A STUDY OF DEFORMATION BEHAVIOUR OF AN INSTRUMENTED SLOPE SUBJECT TO RAINFALL NEAR THADAN DAM THAILAND

Apiniti Jotisankasa[1]
Akihiro Takahashi[2]
Tomohide Takeyama[2]
Warakorn Mairaing[1]

[1] *Department of Civil Engineering, Faculty of Engineering, Kasetsart University, Thailand fengatj@ku.ac.th*
[2] *Department of Civil Engineering, Tokyo Institute of Technology Japan*

ABSTRACT : A methodology has been developed using DACSAR Finite Element programme to predict the deformation of soil slope subjected to known pore water pressure distributions. Field monitoring results of pore water pressure and shear strains within the soil slope near Thadan dam, Nakornnayok Thailand during May 2008 until October 2008 have been used to validate such method. In the simulation, Drucker-Prager constitutive model is assumed for the slope material based on compression and shearing behaviour from consolidated drained direct shear tests. Several assumptions have been made in order to specify the hydraulic conditions for all elements at each step. Firstly, only positive pore water pressure is used and negative pore pressure is neglected in the analysis (pore water pressure assumed to be zero if negative). Secondly, the pore water coefficient, r_u is used to generalize spatial distribution of the pore water pressure from point-wise measurement. The Young's modulus, Poisson ratio, and initial coefficient of earth pressure have been estimated based on the results of direct shear tests and some empirical correlations. The sudden surge of shear strain (1.2%), due to heavy rainfall of September 2008, observed in the slope can be satisfactorily simulated using the FE programme, though the simulated shear strain is about half the measured value. Finally the programme was used to estimate the value of shear strain at the measurement point which is expected to occur when the slope starts to undergo overall failure. This value of shear strain (4-4.5 %) can be used as an approximate threshold value of shear strain for early warning system at the studied slope near Thadan dam site.

KEYWORDS : *Landslide, Slope stability, Numerical analysis, Warning system, Deformation*

1. INTRODUCTION

Problems of slope instability due to heavy rainfall in Thailand have become more serious since the last decade, especially in cut/fill slopes along highways in steep areas. A number of studies have been carried out in Thailand in order to mitigate such threats to the infrastructure such as hazard zonation, slope stabilization, development of early warning system (Mairaing, 2008, Taesiri and Yuwathanon, 2005, Jamnongpipatkul et al., 2008, Soralump & Bunpoat, 2006, Jotisankasa & Vathananukij, 2008). The major mechanism of slope instability in Thailand is related to rain infiltration into soil slope which leads to increase in pore water pressure and consequently reduction in effective stress and shear strength.

In order to gain more in-depth understanding of the actual slope behaviour subjected to rainfall, Jotisankasa & Porlila (2008) developed a slope monitoring system, consisting of MEMs tensiometer/piezometer for pore water pressure measurement (Jotisankasa et al., 2007) and MEMs inclinometer for shear strains. The prototype monitoring system has been installed since May 2007 in a slope of failed soil mass that had been re-graded in 2004 (Figure 1). Slope failure in 2004 was triggered by an intense rainstorm which amounted to about 300mm in three days. The slope is situated in Nakornnayok province, east of the central region, where the geology consists of undifferentiated Permo-triassic volcanics rocks, including rhyolite, andesite, tuffs, and agglomerate (Royal Irrigation Deparment, 2004). The material on the slope is classified as medium plasticity silts (MH/ML) with basic properties summarized in Table 1. Jotisankasa (2008) carried out detailed studies of the material properties including, shear strength-suction relationships, soil-water characteristic curves and field infiltrability.

Table 1 Basic properties of the material at monitored site

Liquid Limit	Plasticity Index	% gravel	% sand	% silt	% clay	Soil Type
46-51	6-18	0.5-5.5	13.7-18.9	38.9-53.9	31.9-36.7	Silts MH/ ML

The finite element mesh of the soil slope profile, shown in Figure 2, has been based on investigation by research students of Kasetsart University using a light weight dynamic penetrometer, so-called Kunzelstab (weight of 10kg, with falling height of 0.5 metre) and a compass-clinometer. It is noted that the mesh only shows the soil mantle, characterized by Kunzelstab blow counts of around 5 or lower (per 0.20 m) (equivalent to SPT blow counts of 3), which has been estimated to be only around 2 to 3 metres in thickness. The sounding tests were carried out during rainy season when pore water pressure in the ground are close to zero (around -2 to 0 kPa). Below the soil mantle lies bedrock of volcanic origin (KPT >20-60), which is assumed to be stationary in the analysis. The freedom of movement is thus disallowed in vertical and horizontal directions for all finite element nodes at the base of soil slope.

