Rainfall-induced landslides in the residual soil of andesitic terrain, western Japan

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Cite this article as: *Dahal R.K., Hasegawa S., Yamanaka M., Bhandary N.P., Yatabe R., 2011, Rainfall-induced landslides in the residual soil of andesitic terrain, western Japan, Journal of Nepal Geological Society, 42: 127–142.*

Abstract

Rainfall triggered landslides are frequent problems in the residual soil of andesitic terrain in west Japan. Characteristics of residual soils over bronzite andesite, procedure of in situ permeability measurement, matric suction and soil moisture content change and stability analyses considering unsaturated-saturated soils as integral system are presented in this paper. The paper highlights two landslides of small andesitic hillock of western Japan and describes modelling of rainwater seepage, slope stability analysis and contributing parameters for landsliding in andesitic terrain. For both landslides, results of geomorphological and geotechnical analyses were used as a direct input to the numerical modelling. For transient conditions, a finite element analysis was used to model the fluctuations in pore water pressure during the rainfall, with the computed hourly rainfall rate as the surface boundary condition. This was then followed by the slope stability analysis using the temporal pore water pressure distributions derived from the seepage analysis. Obtained trend for the factor of safety was indicates that the most critical time step for failure was a few hours following the antecedent moisture content of previously day peak rainfall. Time of failure estimated by modelling has shown good match with time declared by eyewitnesses.

1 Introduction

Landslide occurrence is generally facilitated by combined effect of intrinsic and extrinsic parameters. A trigger is an extrinsic event such as an intense rainfall event, an earthquake, a volcanic eruption, a storm wave, or rapid stream erosion that causes a near-immediate response in the form of a landslide by rapidly increasing the stresses or strains and reducing the strength of the slope-forming materials (Wieczorek, 1996). Similarly, intrinsic parameters also have vital roles in the landslide occurrence and they include bedrock geology, geomorphology, soil depth, soil type, slope gradient, slope aspect, slope curvature, landuse, elevation, engineering properties of the slope material, land use pattern, drainage patterns and so on. When extrinsic parameter rainfall is concerned, the type of landslide largely depends upon intensity and duration of the rainfall events (Campbell 1975; Caine 1980; Brand et al. 1984; Wieczorek 1987; Wilson and Wieczorek, 1995; Crozier 1999; Corominas 2000, Guzzetti et al. 2004; Aleotti 2004; Giannecchini 2006; Dahal and Hasegawa 2008). The existing rainfall-induced landslide studies illustrate the general and numerical relationships between landslides and rainfall in various countries. Studies have showed that rainfall can induce both deep- and shallow-seated landslides, but deep-seated landslides are triggered by rainfall over extended periods with a moderate intensity, while shallow landslides such as soil slips and debris flow are usually triggered by short duration and intense precipitations. Slope material permeability has close link with landslide occurrence (Campbell 1975; Crozier 1999; Guzzetti et al. 2004; Dahal and Hasegawa 2008). Granular slope materials tend to react to short duration extreme rainfall events whereas clayey materials are mainly sensitive to a long duration and rather low rainfall intensity. Studies show that critical quantity of rainfall necessary to produce slope failure depends on vegetation, hydrology and morphology of the slope (Campbell 1975; Brand et al. 1984; Wieczorek 1987; Wilson and Wieczorek, 1995; Corominas 2000,).

Varieties of approaches have already been described in literatures and they showed the relationship between rainfall and slope failures in terms of rainfall thresholds, hydrological models, and, coupled with hydrological and stability models (Reid et al. 1988; Dhakal and Sidle 2004; Borga et al. 2002, Rezaur et al. 2002; Rahardjo et al. 2002; Tsaparas et al. 2002; Kim et al. 2004; Rahardjo et al. 2005; Dapporto et al. 2005; Tofani et al. 2006; Dahal et al. 2008a). Rainfall and liquefaction of slope materials have also been examined by number of researchers (Anderson and Sitar 1995; Montgomery et al. 1997; Sassa 1998; Dai et al. 1999; Lan et al. 2003; Collins and Znidarcic 2004; Cai and Ugai 2004). Rainfall-induced landslides in coarse grained soils are normally caused by increased pore pressures and seepage forces during periods of intense rainfall whereas in fine grained soil having low infiltration rates do not lead to the development of positive pore pressure and failure occurs due to the decrease in shear strength of soils caused by the loss of matric suction. Many studies also suggest that shallow failures are usually associated with the increased positive pore water pressure whilst loss of negative pore water pressure or matric suction is mainly responsible for deep-seated failure (Anderson and Sitar 1995; Lan et al. 2003; Collins and Znidarcic 2004; Cai and Ugai 2004, Dahal et al. 2009). The mobilization of debris flows from slides has also been studied (Ellen and Fleming 1987; Iverson 1997; Iverson et al. 1997; Olivares and Picarelli 2003). These authors showed that liquefaction of slope materials plays a key role in debris-flow initiation.

