Shear and soil-water retention behaviour of a variably saturated residual soil and its implication on slope stability

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ABSTRACT: Numerous shallow slope failures took place in residual soils derived from sedimentary rock formation of Uttaradit province, Northern Thailand in 2005, due to prolonged and intense rainfall. Shear behaviour as well as water retention behaviour of this material has thus been investigated in details in order to investigate the slope failure mechanism. Fully saturated consolidated-drained (CD) as well as suction-monitored direct shear tests have been performed on undisturbed samples collected from depths of 0.3-1 m. A miniature tensiometer has been used for suction measurement during these tests. In addition, influence of number of drying/wetting cycles on saturated shear strength is investigated. The results from a simple infinite slope analysis suggest that the major slope destabilization mechanism is a combination of material degradation and pore water pressure increase.

1 BACKGROUND

1.1 *2006 Uttaradit landslide*

In May 21-23, 2006, a heavy rainfall of about 400mm triggered flash flood and hundreds of shallow landslide in northern provinces of Thailand, including Uttaradit. Nearly a hundred people were reported dead or missing. Considerable damage was also incurred to infrastructure such as bridges, roads, drainage systems, and agricultural lands.

Figure 1. Shallow landslides in Uttaradit in 2006

Although the hourly rainfall data during the incident is not available, the local communities reportedly observed an unprecedented heavy rainfall over a period of two to three hours prior to landslide (ADPC, 2006). The mass movements were categorized into four types: shallow earth slips, gully erosion, failures of cut slopes, as well as erosion-induced bank failure. The predominant category (about 80% of all failures) is the shallow slip type which occurred within the residual soil mantle to a bedrock depth of 40-100 cm (Fig. 1). The topography of these areas consists of hilly terrains with prevailing gradients ranging from about 25 to 45 deg.

Deforestation is believed to be another contributing factor on the shallow landslides in Uttaradit (ADPC, 2006). In addition, slope cutting for the purposes of settlement expansion and road construction is another cause of slope failure in the area. The geology of the landslide vicinity is mainly of Carboniferous/Permian sedimentary rocks including siltstone, mudstone, and shale, which show distinct fold structure. It has been shown that such argillaceous materials are likely to degrade with time after exposure to weather or wet/dry cycles (e.g. Sabatini et al., 2002 and Alonso & Pineda, 2007). Recently cut slopes that at first appear stable will eventually exhibit surficial sloughing after some years if unprotected.

1.2 *Shallow slope failure mechanism*

Such problems of shallow landslide have received considerable attentions by the research communities worldwide (e.g. Vaughan, 1985, Cho & Lee, 2002, Ng & Shi, 2003, Collins & Znidarcic, 2004, and Rahardjo et al., 2007). The slopes involved in shallow failure are in general within the vadose zone, which exhibit soil suction, or negative pore water pressure, most time of the year. Prolonged rainfall and subsequent infiltration diminished soil suction to nearly zero at a critical depth of slope and often become a triggering mechanism of shallow slope failure. Even though many fast-moving shallow landslide and debris flow are finally triggered by positive pore water pressure rise or static liquefaction (e.g. Vaughan, 1985, and Johnson & Sitar, 1990) the incorporation of unsaturated soil properties into slope stability and infiltration analysis has been shown to yield a better representation of actual soil slope behavior. As evidenced by Collins & Znidarcic (2004) and Godt et al., (2009), slopes with a relatively steep gradients (e.g. > 45deg.) might fail when the pore water pressure becomes close to zero and not yet highly positive.

The main objective of this study is therefore to investigate the shear and soil-water retention behaviour of the residual soils from the Uttaradit landslide area both in saturated and unsaturated conditions. A typical slope within the area has also been instrumented in order to obtain a better picture of the pore water pressure variation in the field. In addition, the influence of wetting/drying cycles on shear behaviour of the soil will be studied, together with their implications on slope stability.

