**A simplified monitoring and warning system against shallow rainfall induced slope failures**

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A simplified monitoring and warning system against shallow rainfall induced slope failures

by

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Abstract

A system that involves the use of a low cost micro-electro-mechanical systems (MEMS) based sensor stick and soil water characteristic curve (SWCC) inferred from grain size distribution was developed to assess the factor of safety against rainfall induced shallow slope failure. The system allows water content at multiple depths to be monitored in the field. The corresponding suction and factor of safety against infinite slope failure, are derived based on the estimated SWCC. The new system circumvented the use of tedious and often costly field as well as laboratory procedures while providing a quantitative slope stability assessment on a real-time basis. The
Draft paper introduces the background of the new development and describes the application of the sensor stick system in a field test.

Keywords: unsaturated soil, slope failure, soil water characteristics curve, MEMS, real time monitoring

**Introduction**

Rainfall has been considered as one of the most frequent triggering factors to natural slope failures (De Vita and Reichenbach 1998). It is well recognized that field hydrological and geomechanical properties/conditions are the key elements controlling the stability of a slope under the influence of rainfall (Collins and Znidarcic 2004; Kitamura and Sako 2010; Rahardjo et al. 2010; and Springman et al. 2013). Migration of fines and development of flow channels may also occur over time (Johnson and Sitar 1990). These potential time dependent variations of soil structure make real-time monitoring that reflects the current field conditions more desirable.

Rainfall induced landslides often occur as relatively shallow failure surfaces orientated parallel to the slope surface in areas where a residual or colluvial soil profile has formed over a bedrock interface (Rahardjo et al. 1995). These characteristics allow the use of infinite slope assumption as shown in Figure 1 to
analyze the slope stability.

By treating a shallow landslide as an infinite slope failure, Collins and Znidarcic (2004) showed that a relationship among the critical depth \( d_{cr} \) of slope failure, slope angle, soil strength parameters and pore-water pressure head can be established as,

\[
d_{cr} = \frac{c' + \gamma_w h_c \tan \phi' - \gamma_w h_p \tan \phi'}{\gamma \cos^2 \beta (\tan \beta - \tan \phi')}
\]

where

- \( \beta \) = slope angle
- \( c' \) = drained cohesion of saturated soil
- \( \gamma \) = unit weight of soil
- \( \gamma_w \) = unit weight of water
- \( \phi' \) = drained friction angle of saturated soil
- \( h_p \) = positive pore-water pressure head
- \( h_c \) = capillarity (negative pore-water pressure) head
- \( \phi^b \) = friction angle with respect to matric suction \((= \gamma_w h_c)\)

Equation (1) considers the effects of seepage force induced by the infiltration of surface water. Alternatively, a factor of safety (FS) at any given depth, \( d \) can be assessed for a set of soil parameters associated with that depth as follows:
The critical depth thus is a special case where \( d \) corresponds to \( FS = 1 \). The soil parameters can be obtained from laboratory tests (i.e., \( c', \, \phi', \, \phi^b \) and \( \gamma \)) and field measurement/monitoring (i.e., \( \beta, \, h_c \) or \( h_p \)).

In unsaturated soil, \( h_p = 0 \), and when soil becomes saturated, \( h_c = 0 \). According to Equation (1), for slope angles less than friction angle \( \beta < \phi' \) or coarse grain soil, failure in the unsaturated soil layer is not possible. In the case of \( \beta > \phi' \) or fine grain soil, the slope can fail when soil is unsaturated with negative pore water pressure.

Assuming the infinite slope was subjected to a uniform rainfall and seepage was numerically simulated as a one-dimensional vertical infiltration, Collins and Znidarcic (2004) demonstrated that the coupling of Equation (1) and one-dimensional infiltration analysis was able to quantitatively predict the time and depth of a shallow slope failure. Figures 2 and 3 show the evolution of pressure head profiles (\( h_c \) or \( h_p \) versus depth) as infiltration progresses in a uniform rainfall event, in the case of \( \beta > \phi' \) and \( \beta < \phi' \), respectively. In these analyses, groundwater table was assumed to be at 4 m below surface, \( \gamma = 20 \text{ kN/m}^3, \, c' = 3 \text{ kPa}, \, \phi' = 30^\circ \) and \( \phi^b = 26^\circ \). It was
assumed that the soil hydraulic conductivity varied (generally increased) with degree of saturation. The slope angle \( \beta \) is marked in the respective figures. Slope failure occurs at depth \( d_{cr} \) where the pressure head profile touches the stability envelope. The stability envelope is a graphic representation of Equation (1).