Figure 1 The studied slope (a) in 2004, after failure (b) in current state after regrading and vetiver grassing

2. INSTRUMENTATIONS

The instruments were installed on the slope in two locations as shown in Figure 2. Pore water pressures were monitored using miniature tensiometers developed at Kasetsart University (Jotisankasa et al., 2007). This device is capable of measuring both positive and negative pore water pressure (range of -80 to 600 kPa). The inclinometers were installed at only Point 1 where degrees of tilting have been monitored at depth of 0.7 and 1.35 metre on the side of PVC tube, fixed at the lower end about 0.3 metre into the bedrock. Details of installation procedure for these devices can be found in Jotisankasa et al. (2007) and Jotisankasa & Porlila (2008).

Figure 3 shows the overall results of pore water pressure and rainfall over 1.5 year period. The correspondence between the pore water pressure and rainfall is evident, whereby pore water pressure became progressively more negative during the dry season as evapo-transpiration continued without rainfall. During rainy season, however, the pore water pressure became close to zero and at times become positive after intense rainstorm (greater than ~100 mm/day). The degree of tilting from inclinometer remained relatively unchanged until a heavy rainfall (~230mm) in the week of 13-19 September 2008, when the pulses of tilting, as well as surges in pore water pressure were observed. In this respect, Finite Element analysis using DACSAR-M programme (Iizuka & Ohta, 1987, and Takeyama et al., 2006) were performed, based on the monitored pore water pressure values as input data, in order to reproduce the deformation of the soil slope that has been observed in the field. The analysis was also used to predict the shear strains when slope undergoes overall failure and used as a criteria for early warning.

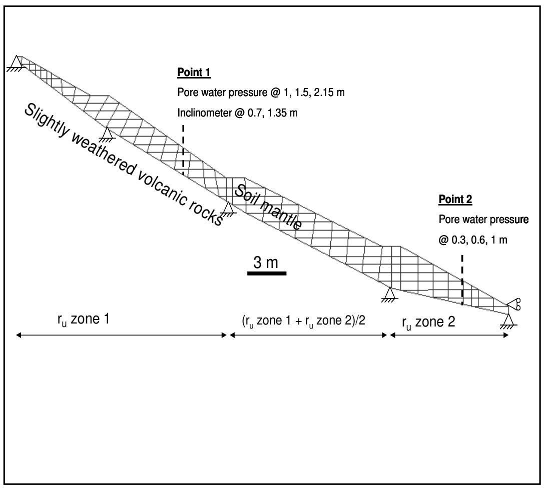


Figure 2 Slope profile with meshes, boundary conditions and locations of measurement points

3. FINITE ELEMENT ANALYSIS

The constitutive model used for the soil slope is Drucker-Prager material in plane-strain condition. The input material properties are shown in Table 2 as obtained using drained direct shear tests for effective normal stress range of 16-65 kPa. The coefficient of earth pressure at rest, is estimated using the Jaky's equation, $K_o = (1 - \sin\phi')$, and Poisson ratio using the elastic theory, $\nu' = \dfrac{K_o}{K_o + 1}$. The coefficient of volume change,

m_v as obtained from the consolidation stage of the direct shear tests was used to estimate the value of Young's modulus, E' , as in Equation 1 and Figure 4. It is noted that the value of m_v varies non-linearly with vertical stress, and thus the effective vertical stress range of 0 to 15 kPa is used to calculated the E' for the rainfall-induced stability problem where confining effective pressure in slopes is relatively low (see Figure 5).

$$m_v = \frac{\Delta e}{(e_i + 1)\cdot \Delta\sigma'_v} = \frac{(1+\nu')(1-2\nu')}{E'\cdot(1-\nu')} \qquad (1)$$

