Collapse behavior of slope due to change in pore water pressure

Effondrement d'une pente à cause d'une variation de la pression interstitielle

Nakata Y., Kajiwara T., Yoshimoto N.
Yamaguchi University ,Ube, Japan

ABSTRACT: In 2010, a slope failure occurred due to heavy rain following an elevation in ground water level. Although a significant rainfall was experienced at the site in the previous year, there was no deformation with a similar elevation. In the past several collapses were recorded at the site due to heavy rain. Thus tests were performed using a plane strain compression apparatus to understand the deformation behavior of sand subjected to cyclic loadings in pore water pressure which were simulated the observed data on the collapsed slope. Rainfall records and data obtained from the observation well and extensometer were also shown. Finally lessons learned from the data were outlined.

RÉSUMÉ : En 2010, un glissement de terrain eu lieu après une élévation du niveau de la nappe phréatique à cause de fortes pluies. L'année précédente, bien que des précipitations légèrement plus fortes aient été enregistrées sur le site, il n'y eu pas de mouvement de terrain avec un niveau similaire de la nappe. Dans le passé, plusieurs effondrements ont été enregistrés à cause de fortes pluies. Des essais ont été effectués en utilisant un appareil de compression en déformation plane pour comprendre le comportement en déformation d'un sable soumis à des pressions interstitielles cycliques qui expliquent les données observées sur la pente effondrée en 2009-2011. Les relevés pluviométriques et les données obtenues dans un puits d'observation et à partir d'extensomètres sont également indiqués. Enfin on souligne les leçons tirées de l'ensemble de ces données .

KEYWORDS: slope stability, heavy rain, ground water level, pore water pressure, monitoring system, .

1 INTRODUCTION

Slope failures induced by heavy rainfall are one of the most destructive natural hazards and these have claimed untold numbers of lives and millions of dollars in infrastructure losses every year in many parts of the world. The rapid progress of global warming is already leading to changes in climate and related environmental problems. In line with global trends, temperatures in Japan are rising. At the same time active seasonal weather fronts carrying heavy clouds are resulting in increasingly heavy local downpours. As a result of these local downpours of torrential rain there is a national increase in geotechnical failures including slope failure and debris flows. These notable tendencies due to climate change have been increasingly recognized since the beginning of the 21st century.

In 2010, a slope failure due to heavy rain occurred at Hagi city in Yamaguchi Prefecture, which has been known as an unstable landslide area. In 2009, rainfall of significant intensity was experienced at the site, but there was no evident the deformation at that time. Additionally in the past, several collapses occurred at the site due to heavy rain. This means that several cycles of rise-and-fall of the ground water level have occurred at the site. The cycle could be linked to the cyclic changes of pore water pressure.

This paper firstly focuses on the cyclic change of pore water pressure. So far, there has been little research work done on cyclic loading of pore water pressure using triaxial compression apparatus (Ohtsuka & Miyata, 2001, and Orense et al., 2004). Therefore, in order to understand the deformation behavior of soil subjected to cyclic loading of pore water pressure, an experimental research using plane strain compression apparatus was carried out for sand specimens and the results are presented. Subsequently, the observed data on the collapsed slope in 2009-2011 is explained. Especially rainfall records and the data obtained from the observation well and extensometer are shown. Finally, the lessons learned from the data are described.

2 DEFORMATION BEHAVIOR SUBJECTED TO CYCLIC CHANGE OF PORE WATER PRESSURE

2.1 Plane strain compression test

In order to simulate the cyclic rise-and-fall of ground water level, experimental tests were performed by applying the cyclic change of pore water pressure in the specimen. In past researchers (Ohtsuka & Miyata, 2001, and Orence et al., 2004), tests were carried out using triaxial compression test apparatus applying monotonic loading of the pore water pressure. Thus, plane strain compression tests were adopted to reproduce stress condition closer to the site.