As demonstrated in Figures 2 and 3, the profiles of \( h_c \) or \( h_p \) are not likely to be linear with depth under the influence of rainfall infiltration. Also, for a given slope geometry and soil conditions, the distribution of \( h_c \) or \( h_p \) has a direct link to the slope stability. Huang et al. (2012) reported the use of an array of fiber optic piezometers to monitor the \( h_p \) profile and used that as a basis for warning against a rainfall induced slope failure. This paper will concentrate on monitoring and warning against rainfall induced slope failures when \( \beta > \phi' \, (h_p = 0) \), based on \( h_c \) values inferred from measurements in unsaturated soils.

According to the extended Mohr-Coulomb criteria by Fredlund et al. (1978), the shear strength of an unsaturated soil, \( \tau \) can be defined as,

\[
\tau = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) \tan \phi^b
\]

(3)

where

\[
\sigma = \text{total stress in soil}
\]
\[ u_a = \text{pore air pressure} \]
\[ u_w = \text{pore water pressure} \]
\[ (\sigma - u_a) = \text{net normal stress} \]
\[ (u_a - u_w) = \text{matric suction} (= \gamma_w h_c) \]

It is apparent from Equations (1) to (3) that \((u_a - u_w)\) and \(\phi^b\) are important strength parameters in assessing the stability of a slope of unsaturated soils. Ideally, \((u_a - u_w)\) can be measured with tensiometers installed in the field. Judging from the fact that \((u_a - u_w)\) may have a non-linear distribution with depth as described in Figure 2, it may be preferable to install tensiometers at multiple depths. Tensiometers installed in the field require maintenance to assure saturation in the pore pressure sensing element. Alternatively, volumetric water content, \(\theta\) or degree of saturation, \(S\) at various depths can be measured with electrical sensors, and the corresponding \((u_a - u_w)\) inferred from soil water characteristic curve (SWCC). The value of \(\phi^b\) can be measured using laboratory shearing tests on unsaturated soil specimens obtained from the field. The determination of SWCC and \(\phi^b\) require special laboratory tests that are tedious, time consuming and costly. Rainfall induced shallow slope failures can occur often and can be of limited scale with limited consequences. It may be difficult to justify the expenses and efforts to execute rigorous field
monitoring and laboratory tests as above described.

The aim of this paper is to develop a practical method to monitor a slope of unsaturated soil and to provide warning against rainfall induced slope failure. The new scheme involves the use of a low cost MEMS (Micro-Electro-Mechanical Systems) sensor stick capable of monitoring the field water content and temperature at multiple depths as well as tilt angle on the ground surface. Shear strength of the unsaturated soil was inferred from a procedure described by Vanapalli et al. (1996) that involved the use of semi-empirical SWCCs, saturated shear strength parameters and water content measurements. A factor of safety was then assessed assuming the slope as infinite. This procedure circumvented the direct measurements of \( u_a - u_w \), SWCC or \( \varphi^b \). The slope suitable for this type of instrumentation usually has a slope angle \( \beta \) larger than the drained friction angle \( \varphi' \) of the soil. The stability of the slope is maintained by the existence of suction. The potential failure depth is shallow enough that the analysis can be simplified as an infinite slope failure. The following sections first describe the background of a simplified method for estimating the SWCCs using grain size distribution and shear strength of the unsaturated soil based on field water content measurements. Details of the MEMS sensor stick and a case of field application of the new system will then be presented.
Estimation of SWCCs

Figure 4 shows the names for various branches of the SWCCs according to Pham et al. (2005). It is well known that the SWCC is hysteretic, or the volumetric water content, \( \theta \) at a given matric suction, \( \psi \) \( \approx (u_a - u_w) \) for a wetting path is less than that for a drying path. In addition to the primary curves; the initial drying curve, the boundary wetting curve, and the boundary drying curve, there are numerous scanning curves inside the hysteresis loop (i.e., space bounded by the boundary wetting curve and the boundary drying curve). To apply to the rainfall induced slope failure analysis, the boundary wetting curve will be used to infer \( \psi \) form \( \theta \).