Understanding of the distribution of positive as well as negative pore water pressure is a fundamental of rainfall triggered landslide study (Rahardjo et al. 1995). Usually, hydrological models have been used to simulate saturated and unsaturated flow in slopes. The hydrological models can be linked to slope stability models to obtain accurate simulations of the conceivable stability conditions of slope during rainfall. TOPOMODEL (Beven and Kirkby 1979), HYSWASOR (Van Genuchten 1980), Combined Hydrology and, Slope Stability (CHASM) model (Anderson and Lloyd 1991; Collison and Anderson 1996) and GWFLUCT (Terlien 1996) are some examples of such hydrological models. The model described by Iverson (2000) illustrates the infiltration process of rainfall and landslide processes. He showed that the use of pressure head response, topographic data, rainfall intensity and duration, along with infinite-slope failure criteria, helps to predict the timing, depth, and acceleration of rainfall-triggered landslides. The Antecedent Soil Water Status Model

(ASWSM) described by Crozier (1999) and Glade et al. (2000) accounts for the draining of early rainfall (total flow out of initial rainfall) and accumulation of late rainfall (infiltration of rain later stage). They provide an equation for estimating the probability of landslide occurrence as a function of daily intensity and previous water accumulation. The GeoStudio (2005) software package is another example of a coupled (SEEP/W and SLOPE/W) hydrological-slope stability modelling software. The coupled SEEP/W–SLOPE/W analyses (Krahn 2004a; Krahn 2004b) have been applied successively to evaluate the dynamic conditions of stability of embankments and slopes (Rinaldi and Casagli 1999, Crosta and Dal Negro 2003; Rinaldi et al. 2004; Collins and Znidarcic 2004; Dapporto et al. 2005). The SEEP/W of GeoStudio software package analyses the seepage problems and adopts an implicit numerical solution to solve Darcy's equation for saturated and unsaturated flow conditions, describing pore-water pressure and movement patterns within porous materials over space and time. The results obtained from seepage modelling can be directly linked to SLOPE/W. Whereas, SLOPE/W can be used as a limit equilibrium slope stability model.

The occurrence of rainfall-induced landslides in both residual (Rahardjo et al. 1995; Rahardjo et al., 2002) and colluvial soil (Rinaldi et al. 2004; Dapporto et al. 2005) slopes are common in many tropical and subtropical regions with abundant rainfalls. Rainfall-induced slope failure involves infiltration through the unsaturated zone above the ground water table. Therefore, unsaturated and saturated soils conditions in a slope have to be considered in the stability analyses. Wide ranges of contributing parameters are involved in rainfall-induced landsliding process. Slope geomorphology, micro climate, bedrock structures, bedrock hydrology, saturated and unsaturated strength of the slope materials, clay mineralogy of slope materials, transient pore water pressure changes, and abrupt loss of strength are the main factors found to be responsible for rainfall-triggered landsliding processes (Rahardjo et al., 2002; Rinaldi et al. 2004; Dapporto et al. 2005; Dahal et al. 2009) . In this context, this paper describes a rainfall triggered landsliding process in residual soil of andesitic terrain in western Japan.

