Evaluation of rainfall induced slope failure in tumulus mounds and conservation of the damaged tumuli

Mai Sawada, Mamoru Mimura, and Mitsugu Yoshimura

ABSTRACT

Tumuli, ancient tombs made of compacted earth mounds, were constructed more than 1300 years ago and they have been conserved as historical cultural heritages. However, many of them are severely damaged due to natural external forces and human activities. Rainfall induced slope failure is one of important and difficult problems that geotechnical engineering should take care of. In this paper, the failure mechanism of tumulus mounds is evaluated by exemplifying the rainfall induced failure of the Kengoshizuka Tumulus in Japan. The surface of the mound is severely embrittled and has lower density and strength than those of the sub-layer, making the boundary of the layers vulnerable to failure. Rainfall infiltration into the layered mound is estimated by seepage flow analysis and the induced instability is evaluated. The strength parameters of the sliding surface are assessed by in-situ and laboratory tests. The result of the evaluation explains the rainfall induced slope failure in 2012.

Keywords: slope stability, seepage flow analysis, unsaturated soil, rainfall, historical cultural heritage

1. INTRODUCTION

Tumuli were constructed as tombs for the ancient imperial family and district rulers and they have been conserved as historical cultural heritages. They are usually constructed by compacted earth mounds covering stone chambers for coffins. Some of them are still kept in good condition more than 1300 years after the construction, but the others are severely damaged due to natural external forces, namely heavy precipitation and earthquake, as well as human activities such as tomb robbery and urban development. It is true that conservation and reconstruction of the damaged tumuli are important issues but they sometimes result in destruction instead of conservation without a due consideration of mechanical and hydraulic stability of the tumulus mounds. It should be emphasized that inappropriate restoration and reconstruction will cause the reoccurrence of failure of the tumulus mounds and harmful infiltration of rain into the chambers. Collaborative action among archeology and the related field of science and technology including geotechnical engineering is strongly required to find the rational solution for the complex problems of conservation of geo-relics.

Rainfall infiltration induced instability of tumulus mounds is one of the most important and difficult problems that geotechnical engineering should take care of. In this paper, the failure mechanism of the tumulus mound in the Kengoshizuka in Japan is exemplified.
compacted earth mound is evaluated by exemplifying the rainfall induced failure of the Kengoshizuka Tumulus in Japan. In the rainy season of 2012, a slope failure took place by a heavy rainfall in the tumulus mound (see Fig.1). A large amount of water infiltrated into the chamber in the mound. Evaluation of the mechanism of the rainfall induced slope failure surely leads to develop the comprehensive scheme for the conservation of tumuli. The procedure of the evaluation is shown in Fig.2.

2 STRUCTURE OF THE TUMULUS MOUND AND SOIL PROPERTIES

2.1 Structure of the tumulus mound

An archeological excavation was conducted around the failure part of the mound. The tumulus mound is made of dense compacted decomposed granite soil called “Hanchiku”. The compacted layer pattern is observed in the deeper part of the mound, while many cracks and pores are found in the surface part instead of the pattern. The cracks and pores in the surface seem to be caused by dead plants and weathering.

The structure of the tumulus mound is quantitatively investigated by elastic wave exploration and some other in-situ tests. Elastic wave exploration shows that there is an S-wave velocity boundary at depth of 0.5-1.0 m and the mound consists of two layers. The surface and sub-layer of the mound has S-wave velocities of 50-100 and 175-200 m/s, respectively. The surface layer hence has lower density and strength than those of the sub-layer.

2.2 Density

Mechanical and hydraulic properties of a soil are controlled by the density. Density is usually measured by destructive ways but they are not applicable to tumulus mounds. In Kengoshizuka Tumulus, some of the excavated soil was available for experiments although destructive in-situ tests and sampling were considerably restricted. Density of the tumulus mound is hence assessed by compaction test using the excavated soil. Density depends on compaction energy. Based on the fact, some tumulus mounds were examined for the densities and relations with compaction energy. As a result, tumulus mounds were found to be constructed by the compaction energy corresponding to 10 to 20 % of $E_c$ (Mimura and Yoshimura, 2011, Sawada et.al., 2013). Here, $E_c$ is standard Proctor compaction energy (=550 kJ/m$^3$).

Compaction tests under various compaction energy levels give a relation between density and compaction energy shown in Fig. 3. Water content of the specimens are adjusted to that in natural condition ($w_n$=29%). The relation shows that the tumulus mound has a density ranging from 1.23 (0.1$E_c$) to 1.29 (0.2$E_c$) g/cm$^3$. The densities of the surface and sub-layer are hence assumed to be 1.23 and 1.29 g/cm$^3$, respectively.