2 INSTRUMENTED SLOPE

2.1 *Site description*

The studied site is situated near the foothill in the agricultural area with a variety of fruit plantations and an average slope angle of about 26deg. The character of this site is considered representative of the major landuse type in the hilly area of Uttaradit. Figure 2 shows the photo of the site and lay out of instruments. Five stations of tensiometers, and inclinometers were installed at five elevations. The tensiometers have been developed by Jotisankasa et al. (2007) and manufactured at Kasetsart University using commercial miniature MEMs pressure sensors and standard 1bar AEV porous stone. The devices, capable of measuring pore water pressure ranging from -80 to 600kPa, were installed at depths of 0.5m and 1m. One automatic tipping bucket raingauge was also installed at the toe of slope. Inclinometers installed were based on MEMs accelerometer attached to a PVC tube buried to a depth of relatively competent bedrock at about 1m. The readings from the sensors were recorded with a logging system that can wirelessly transfer the data to mobile phones.

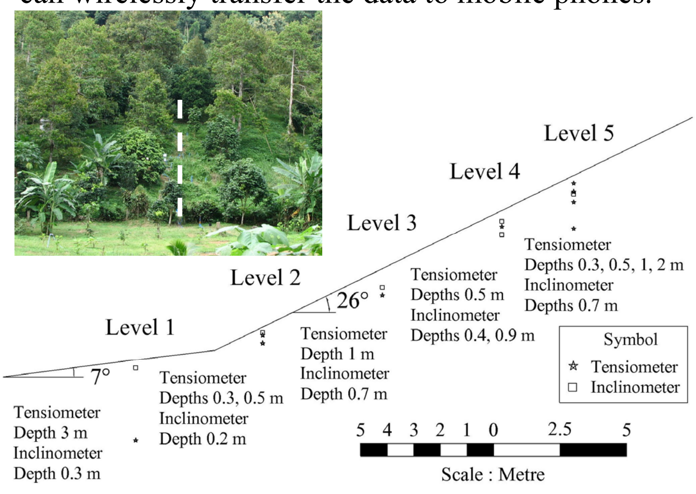

Figure 2. Photo of the studied site and its cross-section

In addition, a test pit was also excavated to a depth of about 1m for visual inspection (Fig.3), thin-wall tube sampling, and double-ring infiltration tests. Falling head tests were performed within lined boreholes to determine the so-called saturated permeability at depths of about 2 and 3m. The interpretation of the falling head test was based on Hvorslev (1951) and Garga & Blight (1997). As shown in Figure 4 and Table 1, at greater depth, the soil becomes coarser, denser and of greater value of specific gravity. The deeper completely weathered rock was with more visible feature of cementation. The upper material likely to be involved in slope failure is classified as low plasticity silt.

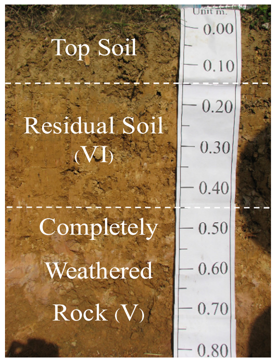

Figure 3. Typical soil profile of the studied area, based on Little (1969) interpretation.

Table 1. Basic soil properties and classification

Depth	Atterberg's limits, %				Void	USCS
(metre)	LL	PL	PI	Gs	ratio, e	
Top soil	48.2	32.4	15.7	2.616	1.090±0.05	ML
0.3	40.5	29.4	11.1	2.724	0.963±0.05	ML
0.8	40.8	29.6	11.2	2.728	0.703±0.04	ML
1.0	39.9	28.1	11.8	2.746	0.736±0.07	SM

Figure 5 shows the decrease in permeability with depth. The value of permeability reduces, by about three to four orders of magnitude, from the upper 1m of the soil to the bedrock at the depth of 2-3m. Large scatter in the permeability data is evident especially for the ground at depth less than 1m, which is probably attributed to the heterogeneity of the residual soil. As also noticed by Vaughan (1985) and Gerscovich et al. (2006), the accentuated variability in permeability can lead to strongly non-uniform distribution of pore water pressure and sometimes highly positive pore water due to impeded seepage.