2 Research objectives

The main objective of this study is to investigate the hydro-mechanical process responsible for rainfall-triggered landsliding in residual soils of andesitic terrain of western Japan. For this purpose, a rainfall-triggered landslide event which occurred during an extreme typhoon rainfall of August 2004, in the north eastern hills of Shikoku Island of Japan was selected for the study. Although hundreds of landslides were triggered in both andesitic and granitic terrains (Dahal et al. 2006; Dahal et al. 2008a) in the north-eastern part of Shikoku Island during various typhoons of 2004, the most affected area of andesitic terrain was chosen for the study. Objectives of this research are explicitly listed as follows:

- To explore the roles of geology and geomorphology setting on the landslide
- To understand the matric suction change in residual soil of andesitic terrain during different intensity of rainfall
- To discover the spatial variation of seepage in relation to soil permeability and other physical properties
- To analyze the stability of slope with respect to high intensity of rainfall and change in pore water pressure
- To outline the contributing parameters responding for landslide in the andesitic terrain.

3 The study area

3.1.1 Location and geology

The study area is located in the Mineyama hillock of Takamatsu City of Japan (Fig. 1). Takamatsu, the capital city of Kagawa Prefecture, Japan, is a harbour city on the northern shore of Shikoku Island, Japan. It is the regional administration centre of Shikoku.

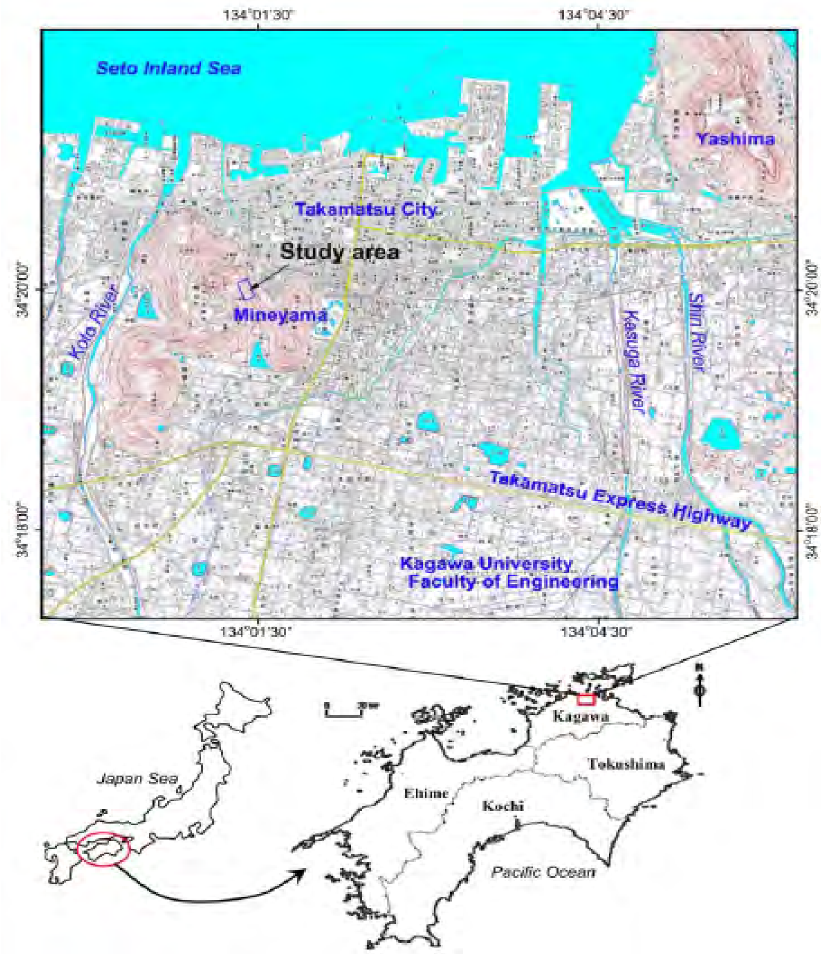

Fig. 1: Location map of study area

Takamatsu City is situated in the Takamatsu plain. The plain is mainly composed of the alluvial fan of the Koto river and the subordinate flood plains of the Shin and Kasuga rivers. The downtown Takamatsu has a well-forested Mineyama hillock in west. The maximum altitude of the Mineyama hillock is 208 m above the mean sea level. The area mainly consists of evergreen broad leaf and deciduous broad leaf types of forest. Japanese red pine (*Pinus deniflora*), camphor (*Cinanamomum camphora*), Japanese oak (*Quercus serrata* and *Quercus variabilis*) are main tree species in the Mineyama. Baby rosa (*Rosa multiflora*) and China root (*Smilax china*) are main shrubs in the forest. The toe of the Mineyama hill, is densely populated and formed a part of downtown Takamatsu. Because of the panoramic view of the Seto Inland Sea and Takamatsu City to the north and east, Mineyama hill is a popular hiking spot.