The slope was first constructed numerically in 31 horizontal layers (thus divided into 31 steps), starting from the toe to crest (Figure 2). This was believed to represent the actual slope regrading procedure carried out after slope failure in 2004. The initial pore water pressure within the soil layer is set to zero and forced to remain so for the rest of construction period. In actual situation, a small amount of suction might present in the soil layer during construction, but subsequent rain infiltration would eventually bring the suction to zero, causing some collapse-on-wetting of the loose soil to its fully saturated compression line, which is in fact the assumed condition. By this method, it was found that the final stress levels within all elements (Figures 5 and 6) appeared to be more reasonable than if assuming the construction of the inclined soil mass in only one step.

Since the measurements of pore water pressure are available at the studied site, it was decided that these monitored piezometric/tensiometric data are used as inputs for the analysis of strains for the slope under imposed hydraulic conditions. The calculated shear strains can thus eventually be compared with the measured values. Nevertheless since the measurements of pore water pressure are only available at two locations, several assumptions had to be made in order to specify the hydraulic conditions for all elements. Firstly, when the measurement showed negative pore pressure, the value of zero will be assumed for pore pressure in the analysis (i.e. suction effect is neglected) and thus only when pore water pressure becomes greater than zero will significant slope deformation occurs. Secondly, the values of pore water coefficient, $r_u = \dfrac{u}{\sigma_v} = \dfrac{u}{\gamma_t \cdot H}$, at measurement points are used to generalize spatial distribution of the pore water pressure. Three zones of r_u have been specified as shown in Figure 2, with the middle zone being the average of the other two.

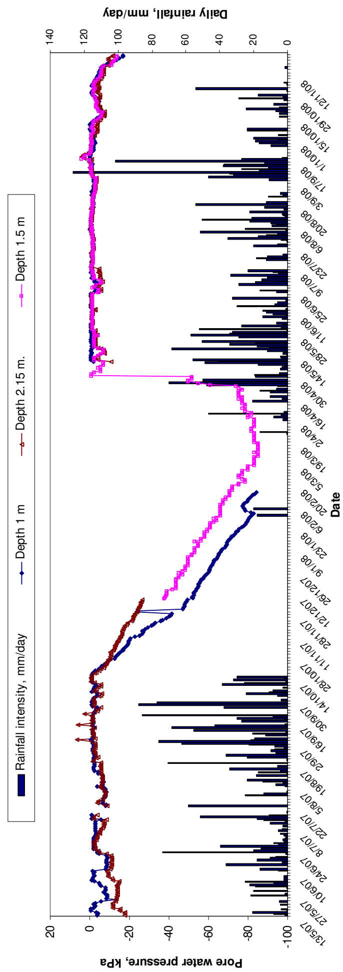


Figure 3 Pore water pressure (point 1) and rainfall during 2007-2008 monitoring period

Table 2 Summary of material properties used in FE analysis

Properties	Values	Properties	Values
Effective cohesion c', kPa	12.8	Young's modulus, E', kPa	299
Effective angle of shearing resistance ϕ', degree	33.1	Poisson ratio, ν'	0.312
Saturated unit weight γ_{sat}, kN/m^3	17.61	Coefficient of earth pressure at rest, K_i	0.454
Permeability k, m/s	2.1 E-4	Initial void ratio, e_i	1.00

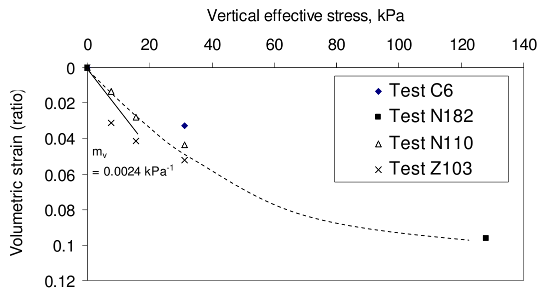


Figure 4 Calculation of coefficient of volume change, m_v as obtained from the consolidation stage of the direct shear tests