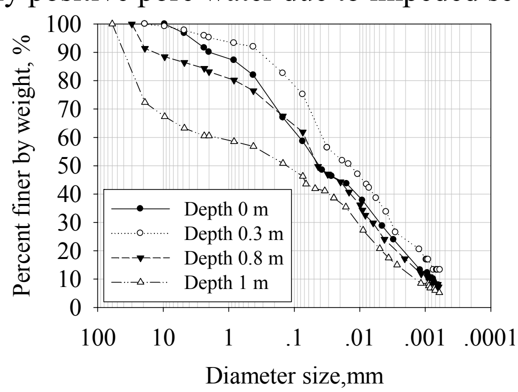

Figure 4. Grain size distribution curves

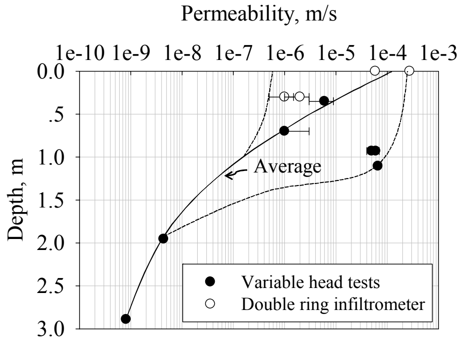

Figure 5. Variation of permeability with depth

2.2 *Monitoring results*

Figure 6 shows the variation of pore water pressures and daily rainfall with time. Both at the depth of 0.5 and 1metres, the pore water pressure remained negative for most time of the year. Despite the rainfall of about 68mm on the day of 3/11/2008, the pore water pressure remained only about zero kPa, and not much greater than that. The rate at which the pore water pressure reduced during the dry season (from 8/11/2008 onwards) appeared to be higher as pore water pressure decreased.

The inclinometer reading did not indicate any significant movement of the slope during the measurement. This observation agrees with the negative pore water pressure of the soil which gave rise to stability of the slope.

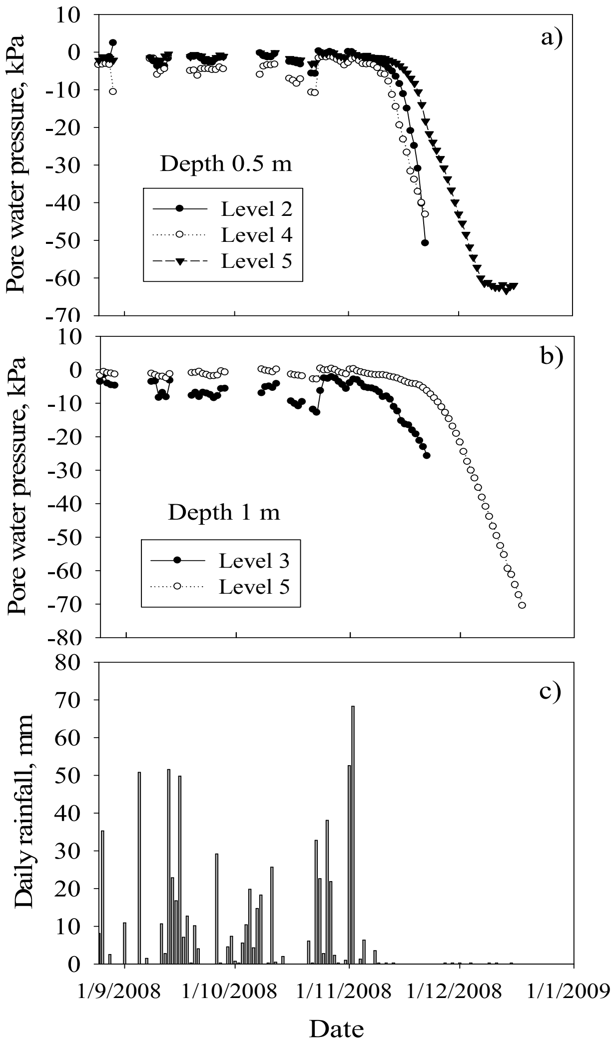

Figure 6. Variation of pore water pressure and daily rainfall

3 SHEAR AND WATER RETENTION BEHAVIOUR

3.1 *Saturated shear behaviour*

A series of multistage and single-stage direct shear tests on saturated and unsaturated undisturbed specimens about 63mm in diameter was performed to investigate shear behaviour. A conventional direct shear box was used to determine the effective strength parameter (c', ϕ') in slow consolidated-drained multistage shearing tests, conducted at three normal stresses of 16, 32, 64 kPa. The samples tested were taken from the agricultural area at ground surface (indicated as 0 metre) and at depths of 0.3, 0.7, 0.8, and 1metre. The soil at 1m depth has the highest effective cohesion intercept due to its cementation: a feature expectable of a structured soil or completely weathered rock.