Geologically, Shikoku Island is roughly divided into three geological zones. They are Ryoke, Sambagawa-Chichibu and Shimanto belts from north to south, respectively. Topographically, the Ryoke Belt in Shikoku is divided into three zones: Seto Inland Sea, Recent fan and Hills having a maximum altitude of approximately 1000 m. This belt consists of Late Cretaceous granitic rocks, Late Cretaceous sedimentary rocks (Izumi Group) and Miocene volcanic rocks

(Sanuki Group). Being the locality of Izumi Group, Mineyama hill also consist of Miocene andesite and rhyolite (Fig. 2). Kuno (1947) called andesite as bronzite-andesite because of the presence of higher percentage of bronzite mineral which locally also known as Sanukite. The toe of the Mineyama hill mainly consists of colluvium soil having weathered angular pebble of bronzite-andesite. On the upper hill slope, Mineyama has 1 to 3 m thick residual soil over bronzite-andesite.

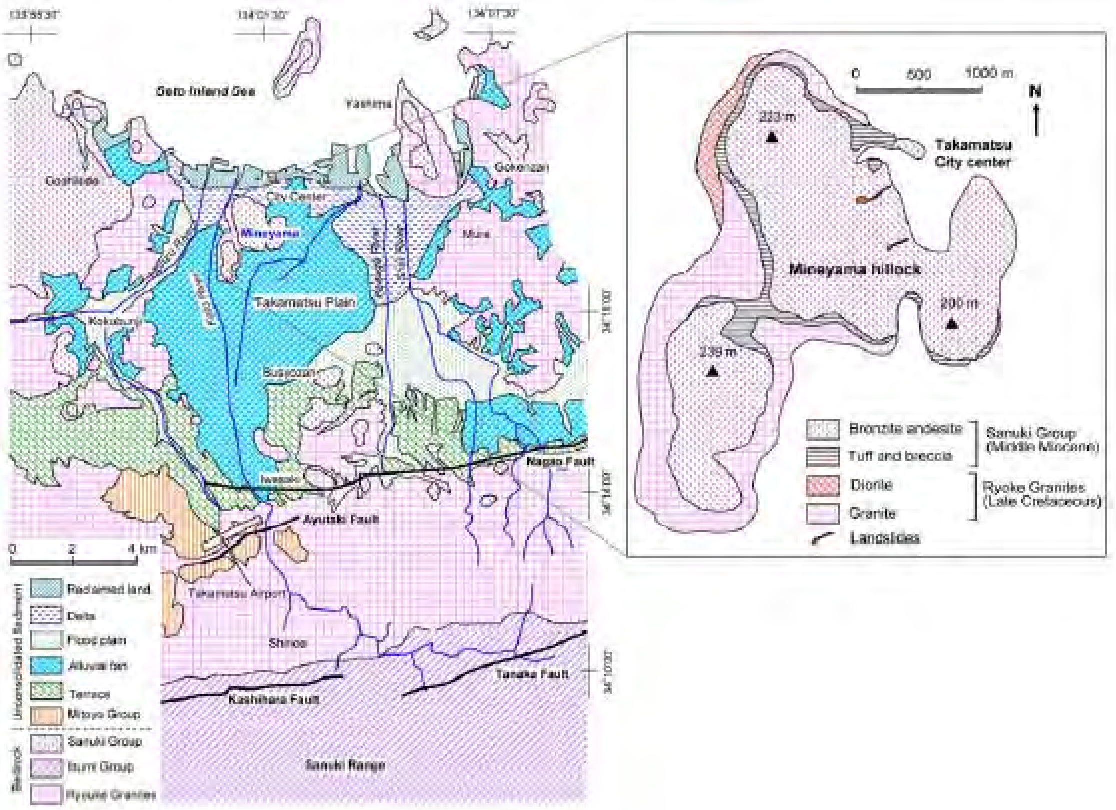

Fig. 2: Geological map of Takamatsu City and Mineyama hillock

3.1.2 Rainfall and landsliding

Shikoku Island of Japan usually gets extensive typhoon rainfalls in southern part in comparison to northern part. As a result, rainfall related slope failure phenomena are very common in central and southern Shikoku. However, northeast Shikoku (Kagawa Prefecture) is also suffering from some events of slope failures. In the year 2004, all together 10 typhoons hit Japanese archipelago and out of which nine typhoons affected the Shikoku Island. Storm and flood damages in 2004 resulted in a total of 227 killed and missing in whole Japan, which is the highest number since 1984 (MLIT, 2004). In 2004, Ehime, Kochi and Kagawa prefectures were mostly affected by Typhoon 23 and typhoon 21 whereas typhoons 4, 6, 10, 11, 15, 16 and 18 caused extensive damage and loss of lives in Kochi, Tokushima and Ehime prefectures (Hiura et al. 2005; Dahal et al., 2008b).