2.3 Permeability and water retention

Seepage flow in the unsaturated tumulus mound is mainly controlled by permeability and water retention of the mound soil. Water retention test is conducted to obtain a water retention curve of the sub-layer. The specimen made of the excavated soil is 5cm height, 5cm diameter and 1.29 g/cm$^3$ density. The test apparatus controls suction both by water column
method (0-5 kPa) and pressure plate method (5-100 kPa). A ceramic disk with 100 kPa air entry value is adopted. The wetting and drying curves obtained are shown in Fig. 4 (a). The experiment data is complemented by van Genuchten’s model (Eq. (1), (2), (3)) (van Genuchten, 1980). In seepage flow analysis discussed below, the wetting curve is used.

The water retention curve of the surface layer is hard to obtain by water retention test on a reconstituted specimen because the surface layer of the mound has many cracks and pores that deteriorate water retention. The curve of the surface layer is hence created based on the experimentally obtained wetting curve of the sub-layer. The physical properties and van Genuchten’s model parameters of the two layers are shown in Table 1. For simplicity, the surface layer is assumed to have the same fitting parameters α, n and m as the sub-layer. The saturated volumetric water content θs of the surface layer is adjusted to the maximum degree of saturation of the sub-layer (Ss=94 %). The residual volumetric water content θr of the surface layer is determined by reference to the natural degree of saturation and corresponding suction of the sub-layer. Here, natural degree of saturation of the sub-layer is observed at suctions Ψ of 5 meters. At the suction, degree of saturation of the surface layer is adjusted to that in natural condition considering the continuity of suction between the layers.

Saturated hydraulic conductivity ks of the sub-layer is obtained by permeability test, while that of the surface layer is assumed to be ten times larger considering the cracks and pores. Relative hydraulic conductivities k(θ) of the two layers are evaluated by Eq. (4) based on van Genuchten’s and Mualem’s models (van Genuchten, 1980). The relations between hydraulic conductivity and suction of the layers are shown in Fig.4 (b).

\[
S_s = \left( \frac{\theta - \theta_r}{\theta_s - \theta_r} \right)^m
\]

(1)

\[
m = 1 - \frac{1}{n}
\]

(n > 1)

(2)

\[
\kappa(\theta) = k_s S_s^{1/2} \left( 1 - \left( S_s^{1/2} \right)^m \right)^2
\]

(4)

4 EVALUATION OF RAINFALL INFILTRATION INTO THE TUMULUS MOUND

4.1 Analysis model and conditions

Rainfall infiltration into the tumulus mound is evaluated by saturate-unsaturated seepage flow analysis (K. Akai et al., 1977) using the properties assessed in the previous chapter. The analytical model adopted is shown in Fig. 5. The boundary of the two layers is obtained by elastic wave exploration. Rainfall infiltrates into the mound by giving inflow discharge on the surface of the mound. Here, the maximum amount of infiltration is restricted to the saturated hydraulic conductivity of the surface layer. The boundary on the

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**Table 1. Properties of the surface and sub-layer.**

| Parameter                              | Surface layer | Sub-layer |
|----------------------------------------|---------------|-----------|
| Soil particle density (g/cm³)          | 2.745         | 2.745     |
| Dry density (g/cm³)                    | 1.23          | 1.29      |
| Void ratio                             | 1.23          | 1.13      |
| Water content (%)                      | 29.0          | 29.0      |
| Degree of saturation (%)               | 64.63         | 70.58     |
| Hydraulic conductivity k_s (cm/s)      | 9.91E-04      | 9.91E-05  |

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Fig. 5. The analytical model of the tumulus mound.

Fig. 6. The amount of rainfall in June 16th to 22nd, 2012.
right side is impermeable because it corresponds to the centerline of the mound. Those on the bottom and left side are also impermeable in unsaturated condition but when it goes into saturated condition, pressure heads on the boundaries reach to zero and water discharges through the boundaries.

Fig. 6 shows the amount of rainfall near the tumulus in June 16th to 22nd, 2012 (Japan Metrological Agency, 2014). The amount of rainfall is measured every 10 minutes. There is a highest rainfall of the month during the seven days. Rainfall infiltration and slope stability of the tumulus mound are evaluated during the seven days.

4.2 Results

The distributions of degree of saturation in the mound during the seven days are shown in Fig. 7. The numbers in the figure are explained in Fig. 6. The initial condition is determined by steady-state pressure head corresponding to the distance from groundwater level. The average initial degrees of saturation of the surface and sub-layer are 63.48 % and 69.90 %, respectively. They are almost the same with those in Table.1.