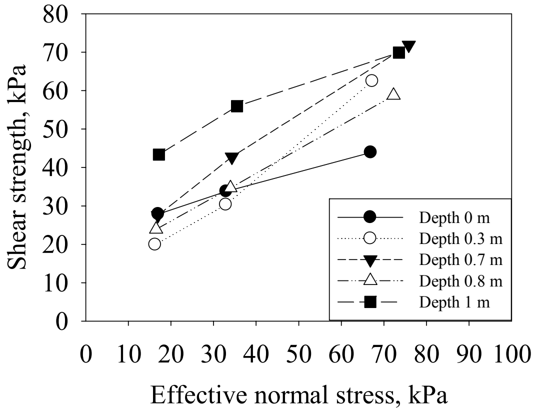

Figure 7. Effective failure envelops of saturated samples

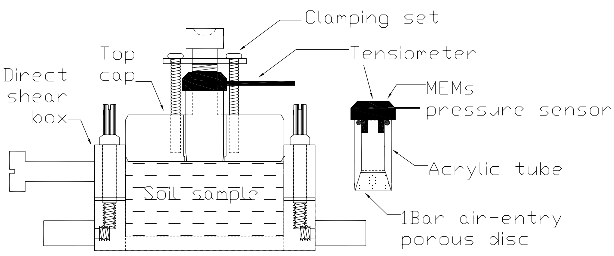

Figure 8. Suction-monitored direct shear box

3.2 *Unsaturated shear behavior*

The suction-monitored direct shear tests on the unsaturated specimens were conducted at a constant normal stress of 16 kPa. This value of normal stress corresponds to a soil thickness of about 0.8 to 1metre, which is the expected failure surface. The apparatus used was a conventional direct shear box, modified in order that the KU-tensiometer can be inserted through the top cap and monitor the soil suction during shear, as shown in Figure 8 (Jotisankasa and Mairaing, 2009). All shearing tests were done with samples maintained in the constant water content condition and at a shearing rate of 0.05 mm/min. Examples of test results are illustrated in Figure 9. The range of samples' suction during testing was chosen to reflect the actual variation in the field as shown in Figure 6.