Feng and Fredlund (1999) proposed an empirical \( \theta-\psi \) scaling method for the estimation of SWCC. The initial drying curve is best-fit using the following equation:

\[
\theta(\psi) = \frac{\theta_s b + c \psi^d}{b + \psi^d}
\]  

(4)

where

\( \theta_s \) = water content at zero soil suction on the initial drying curve

b, c, and d = parameters from curve fitting the initial drying curve

The boundary drying curve is calculated by changing the curve fitting parameter that
controls water content at zero soil suction, $\theta_\alpha$, while keeping the other curve-fitting parameters the same, or

$$\theta(\psi) = \frac{\theta_\alpha b + c \psi^d}{b + \psi^d}$$

(5)

where

$$\theta_\alpha = \text{water content at zero soil suction on boundary drying curve and } \theta_\alpha = 0.9 \theta_s$$

For the boundary wetting curve, Pham et al. (2005) assumed that the $\theta$-$\psi$ correlation can be described in a similar format as:

$$\theta(\psi) = \frac{\theta_\alpha b_w + c_w \psi^{d_w}}{b_w + \psi^{d_w}}$$

(6)

$$b_w = \left[\frac{b_d}{R_{SL}}\right]^{\frac{1}{d_w}}$$

(7)

$$d_w = \frac{d_d}{R_{SL}}$$

(8)

$R_{SL}$ is the ratio of slopes in semilogarithmic coordinate system as shown in Figure 5, and can be calculated as:

$$R_{SL} = \frac{S_{LD}}{S_{LW}}$$

(9)

where

$$b_d = b \text{ from Equation } (4)$$

$$c_w = c \text{ from Equation } (4)$$

$$d_d = d \text{ from Equation } (4)$$
\[ SL_{LD}, SL_{LW} = \text{slope of boundary drying and wetting curve, respectively} \]

\[ D_{SL} = \text{distance between the boundary drying and wetting curves in semilogarithmic coordinate system as shown in Figure 5} \]

Pham et al. (2005) suggested a series of empirical \( R_{SL} \) and \( D_{SL} \) values for various types of soils.

Fredlund and Xing (1994) reported a correction factor, \( C \) to be applied to Equation (6) to force the SWCC through a \( \psi \) value of \( 10^6 \) kPa (i.e., the terminal suction) and corresponding \( \theta = 0 \) (see Figure 5), as follows:

\[ \theta(\psi) = \frac{\theta_u b_w + c_w \psi_{idw}}{b_w + \psi_{idw}} \cdot C \quad (10) \]

and

\[ C = 1 - \frac{\ln(1 + \frac{\psi}{\psi_{rw}})}{\ln(1 + \frac{10^6}{\psi_{rw}})} \quad (11) \]

where

\[ \psi_{rw} = \text{residual soil suction on the boundary wetting curve} \]

For the boundary wetting curve, Pham et al. (2005) suggested that the value of \( \psi_{rw} \) can be calculated as follows:

\[ \psi_{rw} = (2.7 b_w)^{1/d_w} \quad (12) \]
where

\[ b_w \text{ and } d_w = \text{ respective values from Equations (7) and (8)} \]

The above empirical method, albeit much simplified, still requires the initial drying SWCC as a basis to determine the various curve fitting parameters. To further simplify the procedure, a physicoempirical method reported by Arya and Paris (1981) was adopted to establish the SWCC based on grain size distribution and bulk density of the undisturbed soil sample taken from the field. Recognizing that the soil moisture characteristic is essentially a pore-size distribution curve, this method first converts a grain size distribution into a pore-size distribution. Then, the cumulative pore volumes corresponding to progressively increasing pore radii are divided by the sample bulk volume to give the volumetric water contents, and the pore radii are converted to equivalent soil water pressures using the equation of capillarity.

To apply this method, the grain size distribution curve is first divided into \( n \) particle-size fractions. It is assumed that within each size fraction \( i \), the soil particles have a uniform radius. The pore volume for each size fraction \( i \) can be calculated as:

\[ V_{p_i} = \left( M_i / \rho_p \right) e \quad \text{(13)} \]

where
\[ i = 1, 2, 3 \ldots n \]

\[ V_{vi} = \text{pore volume per unit sample mass associated with the solid particles in the} \]

\[ i \text{th particle-size fraction} \]

\[ M_i = \text{solid mass per unit sample mass in the} \ i \text{th particle-size fraction} \]

\[ \rho_p = \text{particle mass density} \]

\[ e = \text{void ratio} \]

\[ M_i \] are obtained from the plots of grain size distribution, and

\[ e = (\rho_p - \rho_b) / \rho_b \quad (14) \]

where

\[ \rho_b = \text{measured bulk dry density of the undisturbed sample} \]

