peak where the water content is slightly smaller than optimum water content. This characteristic seems to be invariable when the void ratio changes. In the construction of embankment slope, usually soil is compacted in the state near the optimum water content, it was ascertained that for the shear strength of compacted soil, it is useful, too.
- (3) By dynamic loading the embankment slope was usually strengthened. After construction of the embankment slope, opening to the traffic for a period is useful. That is the recognized from the tests.
- (4) As "Masa" is weakened by water sensitively, it must be compacted enough for preventing itself from weakening for seepage.

REFERENCES

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