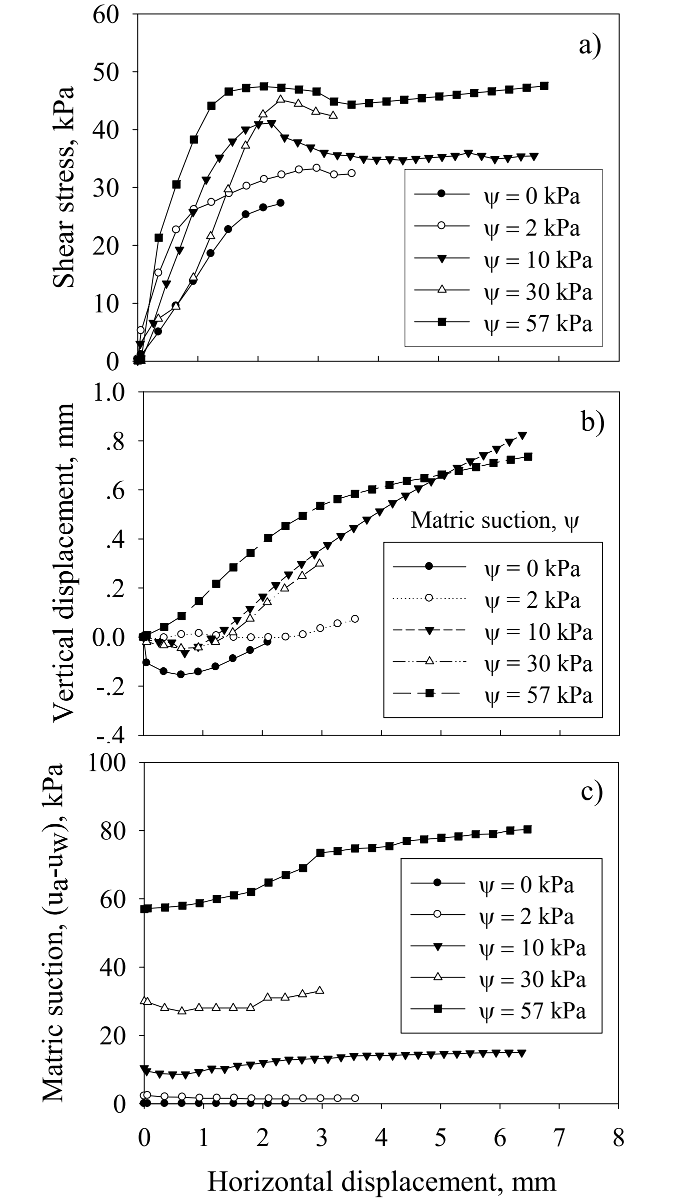

Figure 9 Shearing behaviour of the samples at depth of 0.7 m with different suctions in suction-monitored direct shear box tested at a normal stress of 16 kPa

The samples with higher suctions exhibit higher strength and tend to dilate more. During the initial stage of shearing, the soil volume and the suction slightly decreased. As the samples started to dilate and reaching the peak strength, the suction also appeared to increase accordingly. The sample with suction of zero was tested in a multistage manner and therefore no results are available for large horizontal displacement. Figure 10 shows the variation of shear strength and suction for the samples at depth of 0.7m. The fitting equation used for the unsaturated shear strength is that of Fredlund & Rahardjo (1993) as follows:

$$\tau = c' + (\sigma - u_a)\cdot \tan\phi' + (u_a - u_w)\cdot \tan\phi^b \qquad (1)$$

where c' = effective cohesion intercept, σ = normal total stress, u_a = pore air pressure (for the direct shear tests carried out at atmospheric pressure, u_a equals zero), u_w = pore water pressure, ϕ' is the effective angle of shearing resistance, and ϕ^b the angle of shearing resistance with respect to suction. Table 2 summarizes all the shear strength parameters. The variation between the shear strength and suction appears to be nonlinear. It is appreciated that various mathematical expressions have been proposed to fit this non-linear trend (e.g. Vanapalli et al., 1996), though none of them will be presented here for brevity of the paper.

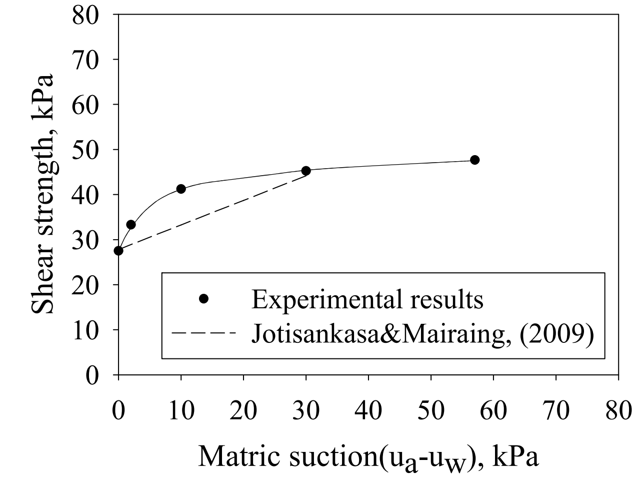

Figure 10 Shear strength versus suction of sample at depth of 0.7 m

Table 2. Shear strength parameters.