The differences in cumulative percentages corresponding to successive particle sizes divided by 100 result in values of \( M_i \) and the sum of all \( M_i \) is unity. The pore volumes \( V_{vi} \) generated by each size fraction are progressively accumulated and considered filled with water. The volumetric water content \( \theta_{vi} \) is then computed as:

\[ \theta_{vi} = \sum_{j=1}^{i} V_{vj} / V_b \quad (15) \]

where

\[ \theta_{vi} = \text{volumetric water content represented by a pore volume for which the largest} \]
size pore corresponds to the upper limit of the \( i \)th particle-size fraction,

\[
V_b = \text{sample bulk volume per unit sample mass given by}
\]

\[
V_b = \sum_{i=1}^{n} M_i / \rho_b = 1 / \rho_b
\]  

The average volumetric water content, \( \theta_{\psi}^* \), represented by a pore volume for which the largest size pore corresponds to the midpoint of a given particle size fraction is approximated as:

\[
\theta_{\psi}^* = (\theta_{\psi_i} + \theta_{\psi_{i+1}}) / 2
\]

It is assumed that: (i) the solid volume in any given particle size fraction can be approximated as that of uniform-size spheres defined by the mean particle radius for the fraction, and (ii) the volume of the resulting pores can be approximated as that of uniform-size cylindrical capillary tubes whose radii are related to the mean particle radius for the fraction. With these assumptions, we can formulate the relationship between pore and particle radii as follows.

If the solid mass in the \( i \)th particle-size fraction is represented by \( n_i \) spherical particles with uniform radius, \( R_i \) and if the entire pore volume formed by the assembly of particles in that fraction is represented by a single cylindrical pore with a radius of \( r_i \) and length of \( h_i \), then the total solid volume in the assembly of the \( i \)th fraction, \( V_{\psi_i} \):

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\[ V_{p_i} = \frac{n_i \pi R_i^3}{3} = \frac{M_i}{\rho_b} \] (18)

and

\[ V_{v_i} = \pi r_i^2 h_i = (M_i/\rho_p)e \] (19)

Dividing Equation (19) by Equation (18) yields a correlation between \( r_i \) and \( h_i \) as follows:

\[ \frac{r_i^2}{R_i^3} = \frac{4n_i e}{3h_i} \] (20)

For a given assembly of uniform particles in the \( i \)th fraction, the pore length \( h_i \) may be approximated as the number of particles that lie along the pore path times the length contributed by each particle, or \( h_i \approx 2n_i R_i \). Recognizing that in natural soil, particle shapes, sizes, and orientations can all affect \( h_i \). It is likely that each particle contributes a length that is greater than the diameter of an equivalent sphere. Thus, the number of spherical particles with radius \( R_i \), required to track the \( h_i \) in a natural soil material will exceed \( n_i \). Arya and Paris (1981) proposed that:

\[ h_i = 2n_i^\alpha R_i \] (21)

where \( \alpha > 1 \) and determined empirically. Substituting \( h_i \) into Equation (20) gives:

\[ r_i = R_i \left[ 4e \frac{n_i^{(1-\alpha)}}{6} \right]^{1/2} \] (22)

The best estimate of \( \alpha \) was 1.38 according to Arya and Paris (1981).

Matric suction \( \psi_i \) is obtained from the equation of capillarity.
\[ \psi_i = 2T \cos \Theta / \rho_w g r_i \quad (23) \]

where

- \( T \) = water surface tension (at a temperature of 25°C, \( T = 72 \times 10^{-3} \, N/m \))
- \( \Theta \) = contact angle (assumed to be zero)
- \( \rho_w \) = density of water
- \( g \) = gravity

The combination of Equations (17) and (23) provides the estimated initial drying SWCC. The boundary wetting curve which will be used to estimate the matric suction and shear strength of unsaturated soil is then derived following Equations (4) through (12).

**Shear strength using SWCC and parameters of the saturated soil**

If \( \phi^b \) is a constant, Equation (3) implies that the additional shear strength due to soil suction increases linearly with suction. Citing many experimental results, Vanapalli et al. (1996) indicated that the relationship between the shear strength and suction was nonlinear when tests were performed over a wide range of suctions. As qualitatively described in Figure 6, a linear increase in shear strength is expected up to the air-entry value. Beyond which the increase in shear strength becomes nonlinear. When suction
exceeded residual conditions, the shear strength may increase, decrease or remain relatively constant in further desaturation. For sands and silts, the water content at residual suction can be low and may not transmit suction effectively. Hence, further increase of suction will not necessarily result in linear increase in shear strength. Soils that are resistant to desaturation, such as highly plastic clays can exhibit a linear strength behavior over a relatively large range of soil suctions (Rahardjo et al. 1995). For these reasons, Vanapalli et al. (1996) proposed the following equation to estimate shear strength based on SWCC.

\[
\tau = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) \left[ \tan \phi' \left( \frac{\theta - \theta_r}{\theta_s - \theta_r} \right) \right]
\] (24)