In 2004, Kagawa Prefecture badly suffered loss of lives and property due to landsliding triggered by typhoon rainfall. Better to say, Kagawa was the most affected prefecture. It was noticed that hourly rainfalls exceeding 50 mm and total 24-hour rainfalls over 200 mm were the main cause of slope failures at different locations in Kagawa. The hardest-hit areas were in granitic terrain of eastern Kagawa, granitic, rhyolitic and andesitic terrain around Takamatsu City (central Kagawa) and sedimentary terrain of western Kagawa (Dahal et al. 2006, 2008b).

Two landslides occurred in eastern face of Mineyama hillock during heavy rainfall of 19th and 20th October of 2004 (Typhoon 0423). A total of 11 landslides occurred in Mineyama after extreme rainfall owing to typhoon 0416 and 0423. In this study, landslide 'A' and landslide ‘B’ were selected for detailed study (Fig. 3). Landslide ‘A’ is located at topographic hollow of uphill section of Mineyama hiking road and front side of Ritsurin lodge. According to an eyewitness account, landslide ‘A’ occurred at 14:00 hours of 20th October, 2004. The debris from the failure accumulated on parking area of a resort and damaged some vehicles. Landslide ‘B’ was located on same east facing slope but situated approximately 250 m south of Landslide ‘A’ and it is also situated at topographic hollow of uphill section of Mineyama hiking road. The exact failure time of Landslide ‘B’ can not be confirmed but it is assumed that it also failed around the same time as of Landslide ‘A’. In both the landslides, failure was first initiated as debris slide and then started to flow down as debris flow. Hereafter landslides ‘A’ and ‘B’ are addressed as Mineyama landslides.