Each specimen was approximately 60mm wide, 160mm high and 80mm deep. Axial loading was applied through an electric motor from the top pedestal of the specimen. Confining and pore water pressures were applied by E/P transducers. All the external forces were controlled by a developed computer program. Grease was painted on the front and back planes of specimen in the direction of zero strain to minimize the friction. Note that the intermediate principal stress could not be measured in this study. Photos of the front plane of the specimen in the direction of zero strain were taken during the shearing process to observe local deformations.

The material used was Toyoura sand which has a higher permeability to propagate the quick change in pore water pressure and to keep the homogeneity of the effective stress condition. The material can be considered to show the relatively small compression deformation component. The specimens were prepared by air pluviation method with a constant falling height of 125mm. The specimen was formed in 5 layers with the falling height kept constant for each layer. As a result, the initial relative density of specimens produced were 80%±3%. To saturate the specimen, de-aired water was flushed through the specimen and then a back pressure of 50 kN/m^2 was applied. All specimens were isotropically consolidated for 1 hour and then were sheared to reach maximum and minimum principal stress ratio of about 4.0. The cyclic change in pore water pressure was applied to the anisotropically consolidated specimen with constant amplitude. The principal stress difference was also kept constant by moving the top pedestal with constant axial deformation rate of 0.1%/min. The loading and unloading of the pore water pressure was conducted through a constant pressure increment of 1 kPa/min. After cyclic loading, monotonic loading of pore water pressure was applied to understand the strength of the specimen.

2.2 *Cyclic deformation behavior due to pore water pressure*

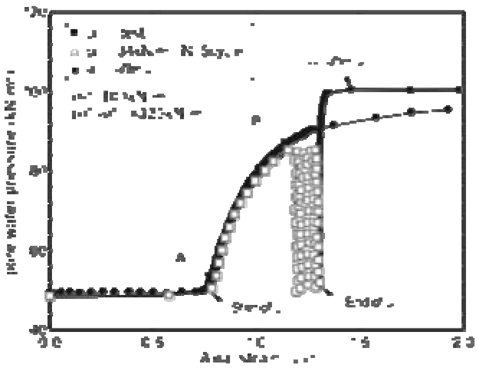

Figure 1 Deformation behavior due to cyclic and monotonic loadings of pore water pressure

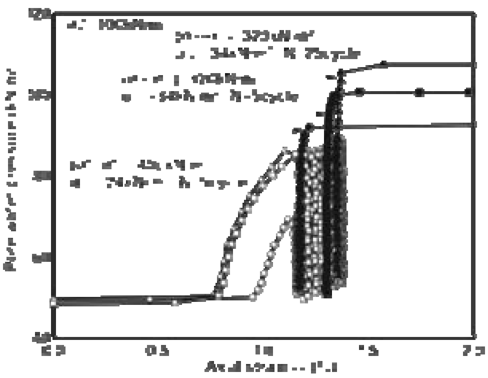

Figure 2 Effect of N and (σ'_1 □ σ'_3) on the deformation behavior

Figure 1 shows the pore water pressure and axial strain relationship for the cyclic test under 100 kN/m^2 of the lateral pressure. The principal stress difference (σ'_1 - σ'_3) was kept constant at 320 kN/m^2. This value corresponded to the residual stress obtained from plane strain compression (PSC) test under 100 kN/m^2 of lateral pressure. The cyclic change in pore water pressure, u_{cyc} was the amplitude of 34 kN/m^2. The maximum principal stress ratio reached corresponded to that of PSC test result at 100 kN/m^2. The sand specimen generated 0.40% of axial distortion from 0.75% (A) to 1.15% (B) during the 1st loading of pore water pressure. After this, the specimen showed 0.15% of axial strain during the rest of the cyclic loading. For comparison, monotonic pore water pressure loading was applied as shown in the figure. The specimen subjected to cyclic pore pressure loading showed rigid-plastic behavior and reached a higher pore water pressure level than the monotonically-loaded specimen without any cyclic loading.