With Equation (24), the shear strength of an unsaturated soil can be estimated based on the key parameters from SWCC (i.e, \( \theta_s \) and \( \theta_r \), volumetric water content (or degree of saturation), matric suction and saturated soil strength parameters (\( c' \) and \( \phi' \)). For safety analysis against rainfall induced slope failure, the authors used the boundary wetting SWCC (i.e., \( \theta_s \) is replaced with \( \theta_u \)) to assess the shear strength as follows:

\[
\tau = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) \left[ \tan \phi' \left( \frac{\theta - \theta_r}{\theta_u - \theta_r} \right) \right]
\] (25)

**The MEMS based sensor stick**

Figure 7a shows the assembled sensor stick and its major components. All electronic
components were commercially available and purchased through internet. An open
source microcontroller unit (MCU) placed in the container box on top of the sensor
stick (Figure 7a) was used to control the sensors, data logging and transmission by
firmware. The MCU board had I²C (Inter-Integrated Circuit) and SPI (Serial
Peripheral Interface) digital bus interfaces and analogue input channels. All sensors
are MEMS based, communicated with the MCU via a digital bus, small in size, low
cost and consume very little power. Up to four soil moisture sensors, spaced at a
minimum of 200 mm, can be mounted on the sensor stick which is made of a 28 mm
diameter PVC pipe. Wires connecting the moisture sensors to the MCU were placed
inside the PVC pipe. The moisture sensors were housed in adapters made with a 3-D
printer, to facilitate direct contact between the moisture sensor and soil. The soil
moisture sensors that also measured temperature, served as the major input to assess
the stability of the slope. The tilt sensor (Figures 7b and 7c), with a full range of
±15° and a resolution of 0.009°, provided a redundancy and direct measurements of
the slope angular movement as a final check of the slope stability. Significant angular
movement is a sign of imminent slope failure and thus has less value in warning
against an upcoming slope failure. Each sensor stick served as a node and its main
function was to collect data and transmit the data to a gateway. There can be multiple
sensor sticks and thus multiple nodes installed at a given site. One of the nodes was
configured as the gateway to collect the data from all nodes and transmit them to the office. A wireless communication system (i.e., Xbee in Figures 7b and 7c) was used to transmit data from the individual sensor sticks (i.e., the nodes) to the gateway. A SIM (Subscriber Identity Module) card was installed in the gateway that allowed the collected data to be transmitted to host computer or cell phone via WCDMA (Wide Band Code Division Multiple Access)/GSM (Global System for Mobile Communications) (Figure 7c) or the 3G system typically used for cell phone data transmission. All components were compatible with the MCU by digital bus interface.

The moisture sensor was based on capacitance measurements and the fact that soil dielectric constant (and thus capacitance) had a positive but non-linear relationship with the soil volumetric water content. The correlation between dielectric constant and moisture content can be affected by soil conductivity and particle sizes. A sensor operating frequency between 5 and 500 MHz is desirable (Wobschall and Lakshmanan 2005) to minimize the effects of soil conductivity. The moisture sensors used in the sensor stick had an operating frequency of 16 MHz. As will be reported later, the moisture sensor was calibrated in the soil taken from the test site to minimize the effects of soil grain characteristics.
The Xbee coupled with the real-time clock (Figures 7b and 7c) allowed the MCU’s in different nodes to take readings at a synchronized and constant time interval (i.e., 10, 20, 30 minutes or longer as required). The time interval was set by the firmware. If multiple sensor sticks were used, the data transmitted from each sensor stick were separated by 10 seconds. The transmitted data were simultaneously recorded in the data logger (Figures 7b and 7c) as a backup. To reduce power consumption, the sensor stick system was put in sleep when not taking or transmitting data. When in full operation, the sensor stick consumed approximately 0.9 W of electric power. A 10 W solar panel coupled with a 40 Ampere-Hour (12 V DC output) battery was used to provide the power for each sensor stick or gateway. There was no power outage during the approximately 5 month long field monitoring to be reported later.

The field test

A test site was setup at a natural slope of Xihu service area on No. 3 national expressway in Miaoli County, Taiwan with its location shown in Figure 8. The monitored area depicted in Figure 9 had a slope (i.e., angle of $\beta$ in Figure 1) of 38 to 42 degrees. The current slope surface was a result of earlier landslides. Probing of the slope surface with an electric drill showed 0.7 to 0.8 m thick soil underlain by a young and poorly cemented sandstone. The surficial soil consisted of sand and silt mixture
originated from the weathering of sandstone. The ground conditions at the test site fit well with the infinite slope simplification described earlier. A sensor stick, 950 mm long (from below the container box to tip of the sensor stick) with three moisture sensors, similar to that shown in Figure 7a was installed as a node. Upon installation, the moisture sensors were located at 100, 500 and 700 mm below the slope surface. A gateway with its electronic components similar to those of Figure 7c was installed for data transmission. No moisture sensors were included in the gateway. Locations of the sensor stick and gateway along with the solar panels are shown in Figure 9.