Water seeps in the surface layer toward the foot of the mound and degree of saturation on the boundary of the layers increases from the foot. This is because of the difference in permeability of the two layers. The discontinuity of density and strength at the boundary is one of the endogenous factors of slope failure. Furthermore, the analytically obtained distribution of degree of saturation in the mound shows that precipitation makes the boundary to be more vulnerable to failure.

5 SLOPE STABILITY OF THE TUMULUS MOUND

5.1 Methodology

Safety factor in terms of slope failure is evaluated during the seven days, assuming the slope failure occurs at the boundary. Infinite slope method (Eq. (5)) is adapted to a sliding soil mass shown in Fig. 5. The sliding soil mass is geometrically most vulnerable to failure in the analytical model because it has the largest thickness and gradient. The thickness and gradient are $H=0.80$ meters and $i=32.33$ degrees, respectively.

Calculation of safety factor of slope requires assessment of three parameters; wet unit weight of the sliding surface $γ$, cohesion $c$ and angle of shear resistance $φ$ of the sliding surface. Cohesion is the most influential parameters and largely depends on water content in unsaturated condition. In the present study, cohesion of the sliding surface is evaluated in a comprehensive way that consists of in-situ and laboratory experiments.

$$F_s = \frac{c}{\gamma H \sin i \cos i} + \frac{\tan φ}{\tan i}$$

5.2 Strength evaluation by in-situ needle penetration test

In-situ strength evaluation is helpful especially for such embrittled tumulus mound because laboratory experiments using reconstituted samples probably do not give accurate evaluations. Although destructive in-situ tests are not applicable to tumulus, almost non-destructive needle penetration test (The Japanese
Geotechnical Society, 2013) is one of acceptable strength evaluations for the tumulus mound. A schematic view of a needle penetration test apparatus is shown in Fig. 8 (a). The amounts of compression of the spring and penetration length of the needle are measured when the needle penetrates into the test surface. Unconfined compression strength $q_u$ is converted from penetration resistance using the calibration curve shown in Fig. 8 (b) (Sawada et al., 2013). Here, penetration resistance is defined as ratio of penetration force to length.

Needle penetration test was conducted on the excavated surface layer of the tumulus mound. Measurement points were placed on the surface layer at 5 centimeter intervals. Here, let the water content measured on the testing day to be natural water content $w_n$ (=29%). Distribution of converted $q_u$ of the surface layer is shown in Fig. 9. Unconfined compression strength of the surface layer is approximately 15-30 kN/m² hence cohesion is $q_u/2 = 7.5-15$ kN/m². Strength of the sub-layer is also evaluated by needle penetration test in the same way. The sub-layer has 5 to 6 times the strength of the surface layer.

5.3 Strength evaluation by direct shear test in natural and wetter conditions

Cohesion of unsaturated soil decreases with saturation because part of cohesion is caused by capillary pressure. In-situ needle penetration test after rain is a possible way to evaluate cohesion in wetter condition but the damaged tumulus mound should not be exposed to rainfall. The tumulus mound has been covered by a tarpaulin after the slope failure for prevention of further damage. For this case, combination of in-situ and laboratory experiments is feasible.

![Fig. 9. Distribution of unconfined compression strength of the surface layer given by needle penetration test.](image)

Here, cohesion is evaluated by direct shear test on specimens both in natural and wetter conditions. A specimen is made of the excavated tumulus soil and consists of two layers with different densities reflecting the layered structure of the tumulus mound. Both the layers of the specimen have 6 cm diameter and 1 cm height. The densities of the upper and lower layers are 1.23 and 1.29 g/cm³, respectively. The boundary of the layers is sheared before the test to express its vulnerability. The specimen is sheared at a rate of 0.2 mm/min until the displacement reaches 7.0 mm under a constant pressure of 15, 30, 60, 90, 120 kPa. After the first shear test, the 7.0 mm displacement is returned to the initial position and the amount of water necessary to saturate the specimen is given on the surface of the specimen. The wet specimen is sheared again. The average degrees of saturation of the specimens used in the first and second shear tests are 74.5 % and 95.4 %, respectively. A comparison of shear strengths in the two conditions shows 9.07 kN/m² reductions in shear strength by saturation (see Fig. 10).

5.4 Safety factor of slope

A relation between cohesion and degree of saturation of the sliding surface shown in Fig. 11 is provided by combining the results of in-situ needle penetration test and direct shear test. Here, cohesion in natural condition and its amount of decrease by saturation are determined by in-situ needle penetration.