Figure 2 shows the cyclic loading test results with 100 kN/m^2 of lateral pressure. As the number of cycles increase, the axial deformation becomes larger during the cyclic loading. Also, as the principal stress difference (σ'_1 - σ'_3) is kept higher, the peak pore water pressure during the monotonic loading after the u_{cyc} process becomes lower. This is because the higher stress difference (σ'_1 - σ'_3) is closer to the failure stress condition. After first loading, relatively small axial deformations were obtained irrespective of the amount of u_{cyc} and number of cycles N. This indicates that the cyclic loading of pore water pressure never lead to any larger deformation and failure.

2.3 *Failure after cyclic change of pore water pressure*

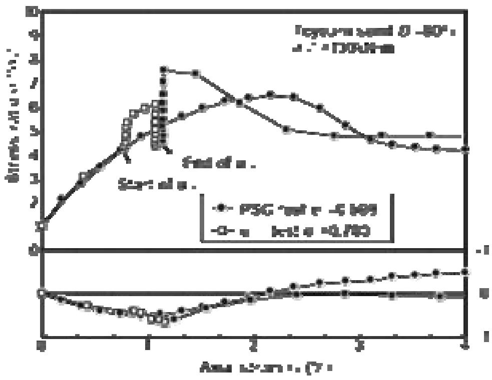

Figure 3 Stress-strain behavior from monotonic and cyclic loading of pore water pressure

Figure 3 shows the principal stress ratio σ'_1/σ'_3 and axial strain relationship for the test results shown in Figure 1. The stress-strain curves before u_{cyc} loading are the same indicating good reproducibility of the specimen. Then, cyclic loading was initiated from the residual stress condition as described earlier. The sand subjected to the loading with u_{cyc} = 34 kN/m^2 reaches a higher maximum stress ratio (σ'_1/σ'_3)$_{max}$ = 7.5 compared with (σ'_1/σ'_3)$_{max}$ = 6.0 for the conventional PSC test. The volume change during the cyclic loading is relatively small.

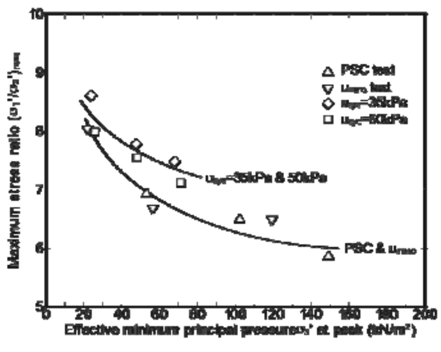

Figure 4 Stress dependency on peak stress ratio

Figure 4 shows the maximum stress ratio (σ'_1/σ'_3)$_{max}$ plotted against the minimum principal stress at the peak stress condition. It can be seen that the value of (σ'_1/σ'_3)$_{max}$ decreases as σ_3 at peak increases. As generally well known the stress dependency on the peak stress has been observed. In addition, the results of the cyclic loading of pore water pressure shows a higher (σ'_1/σ'_3)$_{max}$ than PSC tests. The results of the monotonic tests were similar with those of PSC test results. It can be recognized that the cyclic loading may induce the hardening and strengthening of the soil.

3 COLLAPSE BEHAVIOR OF SLOPE DUE TO GWL

The Kiyo District of Hagi City, Yamaguchi Prefecture has been known to be an unstable landslide area. The slope has geologic profile consisting of rhyolite and granite with a lower permeability. Previous failures have occurred at the site in 1973, 1981, 1991 and 1994. In 2000, a landslide occurred as far as the lower debris flow barrier. In 2002 ground surface movements were observed without any apparent flow of soil mass. In order to monitor the stability of the slope, a monitoring system was installed since 1988 as shown in Figures 5 and 6. The main parameters measured were rainfall, ground water level (GWL) and slope movements using extensometers. The data were recorded remotely at the National Highways Bureau in Hagi. The system was upgraded in 2004 by installing inclinometers in the slope and debris barrier. In addition, a strain-gauged tell-tale system was installed across a crack in the barrier. The data logging and communications system was updated with wireless technology. The data has been automatically updated on a website. Moreover a mailing notification system has been installed in case of unusual conditions developing at the site.