Depth (metre)	c' kPa	ϕ' deg.	ϕ^b deg.
Top soil	22.8	17.6	-
0.3	4.6	40.4	-
0.7	15.9	36.6	27.7
0.8	13.7	32.0	-
1.0	37.2	24.5	-

3.3 *Soil-water retention curves*

The volumetric water content-suction relationships have also been determined for all samples, as shown in Figure 11. Suction measurements were made using the miniature tensiometers and relative humidity sensor (Jotisankasas et al., 2007). The test procedure involved gradually wetting and drying the sample while measuring the suction, weight and dimensions at each stage. During the wetting path, once the soil suction fall below 1kPa, the specimen was submerged under water with a nominal vertical overburden stress of 1kPa for a period of at least 5 days. The values of water content at suction of 0.1kPa, shown in Figure 11, was in fact arbitrarily chosen to indicate the water content of soaked samples, θ_s, in the logarithmic suction plot. This soil water retention curve was also used to predict the failure envelope, using the simplified approach by Jotisankasa & Mairaing (2009) (Eq.2),

$$\tau = \left[c' + (\sigma_n - u_a)\tan\phi'\right] + \left[(u_a - u_w)\left(\frac{\theta_{33}}{\theta_s}\right)(\tan\phi')\right] \qquad (2)$$

where, θ_s = saturated volumetric water content, and θ_{33} is the volumetric water content at 33 kPa suction or at the nominal field capacity. As shown in Figure 10, Jotisankasa & Mairaing (2009) approach could predict the lower bound of the shear strength satisfactorily for suction less than about 30 kPa. As suggested by Jotisankasa & Mairaing (2009), this approach is suitable to be applied for approximation of unsaturated shear strength for very large areas where only available data are the field capacity of the soils and not the complete soil-water retention curves.

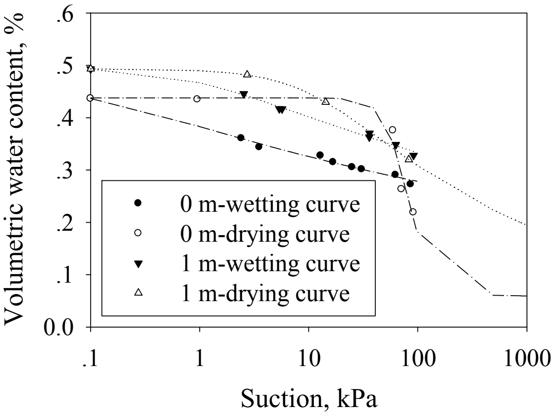

Figure 11 Soil-water retention curves

3.4 *Influence of wetting/drying cycles*

In order to clarify the expected degradable properties of the material, a series of shearing tests were carried out on samples taken at depth of 1metre which followed varying cycles of wetting/drying. For each cycle, the sample was soaked in water for a couple of days before oven-dried at 105°C for a day. This was believed to represent the extreme case of temperature and moisture content change that soil can undergo in the field.

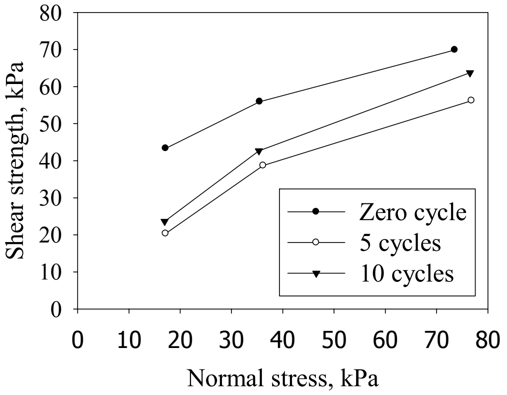

Figure 12 Influence of drying/wetting cycles on failure envelope

It is noted that the soil at 1m depth in the field (zero wet/dry cycle) behaves as structured soil exhibiting some effective cohesions. The effective cohesion of sample subjected to 5 cycles of wetting/drying was reduced to nearly zero by destructuration process (Fig. 12). With greater cycles, the failure envelope still remained relatively unchanged. The sample must have been degraded to the extent that nearly all the cementation structure has been destroyed after no more than 5 cycles.