![Fig. 10. The amount of decrease in shear strength by saturation.](image)

![Fig. 11. The amount of decrease in shear strength by saturation.](image)
test and direct shear test, respectively. Cohesion corresponding to analytically obtained degree of saturation is used in evaluation of safety factor of slope. Whereas, $\phi$ is not considered ($\phi = 0$) because $\phi$ is a less dominant factor of shear strength than cohesion when the thickness of the sliding soil mass is thin.

Table 2 and Fig. 12 show the calculated safety factor and the parameters of slope stability analysis. Safety factor drops with rainfall infiltration and recovers after rain. Cohesion of the sliding surface changes significantly although wet unit weight of the sliding soil mass fluctuates within a narrow range. This shows that rainfall induced instability of the tumulus mound is more controlled by decrease in shear strength than increase in shear stress. Evaluation of cohesion is hence a key factor for accuracy of stability analysis. Furthermore, highly accurate seepage flow analysis is required especially when cohesion has a high sensitivity for degree of saturation. Safety factor reaches a minimum at 144 hours after the highest rainfall. This is one of the results that explain the rainfall induced slope failure in the tumulus in 2012.

| Time | Degree of saturation (°) | Wet unit weight of the sliding soil mass (kN/m²) | Vertical stress on the sliding mass (kN/m²) | Shear stress on the sliding surface (kN/m²) | Cohesion on the sliding surface (kN/m²) | Safety factor $F_s$ (w/r) |
|------|--------------------------|-----------------------------------------------|-------------------------------------------|-------------------------------------------|--------------------------------------|-------------------------|
| 1    | 0.00                     | 63.66                                         | 66.77                                      | 15.50                                     | 8.86                                 | 5.60                    |
| 2    | 6.51                     | 66.77                                         | 15.58                                     | 8.90                                      | 5.63                                 | 11.25                   |
| 3    | 8.66                     | 66.77                                         | 15.66                                     | 8.95                                      | 5.66                                 | 11.25                   |
| 4    | 7.02                     | 66.77                                         | 15.86                                     | 9.06                                      | 5.73                                 | 11.25                   |
| 5    | 7.07                     | 66.77                                         | 15.89                                     | 9.08                                      | 5.74                                 | 11.25                   |
| 6    | 7.27                     | 66.77                                         | 15.97                                     | 9.12                                      | 5.77                                 | 11.25                   |
| 7    | 7.22                     | 66.83                                         | 15.98                                     | 9.13                                      | 5.77                                 | 11.25                   |
| 8    | 7.22                     | 67.58                                         | 15.96                                     | 9.12                                      | 5.77                                 | 11.25                   |
| 9    | 7.43                     | 68.13                                         | 16.03                                     | 9.19                                      | 5.81                                 | 11.25                   |
| 10   | 7.50                     | 68.79                                         | 16.08                                     | 9.28                                      | 5.83                                 | 11.25                   |
| 11   | 7.69                     | 72.49                                         | 16.22                                     | 9.27                                      | 5.86                                 | 11.25                   |
| 12   | 7.99                     | 73.75                                         | 16.39                                     | 9.33                                      | 5.92                                 | 11.25                   |
| 13   | 8.33                     | 73.95                                         | 16.60                                     | 9.48                                      | 6.00                                 | 11.25                   |
| 14   | 8.44                     | 83.44                                         | 16.73                                     | 9.61                                      | 6.08                                 | 7.37                    |
| 15   | 9.74                     | 89.22                                         | 17.33                                     | 9.90                                      | 6.26                                 | 0.95                    |
| 16   | 83.20                    | 89.49                                         | 16.56                                     | 9.46                                      | 5.99                                 | 4.74                    |

6 CONCLUSIONS

Evaluation of rainfall induced instability of tumulus mounds is discussed through the case study of Kengoshizuka Tumulus. A series of investigations shows that the surface layer is severely embrittled and has lower density and strength than those of the sub-layer of the mound. Rainfall infiltration into the mound is evaluated by seepage flow analysis. The result shows that the boundary of the layers is highly saturated because of the difference in permeability of the layers, suggesting that the boundary is vulnerable to failure. Safety factor of slope is hence evaluated assuming the slope failure occurs at the boundary. Strength parameters of the sliding surface are assessed by in-situ and laboratory tests. The calculated safety factor of slope explains the rainfall induced slope failure in the tumulus in 2012.

The result of the evaluation implies a risk of slope failure in other parts of the mound considering the whole surface of the mound is probably embrittled. The damaged tumuli would not exist as it is for a long period. Now, the first priority is conservation of the sub-layer and the chamber that are still kept in good condition. The damaged surface layer should be removed and restored to prevent them from further damage, although such conservation accompanying destruction may be usually unacceptable in terms of conservation of archeological authenticity.

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