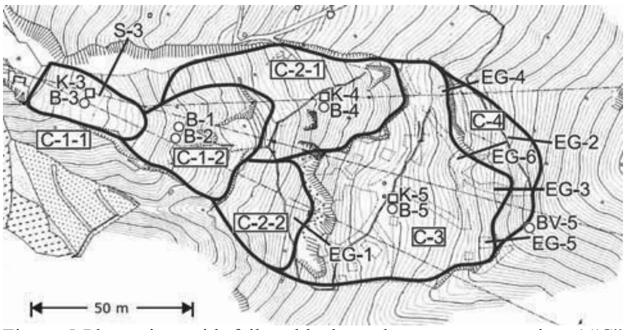

Figure 5 Plane view with failure blocks and measurement points (“C” represenrts failure block, “S” and “EG” are extensometers, “B” and “BV” are observation wells and “K” are for inclinometers)

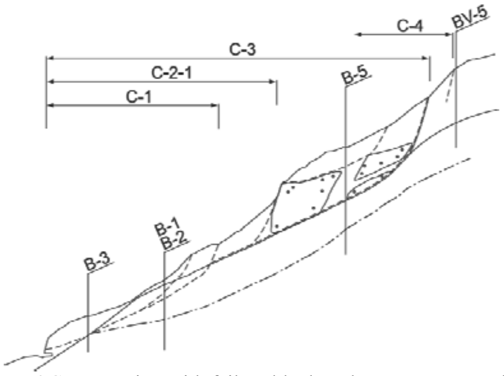

Figure 6 Cross section with failure block and measurement points

Three types of countermeasures were implemented to mitigate slope failure. Firstly the top third of the slope was covered with rubber sheeting to prevent inflow of rainwater. Secondly, water head relief tubes were installed below the area covered by the rubber sheets. And thirdly, a debris flow protection gallery was built over the road in front of the collapsed slope. This gallery was designed to resist an overburden of 4m thick of debris.

4 RAINFALL RECORD AND FAILURE PROCESS

As shown in Figure 7, the rainfall commenced at 22.00 on 10/7/10 with a maximum recorded intensity of 25 mm/hr. At the time of failure, i.e. 5:00 on 10/7/14, the rainfall rate of 18 mm/hr on 7/12 led to a cumulative rainfall of 269mm. By 10:00 on 7/15 the accumulated rainfall was 351mm. The first failure occurred at 5:00 on 7/14 at the lower slope area C-1-2 over a slope length of 80m, width of 50m and depth of 2m. The total volume of the slip was 1600m^3. The second failure occurred between 15:00 on 7/14 and 10.00 on 7/15 at Block C-2-1 over a slope length of 30m, width of 35m and depth of 2m. With these failures, the lower part of the slope was covered by a 2m to 3m thick layer of debris. This layer was unstable and was liable to flow failure under future heavy rainfall. The upper part of the debris deposit had a vertical slope face. Also a visible continuous crack on the topside of block C-3 was discovered. This means higher potential for further movement of this block under the next heavy rainfall. So the stability of blocks C-2-2 and C-3 was worsened by the rainfall in July 2010.

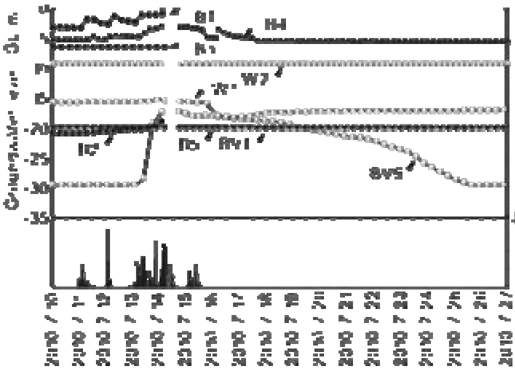

Figure 7 Variation of ground water level in July 2010

5 GROUND WATER LEVEL RECORD

B1 and B2 were located in Block C-1-2. The elevation of GWL firstly occurred in B1 following the rainfall as shown in Figure 7. The GWL rose to within 0.51m below ground level (BGL). In the case of B2 the GWL rose to 18.54m BGL before the first failure, which represented a rise of 3m from the dry period. In previous failures in 1991 and 1994 similar GWL observations were made. It is therefore thought that the GWL was the trigger for the series of failures. The GWL in C3 had not apparently risen above normal levels.