Procedure used for the installation of the sensor stick is described as follows:

- Drill a nearly vertical borehole slightly larger than 28 mm diameter to allow a tight fit between the borehole and the sensor stick. An electric drill was used for drilling the borehole.

- Insert the sensor stick to the borehole so that the container box stays above the slope surface by approximately 200 mm. Thus, the tip of the sensor stick was at 750 mm below slope surface.

- Fill the gap between the borehole and the sensor stick with the soil cutting from the borehole. Place bentonite pellets on the slope surface surrounding the borehole. This procedure ensures good contact between the moisture sensor and
the surrounding soil and prevents surface water from entering the borehole through the gap between the borehole and the sensor stick.

- Excavate a hole at approximately 2 m away from the sensor stick to a depth of 250 mm. Drive a 200 mm long and 74 mm outside diameter section of thin wall sampling tube into the soil from the bottom of the excavation. A rubber mallet was used to drive the sampling tube into soil. A bag sample of approximately 5 kg was taken from the bottom of the excavation using a shovel. The tube sample was used to determine the total unit weight, $\gamma$, bulk dry density, $\rho_b$ and void ratio, $e$ of the natural soil. The bag sample was used for specific gravity, $G_s$ or soil particle mass density, $\rho_p$ ($=\rho_w G_s$), triaxial tests, grain size distribution tests and calibration of the moisture sensor.

The sensor stick installed as the gateway had no moisture sensors but contain all the components shown in Figure 7c. The PVC pipe below the container box in this case was to fix the position of the gateway.

Key parameters obtained from the tube and bag samples are shown in Table 1. The drained cohesion, $c'$ and friction angle $\phi'$ was determined based on undrained triaxial tests on reconstituted specimens made from the bag sample. The triaxial
specimen was dry pluviated in five layers and tamped to a dry density of 1.47 g/cm³. Figure 10 shows the grain size distribution curves from two tests on the bag sample. The moisture sensor was calibrated inside of a soil sample taken from the test site. A 500 cc beaker was used to hold the soil with a moisture sensor stick inserted to the center of the beaker. The beaker was first filled with saturated soil and set on top of an electronic scale. The beaker and scale were placed inside of a thermo-moisture controlled chamber. The chamber was set at the relative humidity of 20% and temperature of 45°C. The scale kept track of the change of total soil weight as the soil sample slowly changed from wet to dry while keeping track of the moisture sensor readings. For dry to wet calibration, the soil was mixed to a known degree of saturation at a dry density similar to that in the wet to dry calibration. The moisture sensor reading was taken when it became stable. The process was repeated a few times with different degrees of saturation. In all the calibration tests, the soil sample was tamped to a bulk dry density, ρb, similar to that shown in Table 1. Because of the difference in calibration procedure, there was some scattering in test data and difference in the number in data points. Results of the calibration are included in Figure 11. A nonlinear equation fitted to the data points using a least square method, marked in Figure 11, was used to infer degree of saturation from moisture sensor
readings. This equation is repeated as follows:

\[
S = 0.00213(x - 390)^{1.472}
\]  

(26)

where

\[S = \text{degree of saturation in percentage}\]

\[x = \text{digital reading from the moisture sensor}\]

The sensors were installed in the field in December, 2016. No significant rainfall was recorded until April of 2017. Figure 12 shows a complete set of readings from the sensor stick from April 11 to April 16, 2017. The inclination shown in Figure 12a was in reference to gravity, taken above ground. The inclination increased by not more than 0.2 degree during that period. This variation can be due to the fact that the sensor box was supported by PVC pipe which is flexible and can distort by the fluctuation of temperature. In any case, as stated earlier the inclination sensor was used mainly as a redundancy and a final check of slope failure. A firm sensor stick made with more temperature stable material should minimize the problem. April is the time when spring turns into summer and temperature starts to increase significantly. This is reflected in the temperature records at 100 mm below ground as shown in Figure 12b. The temperature remained low in the rainy days but fluctuated significantly between day and night after the rain stopped. The temperatures at 500 and 700 mm depth
remained rather stable apparently due to the insulation effects of the ground.