4 IMPLICATIONS ON SLOPE STABILITY

In order to put all these observed behaviour in the context of slope stability, some simple infinite slope analyses were carried out which take into account the saturated/unsaturated shear strength. Equation (4) was used to calculate factor of safety, F, for both saturated and unsaturated case,

$$F = \frac{c' + (\gamma \cdot z \cos^2 \beta) \cdot \tan\phi' - u_w \cdot \tan\phi''}{\gamma \cdot z \sin\beta \cdot \cos\beta} \tag{4}$$

where, $\phi'' = \phi'$ if $u_w > 0$, and $\phi'' = \phi^b$ if $u_w \leq 0$. Hypothetical slopes with depth of failure, z = 0.5 and 1m, and slope gradient, β = 25, 45, and 60 degrees, were analyzed. These slopes represent the typical range of terrains in Uttaradit area that suffered from landslide. The soil properties used were those from Table 2 and Figure 12. The variation between the factor of safety and pore water pressure were then plotted in order to find out the triggering pore water pressure for different scenarios. It can be seen in Figure 12 that the pore water pressure which triggers the slope failure (at F =1) is of positive values for both slope gradients. As for the influence of drying/wetting cycles (Figure 13), for steep slope (β = 60deg.) with nearly zero suction, only five cycles (or less) of extreme wetting/drying cycles might cause some instability of the slope. The soil suction thus provides a crucial stabilizing effect for the degraded slope (destructured/cementation destroyed) with gradient more than about 60deg.

5 CONCLUSIONS

This paper describes some important soil behaviour affecting mechanism of rainfall-induced shallow slope failures in residual soils derived from sedimentary rock formation of Uttaradit province. The shear strength of the soil increases with depth, whereby the soil at 1m depth has the highest effective cohesion intercept due to its cementation. This cohesion however can be destroyed by a couple of cycles of extreme wetting/drying. Unsaturated shear strength has also been determined using the suction-monitored direct shear tests. The variation between the shear strength and suction appears to be nonlinear. The results from a simple infinite slope analysis suggest that the major slope destabilization mechanism is a combination of material degradation and pore water pressure increase.

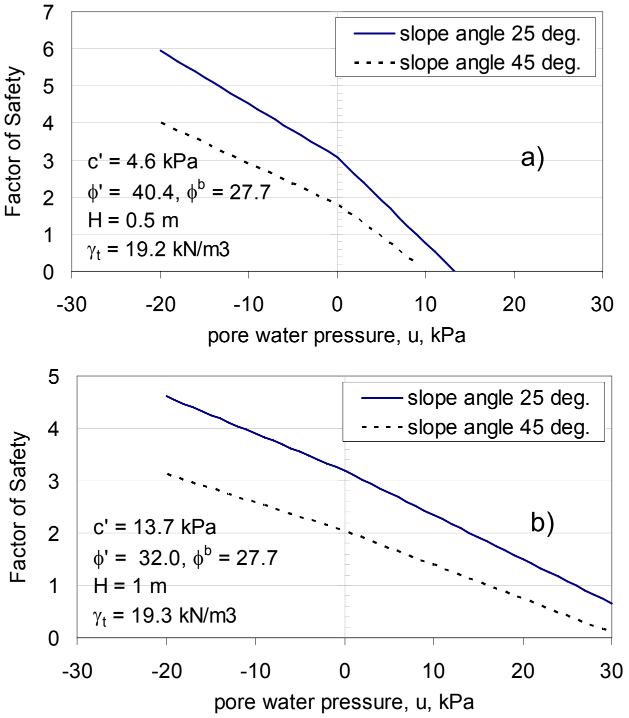

Figure 12 Variation between factor of safety and pore water pressure for different slopes (zero wetting/drying cycles)

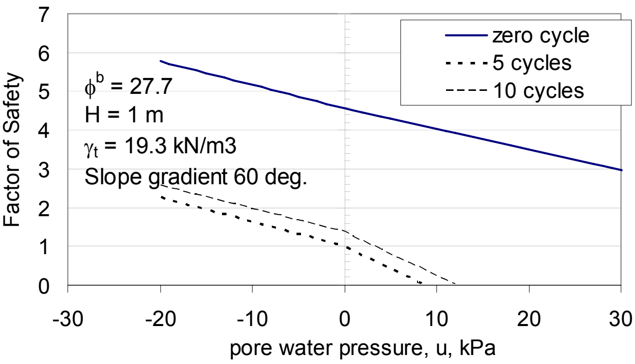

Figure 13 Variation between factor of safety and pore water pressure for slope with varying drying/wetting cycles