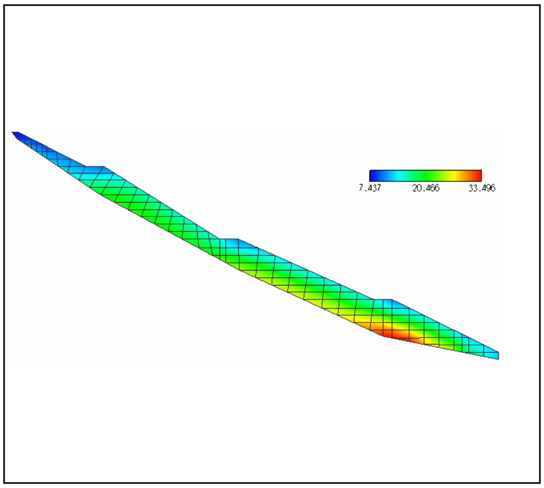


Figure 5 Contour of mean effective stress, p',after construction of the slope (with zero pore water pressure assumption) (Unit in kPa)

The field measurements of rainfall, pore water pressure and inclinometer readings of shear strain as well as simulated shear strains are shown in Figure 7. It is evident that the heavy rainfall during 11/9/08 until 19/9/08 brought about the increase in pore water pressure and corresponding shear deformation in the soil slope. At depth of 1.35 metre, the shear strain on 19/9/08 was reasonably well captured in the numerical analysis, though the simulated values are only about half the measured values. The strains from field measurements at 0.70 m depth appeared to rotate backwards (negative values) on 12/09/08 and were somehow reproduced in the FEM analysis. Yet this backwards rotation shift to positive value as the pore water pressure increased further. The backward rotation was thought to be due to the initial pore water pressure surge at the toe of the slope while the water pressure at the upper part was still relatively unchanged. It is interesting to note that Uchimura et al. (2008) also observed this type of behaviour at ground surface in their instrumented slope model.

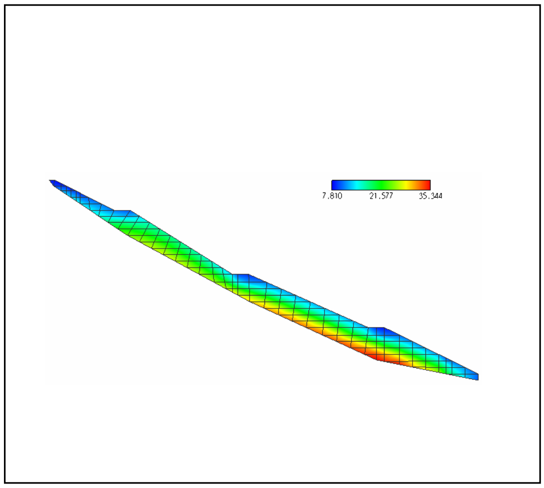


Figure 6 Contour of deviatoric stress, q, after construction of the slope (with zero pore water pressure assumption) (Unit in kPa)

4. SHEAR STRAINS AT FAILURE STATE

Another usefulness of this type of numerical simulation is to predict the value of threshold shear strain for early warning of slope failure. An analysis has been carried out whereby the pore water pressure coefficient r_u is increased incrementally in the slope from 0.1 to 0.9. The results are shown in terms of shear strain plotted against r_u in Figure 8a. Nevertheless, since the soil is assumed to have true effective cohesion, c' (see Table 2), hence even at relatively high pore water pressure when r_u = 0.9, the slope did not seem to fail in the FE analysis. In this respect, Limit Equilibrium approach, based on Infinite slope model (e.g Skempton & Hutchinson, 1969), as in Equation 2, was used instead with the sliding mass having representative gradient β of 24^{o} and thickness of 2 metres.

$$F = \frac{c' + (\gamma \cdot z \cos^2 \beta) \cdot \tan\phi' - u_w \cdot \tan\phi'}{\gamma \cdot z \sin\beta \cdot \cos\beta} \qquad (2)$$

The critical value of r_u, when factor of safety becomes 1, is found to be 0.75. This value corresponds to the shear strain of about 0.044, as shown in Figure 8a.