**Derivation of SWCCs**

The initial drying SWCC was estimated following the procedure reported by Arya and Paris (1981) using the results of grain size distribution Test 1 shown in Figure 10 and soil properties summarized in Table 1. The grain size distribution curve was divided into 19 size fractions, numbered from left to right of the grain size distribution curve.

Each size fraction was bounded by the data points marked on the grain size distribution curve. Following Equations (13) to (23) resulted in 17 pairs of data points \((\theta^*_v, \psi_i)\) as shown in Figure 13. A spread sheet showing the computation of \((\theta^*_v, \psi_i)\) is included in the Appendix.

Point 2 to 10 (a total of 9 points) were used to curve fit Equation (4) for the initial drying SWCC, using the least square method. The curve fit had a coefficient of correlation, \(R^2\) of 0.99. The data points were selected because they cover the range of interest in matric suction and the quality of curve fitting became poor when data points of higher suction values were involved. The derived coefficients of Equation (4) according to the curve fitting, and other derived coefficients for the boundary drying and boundary wetting SWCCs, following Equations (5) and (6), respectively are

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shown in Table 2. Followed the suggestion by Feng and Fredlund (1999), $\theta_u = 0.9 \theta_s$.

$R_{SL} = 1.7$ and $D_{SL} = 0.22$ considering the values suggested by Pham et al. (2005) for silty sand soil and $\psi_{r,w} = 19.67$ kPa, to be used in Equations (10) and (11), was calculated according to Equation (12). Figure 14 shows all the derived SWCCs based on the grain size distribution.

**Determination of the stability envelope and factor of safety**

The field monitoring provides the variation of degree of saturation $S$ (or volumetric water content $\theta$) readings at the respective depth of the moisture sensors. The measured $S$ or $\theta$ are used to infer the $(u_a - u_w)$ based on the estimated boundary wetting SWCC as described above. With the available data from laboratory tests and field monitoring, it is possible to assess the stability of the slope on the real-time basis. For the unsaturated slope ($h_p = 0$), we can establish a stability envelope similar to that shown in Figure 2. The stability envelope is the correlation between $d_{cr}$ and soil suction $(u_a - u_w)$ (or $h_c$) as defined by Equation (1), for the given slope and soil conditions (i.e., $\beta$, $\gamma$, $\phi^b$ and $\phi'$). For a given depth of measurement $d$, the closer the inferred $(u_a - u_w)$ is to the stability envelope the closer the slope is to failure. Figure 15 shows two versions of the stability envelopes. The linear stability envelope is determined based on an assumed $\phi^b$ that equals to $\phi'$ of 39 degrees according to
Table 1 \( (c' = 0) \). The nonlinear stability envelope is defined using the shear strength according to Vanapalli et al. (1996) as follows:

\[
d_{cr} = \frac{c' + (u_a - u_w) \tan\phi' \left( \frac{\theta - \theta_r}{\theta_u - \theta_r} \right)}{\gamma \cos^2 \beta \cdot \left( \tan \beta - \tan \phi' \right)}
\]  

\( \theta_r \) was taken as 0.044 according to the derived SWCC. As shown in Figure 15, the critical depth \( d_{cr} \) can decrease as \( (u_a - u_w) \) continues to increase. This is due to the fact the suction related shear strength can decrease with suction after reaching a peak value, when defined according to Vanapalli et al. (1996). For the case of field monitoring reported herein, significant difference between the linear and nonlinear stability envelopes occurred only in depth much greater than that of the soil-rock interface as marked in Figure 15 and thus the selection of stability envelope should have no effect in the assessment of factor of safety.

A more direct application of the field data would be to determine the factor of safety using Equation (2) but use the shear strength defined by Vanapalli et al. (1996) as follows:

\[
FS = \frac{c' + (u_a - u_w) \tan\phi' \left( \frac{\theta - \theta_r}{\theta_u - \theta_r} \right) + d \cdot \gamma \cos^2 \beta \cdot \tan \phi'}{d \cdot \gamma \cos^2 \beta \cdot \tan \beta}
\]

where

\[ d = \text{depth of the moisture sensor} \]
\( \theta \) is converted from degree of saturation \( S \) as follows:

\[
\theta = \frac{S_e}{100(1+e)} \cdot \frac{\theta_u}{\theta_s} \tag{29}
\]