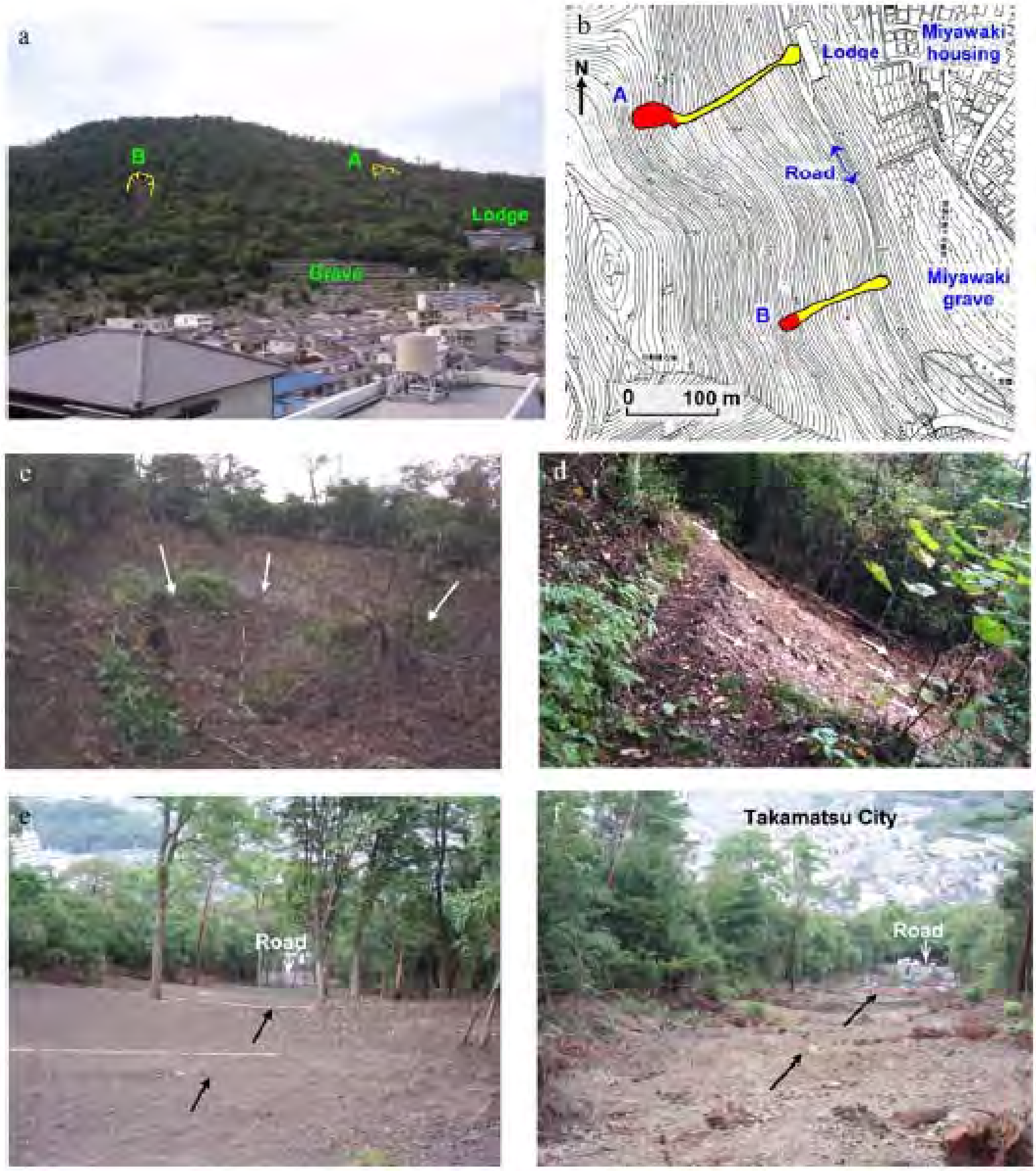

Fig. 3: View of Mineyama landslides. a. Photographic view of landslides ‘A’ and ‘B’, b. Topographical settings of Mineyama landslides, c. Scar of landslide ‘A’ after failure, d. Scar of landslide ‘B’ immediately after failure, e. Down slope view of landslides ‘A’ in October 2005 and f. Downslope view of landslides ‘B’ in October 2005

4 Methodology

4.1.1 Field Investigations

A total of 22 days of field visits and observations as well as measurements were carried out in the Mineyama landslides site in the year 2004, 2005, 2006 and 2007. The first preliminary field observations in the study area were performed immediately after the main landslide events in 2004. The regular visits has been made in 2005, 2006, and 2007 to observe the change in vegetation and response of the slopes during various rainfall events in theses years. In the typhoon season of 2007, a detailed field investigation of permeability and matric suction test was carried out. Soil thickness along the landslide scar and debris flow path was recorded during the field survey. GPS (Global Positioning System) was used to locate landslide scarp.

The main factor triggering the Mineyama landslides was extreme rainfall due to typhoon 0423 (October 19 and 20, 2004). Thus, permeability of the soil has to be ascertained to understand the hydrological characteristics of the slope materials. Thus, in situ permeability tests were performed. A total of five locations were selected for the permeability tests. The locations of the permeability tests are given in Fig 4., (A1 and A2 at landslide ‘A’ and B1, B2 and B3 at landslide ‘B’). These locations were chosen considering the fact that the soil properties on the crown part are responsible for the failure. The locations immediately after the failure scarp were selected for the study.

Hasegawa in situ permeability tester was used to determine saturated permeability of soil (Daitou Techno Green, 2009). Tensiometers were used to measure soil matric suction. Soil samples were collected for soil classification, clay mineral identification, and measurement of shear strength parameters in the laboratory. 100 cc tubes were used to collect samples to determine soil density. Undisturbed sample were recovered at locations A1 and B2. However, in other locations, undisturbed sampling was not possible owing to the presence of granule and small pebble in soil. So, disturbed samples were taken from those locations.