Another FE analysis was also carried out, but with the soil assumed to have zero effective cohesion, $c' = 0$, to represent another worst case scenario. The results are shown in Figure 8b. It was also assumed that the slope started to fail at the inflection point when shear strain is around 0.039 and r_u becomes about 0.5. Nevertheless, the critical r_u obtained by Limit Equilibrium method (infinite slope) assuming zero effective cohesion, was found to be 0.25 only. The value of r_u from FE analysis is thought to be more realistic. These values of r_u and shear strains can be used as an approximate threshold values for early warning of the current landslide monitoring system, in addition to total critical rainfall of 300 mm (Jotisankasa & Vathananukij, 2008). It should also be noted that since the current DACSAR code is not intended for reproducing the reduction in the p' and q along the failure line, and not for progressive failure type, the results of stress states at failure are not presented in this paper, before more confidence in the analysis is obtained.

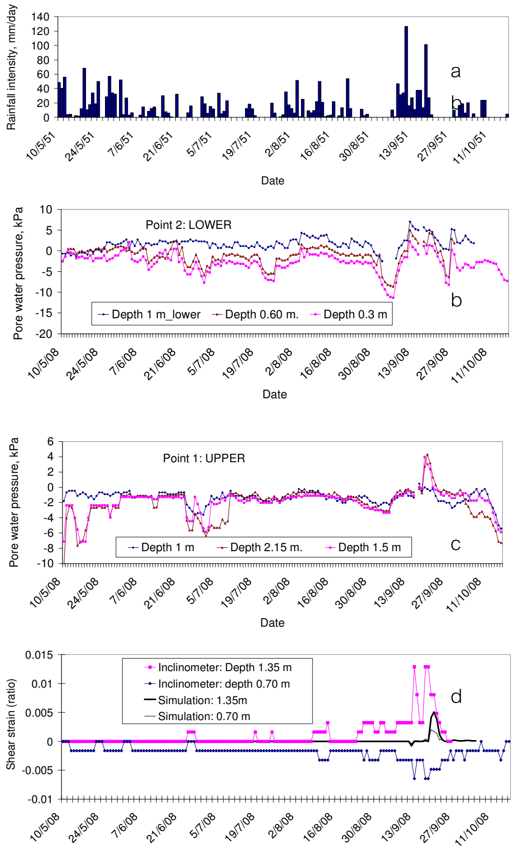
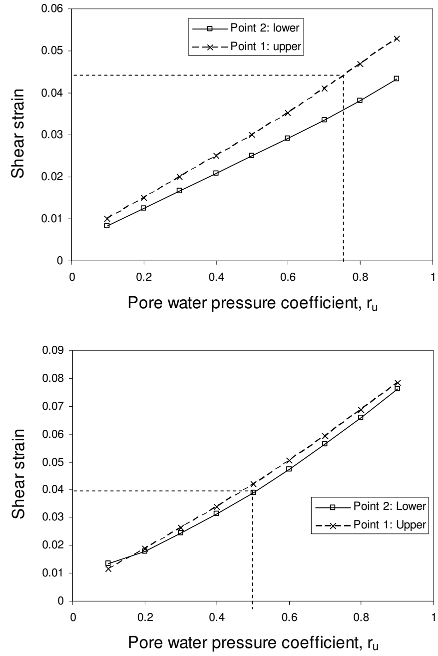


Figure 7 Field measurements of rainfall (a), pore water pressure (b,c) and shear strains from inclinometer and simulation (d)

5. CONCLUSIONS

A slope monitoring system, developed at Kasetsart University consisting of MEMs tensiometer/piezometer for pore water pressure measurement and MEMs inclinometer for shear strains, has been used to monitor behaviour of slope subjected to rainfall from year 2007-2008 in Nakonnayok, Thailand. The pore water pressure appeared to be negative during dry season, while becoming close to zero during rainy seasons, and at times showing positive values during intense storms. Only during these sudden jumps in positive pore water pressure did the slope start to deform significantly.



a)

b)

Figure 8 Relationship between deviatoric strain and pore water pressure coefficient from the Finite Element Analysis for a) assuming $c' = 12.8kPa$ and b) assuming $c' = 0$

Based on these measured positive pore water pressure as input parameters, DACSAR Finite Element programme, assuming Drucker-Prager material, has been used to predict the shear strain within soil slope. The values from numerical analysis are in a reasonably good agreement with the measurement from inclinometer. The range of threshold shear strain, expected to occur when the slope starts to undergo overall failure, has been estimated to be around 4-4.5 % in the analysis which assumes gradual rise in pore water pressure in the slope. This value of shear strain, in addition to critical values of pore water pressure (r_u of 0.5 to 0.75) and rainfall amount (300mm), can be used as criteria for early warning system at the studied slope near Thadan dam site.