BV-5 was located at the top of the slope and recorded a significant rise in GWL which correlated to the failure of C-3. During the dry period the GWL was at a depth of 29m. The GWL rose from 12:00 on 7/13 reaching a depth of 17.35m BGL at the initiation of failure and started to fall slowly again after 17:00 on 7/14. From 7/14 to 7/21 the rate of decrease was between 0.3m and 0.8m/day and from 7/22 to 7/25 the rate was 1.85m/day, finally reaching 29m BGL on 7/25. The movement of the extensometers stabilised when the ground water level dropped to 20m BGL. B4 was located at the top of the C-2-1 block. The GWL rose to 2.03m BGL which was a 3.5m rise. The rate of decrease was more than BV5 and on 7/17 it fell to 5m BGL. The major movement of Block C3 was considered to be the result of the earlier failure of C1-2 and C2-1 which effectively unloaded the toe of C3.

6 GROUND MOVEMENT

Before the rainfall there were no indications of movement from the monitoring system located in Block C-3. However after the second failure, movement of the C-3 block was observed with rising GWL. After the first failure the monitoring system failed to work for a temporary period of time and was reinstated at 15.00 on 7/14. At this time it was noted that Block C-3 was moving at a rate of 30 mm/hr. This movement was monitored by the inclinometers at K-3. According to the inclinometer data, the slip surface was located at a depth of 20m. The movement at the surface was the same as that at depth, indicating that Block C-3 was moving as a single mass. From 7/15 the rate of movement decreased from 15 mm/hr to 1 mm/hr on 7/19, coming to a virtual halt on 7/22. This decrease correlated with the decrease in GWL. The S-3 extensometer became inactive due to the failure. No movement was observed by the extensometers EG1 and EG2 as shown in Figure 8. Movement was measured by EG3, EG4, EG5 and EG6 after 17.00 on 7/14 with greater than full-scale readings.

From 7/14 to 7/15 the rate of deformation was 5 mm/hr to 15 mm/hr and on 7/16 to 7/18 it reduced to 1 to 5 mm/hr. From 7/18 it gradually decreased. From 7/19 it reduced to 1mm/hr ceasing on the 7/22. The S-4 extensometer was set up on 7/17 because of the failure of C-2-1 and the enhanced risk of failure of the RHS of the slope. During the time the ground water level was going up, progression of displacement was not seen at all. Moreover, displacement between -17m and -20 m of the groundwater level was very remarkable. Furthermore, if the groundwater level falls lower than -23m, it can be considered that the movement of the block stopped in general.