6 ACKNOWLEDGEMENTS

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7 REFERENCES

ADPC-Asian Disaster Preparedness Center (2006). *Rapid assessment, Flashflood and landslide disaster in the provinces of uttaradit and sukhothai, northern Thailand*, May 2006. (www.adpc.net/enewsjuly/Uttaradit_rapidassessment.pdf)

Alonso, E.E., and Pineda, J.A. (2007). Degradation of argillaceous rocks: a challenge for unsaturated geomechanics. *Proc. 3rd Asian Conference on Unsaturated Soils (Unsat-Asia 2007), Nanjing, China, 21-23 April 1007, 3-26*

Cho, S. E. and Lee, S. R. (2002). Evaluation of Surficial Stability for Homogeneous Slopes Considering Rainfall Characteristics. *Journal of Geotechnical and Geoenvironmental Engineering. ASCE. Vol. 128, No. 9, September 1, 756-763*

Collins, B. D, and Znidarcic, D. (2004). Stability analyses of rainfall-induced landslides. *Journal of Geotechnical and Geoenvironmental Engineering. ASCE. April 2004, Vol. 130, No. 4, 362–372*

Garga, V.K. and Blight, G.E. (1997). Permeability. *Mechanics of residual soils. (ed. G.E. Blight)*, A.A. Balkema, Rotterdam, 79-94

Gerscovich, D.M.S., Vargas, E.A., de Campos, T.M.P. (2006) On the evaluation of unsaturated flow in a natural slope in Rio de Janeiro, Brazil. *Engineering Geology, 88 (2006) 23–40, Elsevier B.V.*

Godt, J.W., Baum, R.L and Lu, N (2009). Landsliding in partially saturated materials. *Geophysical Research Letters, VOL. 36, 1-5, L02403, doi:10.1029/2008GL035996*

Hvorslev, M.J. (1951). Time-lag and soil permeability in groundwater observations. *Bulletin No.36, US Waterways Experiment Station. Vicksburg, Miss., USA*

Johnson, K.A., and Sitar, N. (1990) Hydrological conditions leading to debris-flow initiation. *Canadian Geotechnical Journal, 27: 789-801*

Jotisankasa, A., Porlila, W., Soralump, S., and 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, 21-23 April 1007, 475-480*

Jotisankasa, A. and Mairaing, W. (2009). Suction-monitored direct shear testing of residual soils from landslide-prone areas, *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*

Ng, C.W.W and Shi, Q. (2003). The stability of unsaturated soil slopes subjected to rainfall infiltration. *Numerical Analysis and Modelling in Geomechanics (John W. Bull) London : Spon Press, 101-128*

Sabatini, P.J., Bachus, R.C., Mayne, P.W., Schneider, J.A. & Zettler, T.E. (2002). Shales and degradable materials. *Evaluation of Soil and Rock Properties, Report no. FHWA-IF-02-034, GEOTECHNICAL ENGINEERING CIRCULAR NO. 5, 243*

Rahardjo, H., Ong, T.H., Rezaur, R.B., & Leong, E. C. (2007). Factors controlling instability of homogeneous soil slopes under rainfall. *Journal of Geotechnical and Geoenvironmental Engineering. ASCE. Vol. 133, No. 12, December 1, 1532-1543*

Vanapalli, S.K., Fredlund, D.G., Pufhal D.E. & Clifton, A. W. (1996). Model for the prediction of shear strength with respect to soil suction. *Can. Geot. J. 33, 379-392.*

Vaughan, P.R. (1985). Pore pressures due to infiltration into partly saturated slopes. *Proc. 1st International Conference on Geomechanics in Tropical Lateritic and Saprolitic soils. Brazil, Vol 2, 61-71*