6. ACKNOWLEDGEMENTS

The authors are grateful to the Japan Society for the Promotion of Science (JSPS) for giving the first author an opportunity as an exchange scientist at Tokyo Institute of Technology under the Core University Program. The research grant no. MRG4980086 of the Thailand Research Fund and Commissions of Higher Education, Thailand, provided for continuous monitoring of the slope is gratefully acknowledged. The first author also appreciates the hospitality of all host academic staffs as well as students of Tokyo Tech during his stay.

REFERENCES

[1] Skempton, A.W. and Hutchinson, J.N. (1969). Stability of natural slopes and embankment foundations. *Proc. 7th Int. Conf. Soil Mechanics and Foundation Engineering., State-of-the Art Volume*, 291-340.

[2] Iizuka, A. and Ohta, H. (1987). A determination procedure of input parameters in elasto-viscoplastic finite element analysis, *Soils and Foundations*, Vol.27, No.3,71-87.

[3] Jamnongpipatkul, P. Dechasakulsom, M. Sukolrat, J. and Sawangsuriya, A. (2008). Slope management in thailand: a case study of reinforced slope from highway route No.1095. *Proceedings of the International Conference on Slope, Landslide Awareness: Slope Mitigation & Prevention*, Kuala Lumpur, Malaysia 2008

[4] Jotisankasa, A., Porlila, W., Soralump, S., Mairiang W. (2007). Development of a low cost miniature tensiometer and its applications. *Proc. 3rd Asian Conference on Unsaturated Soils (Unsat-Asia 2007),* Nanjing, China, 475-480.

[5] Jotisankasa, A. & Porlila, W. (2008). Development of a landslide monitoring system. *In the Proceedings of the 2nd Technology and Innovation for Sustainable Development Conference.* January 28-29, 2008, Khon Kaen, Thailand. (in Thai)

[6] Jotisankasa, A. and Vathananukij, H. (2008). Investigation of soil moisture characteristics of landslide-prone slopes in Thailand. *Proceedings of the International Conference on Management of Landslide Hazard in the Asia-Pacific Region. Sendai, Japan,* November 11-15

[7] Jotisankasa, A. (2008). Final report, Investigation of behaviour of unsaturated soils in Thailand, Project Code: MRG4980086, Thailand Research Fund (in Thai with English summary)

[8] Mairaing, W. (2008). Landslide problems and warning by geotechnical methods. *Proceedings of the 26th Conference of ASEAN Federation of Engineering* Organizations, Bangkok, Thailand.

[9] Soralump, S. and Kulsuwan, B. (2006). Landslide Risk Prioritization of Tsunami Affected Area in Thailand, *International Symposium on Environmental Engineering and 5th Regional Symposium on Infrastructure Development in Civil Engineering.,* Philippines

[10] Royal Irrigation Department (2004). Report on source areas for rock and aggregates in Klong Thadan Dam construction project, prepared by Team Consultant Engineering and Management. (in Thai)

[11] Taesiri, Y, and Yuwathanon, K. (2005). Prevention and Stabilization methods for slope movements. *Internal report of seminar by Bureau of materials, analysis and inspection,* Department of Highways, Ministry of transport and communication. (in Thai)

[12] Takeyama, T., Iizuka, A. and Ohta, H. (2006). Spatial descritization of water using approximation by linear function, *Proc. of 41th Japanese National Conference on Geotechnical Engineering,* 321-322 (in Japanese)

[13] Uchimura, T., Towhata, I, Ling, W, & Seko, I (2008). Simple and Low-Cost Wireless Monitoring Units for Slope Failure. *The First World Landslide Forum.* 18-21 November 2008, Tokyo, Japan.