The value of void ratio, \( e \) is given in Table 1. Multiplication of \( \theta_u/\theta_s (= 0.9) \) in Equation (29) forces the volumetric water content to conform with the boundary wetting SWCC. The matric suction, \( (u_a - u_w) (= \psi) \) is calculated from \( \theta \) following Equation (10). With all the other parameters known from laboratory tests and the estimated SWCC, the FS for the given depth of moisture sensor, \( d \) can then be assessed using Equation (28). Theoretically, the total unit weight \( \gamma \) varies as the degree of saturation changes. For simplicity, a constant \( \gamma \) considering 20% of gravity water content was used in all the computations. This simplification is not expected to induce significant error. Figure 16 shows the evolution of FS during the period from April 11 to April 16, 2017, corresponding to the degree of saturation readings shown in Figure 12. Consistent and significant drop in FS at all moisture sensor depths occurred only after the heavy rainfall event on late April 12. The FS started to increase after that rainfall except for the moisture sensor at depth of 700 mm that was placed close to the soil-rock interface. Apparently there were favorable seepage paths in the soil layer that facilitated the accumulation of moisture at the soil-rock interface and the degree of saturation reaching 100% as indicated in Figure 12d. This high degree of saturation was maintained until the middle of April 15. The fact that the
slope did not fail while the apparent FS became lower than 1 was because \( c' = 0 \) was used in Equation (28) in the computation of FS but in reality, the \( c' \) of the sandstone bedrock at test site should be much higher than zero. In any case the FS at all three sensor depths became high towards the end of April 15 when the rain stopped and the relatively high temperature as shown in Figure 12b evaporated the moisture.

**Concluding remarks**

A new sensor stick and a simplified data interpretation framework have been developed. The sensor stick allows water content at multiple depths be monitored. The data interpretation is based on SWCC inferred from grain size distribution and in situ soil bulk density. The system is low cost and easy to apply. A factor of safety on a real-time basis can be assessed based on the mechanics of unsaturated soil and assuming infinite slope failure. The technique is ideal for monitoring slopes that does not involve serious risks. For more elaborate slope monitoring, the required SWCC as well as the shear strength of unsaturated soil should be obtained from rigorous laboratory tests.

In addition to providing the factor of safety, water content measurements from the field trial also indicated that the natural soil deposit did not behave as a uniform soil
mass. There could be drainage paths that allowed moisture to quickly pass through the surficial soil layer and accumulate on the surface of the bedrock. This phenomenon demonstrated the advantage of monitoring water content at various depths. It also showed the value of real-time field monitoring over numerical simulations based on assumed soil conditions.

Acknowledgements

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**Figure Captions**

Figure 1. Infinite slope failure

Figure 2. Infiltration results for the case of $\beta > \varphi'$ or fine grain soil with superimposed stability envelope (after Collins and Znidarcic 2004)

Figure 3. Infiltration results for the case of $\beta < \varphi'$ or coarse grain soil with superimposed stability envelope (after Collins and Znidarcic 2004)

Figure 4. Various branches of hysteretic soil-water characteristic curves

Figure 5. Illustration of the slope and distance between the two boundary SWCCs

Figure 6. Increase of shear strength with matric suction

Figure 7. The MEMS based sensor stick (a) the assembled system (b) the node and (c) the gateway

Figure 8. The Xihu service area.

Figure 9. Layout of the test site

Figure 10. Grain size distribution of soil samples taken from the test site

Figure 11. Calibration results of the moisture sensor.

Figure 12. Readings from April 11 to April 16, 2017: (a) Rainfall record and inclination readings; (b) Moisture and temperature sensor at 100 mm depth; (c) Moisture and temperature sensor at 500 mm depth; (d) Moisture temperature sensor at 700 mm depth.

Figure 13. Data points of $(\theta^*_\psi, \psi)$ and fitted curve of initial drying SWCC

Figure 14. The derived SWCCs

Figure 15. Comparison of stability envelopes

Figure 16. Evolution of FS at the three moisture sensor depths
Table 1 Key parameters obtained from field soil samples.

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<th>$\phi'$, degree</th>
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*: based on test using the tube sample

$: based on undrained triaxial tests on reconstituted specimens
Table 2 Derived coefficients for the SWCC equations

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(1) Gateway  
(2) Sensor node  
(3) Abandoned sensor  
(4) Solar panels  

Figure 9. Layout of the test site
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Figure 12 Readings from April 11 to April 16, 2017: (a) Rainfall record and inclination readings; (b) Moisture and temperature sensor at 100 mm depth; (c) Moisture and temperature sensor at 500 mm depth; (d) Moisture temperature sensor at 700 mm depth.
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Figure 14. The derived SWCCs
Figure 15. Comparison of stability envelopes
Figure 16 Evolution of FS at the three moisture sensor depths
Appendix: Computation of \( (\theta^*_v, \psi_i) \) based on grain size distribution and in situ bulk density

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