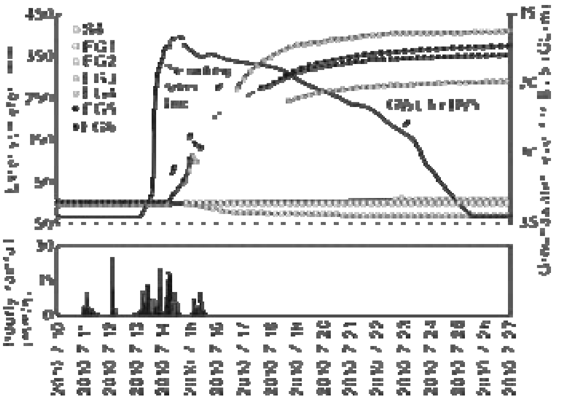

Figure 8 Data of extensometers observed in July 2010

6.1 *Rainfall record in 2009*

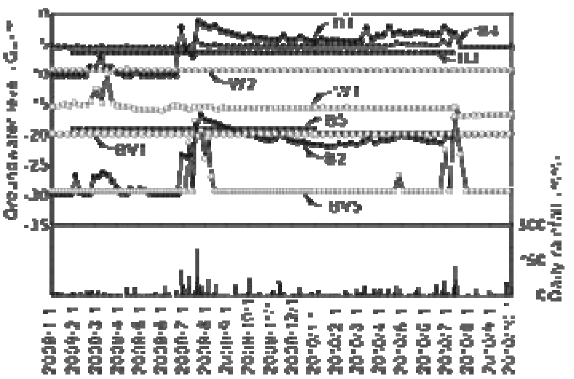

Figure 9 Rainfall record between 2009 – 2010

In 2009, a significant rainfall had been experienced at the same site as shown in Figure 9 with a maximum intensity of 62 mm/hr and cumulative intensity of 289mm over 3 days. However there was no evident of deformation at that time. The evidence indicates that it is difficult to predict the collapse solely by the rainfall forecast. The GWL in B1 in 2009 was 10m BGL compared to 3m BGL in 2010. The potential for slope failure in 2010 was therefore higher. The general tendency over the period 2009 to 2010 was for the GWL to be above normal.

6.2 *Collapse behavior in 2011*

Deformation of the slope was observed in 2011 due to the smaller rainfall shown in Figure 10. The rainfall in 2011 started on 6/27 and continued intermittently to 7/17. For 21 days, 4 major daily rainfall events were recorded; 60mm on 6/27, 40mm on 7/1, 30mm on 7/4 and 50mm on 7/8. The extensometer begun to act on 7/1 before the GWL for BV5 elevated on 7/3. From this date, the deformation increased gradually and accelerated on 7/7, rising with the rapid increase in the GWL. During the ground water level rise and descent, the displacement of the C-3 block was recorded. The maximum displacement rate appeared during the water level rise while in 2010 it was during water level descent. This implies that the timing of the warning official announcement would be earlier more for the next heavy rainfall.

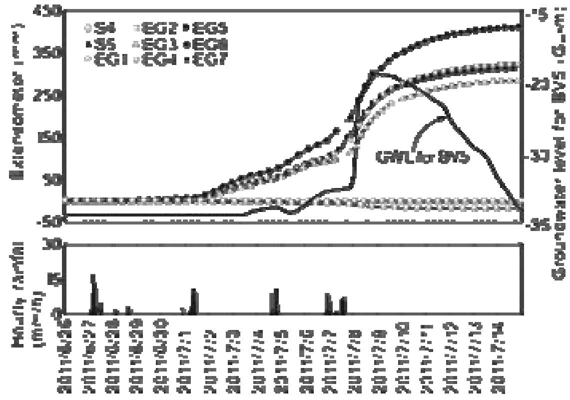

Figure 10 Data of extensometers observed at July in 2011

7 CONCLUSIONS

The paper focused on the evidence of collapse behavior for a slope due to heavy rain in 2010. In the past the site had experienced several collapses. An experimental research was performed to understand the deformation behavior of sand subjected to cyclic loading of pore water pressure using plane strain compression apparatus. Then the observed data on the collapsed slope in 2009-2011 was explained. Finally the lessons from the data were discussed.

1) The experimental test results showed that cyclic loading of pore water pressure never lead to any larger deformation and failure except the first loading. The rigid-perfectly plastic type behavior was observed for the sand specimen subjected to the cyclic changes in pore water pressure.
2) In 2009, a significant rainfall fell at the same site. However, there was no evidence of deformation at that time. This indicated that it was difficult to predict the collapse by the rainfall forecast only and a monitoring system is needed to predict any deformation.
3) The collapse behavior started at the surface layer of the toe part of the slope and then the movement of the major slip block correlated with the rise of ground water level behind the block. Therefore it is important to monitor the ground water level at the appropriate points to understand the collapse behavior.

8 ACKNOWLEDGEMENTS

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9 REFERENCES

Ohtsuka S. and Miyata Y. 2001. Failure mechanism of clay in pore water pressure loading test. Landslide 37(4), 10-17 (in Japanese).

Orense R., Farooq K. and Towhata I. 2004. Deformation behavior of sandy slopes during rainwater infiltration. Soils and Foundations 44(2), 15-30.