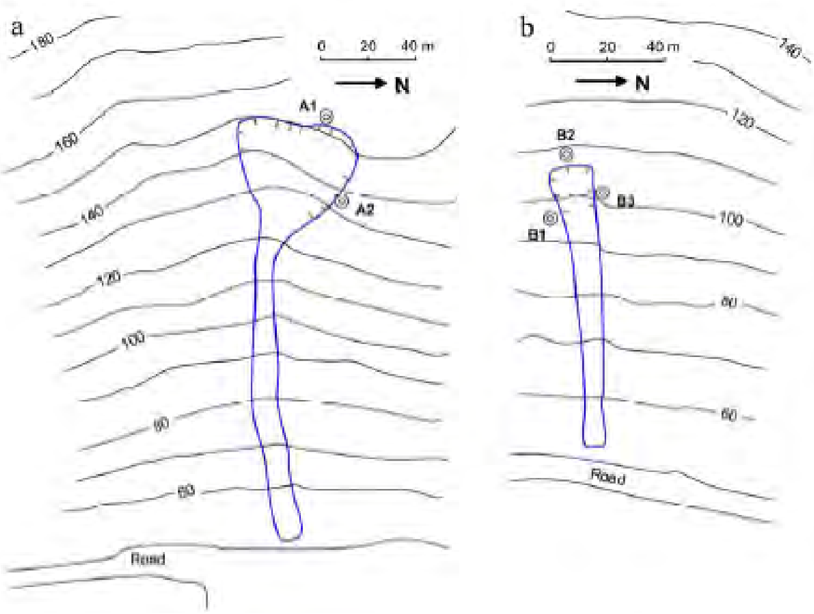

Fig. 4: Location of in situ permeability test, a. in landslide ‘A’ and b. in landslide ‘B’

4.1.2 Hasegawa in situ permeability tester

Hasegawa in situ permeability tester kit is a simple and easy to use in situ soil permeability test kit developed by Daitou Techno Green, Inc., Japan. Basically, till date, it has been used to understand ground permeability for tree and shrub plantation. In this research, Hasegawa in situ permeability tester was used for landslide study. Total five permeability tests were performed at locations A1, A2, B1, B2, and B3.

The Hasegawa permeability test kit consists of hole cover plate, float plate, scale of 60 cm and 100 cm, fixed pin and scale guide are main parts of the test kit. Each set of test kit is provided with double sets of accessories. As a result, simultaneous test on two nearby site are possible in one attempt. Depending upon the slope materials, ten to twenty litres of water is necessary for each test. Poly tank of twenty litres capacity is suitable for this purpose. Brief summary of test procedures are as follow.

1. Remove any upper part of organic materials of sites.
2. Make a hole of 20 to 40 cm deep using double scoop shovel (Fig. 5a).
3. Remove any gravel materials from base of the hole (Fig. 5b).
4. Install tubular scale with float plate and insert within scale guide bar (Fig. 5c).
5. Place hole cover plate, fix it by fix pin and tight scale guide bar with cover plate (Fig. 5d and Fig. 5e).
6. Note down reading of tubular scale when float attached with tubular scale touch base of hole (Fig. 5f).
7. Use siphon to pour water from poly tank into the hole (Fig. 5g).
8. Fill test hole with water and wait for one hour.
9. After one hour, again fill test hole with water and take reading of tubular scale after 20 minutes and 40 minutes (Fig 5h.).
10. If water is finished before 20 or 40 minutes, note down that time (Fig 5i.).
11. Relationship given in Fig 5j. is used to determine permeability of the soil layer in m/s.

4.1.3 In situ soil matric suction test

Commercially available Daiki tensiometer was used to measure in situ soil matric suction For the measurement of soil matric suction, display type tensiometer having ceramic sensitive part and pressure sensor was used in field. Illustration of Daiki tensiometer with display unit is given in Fig. 6. Twenty three locations were selected for soil matric suction of which eight locations were selected at Landslide ‘A’ and 15 locations were selected at Landslide ‘B’. Matric suction test points were located on both left and right as well as up slope and down slope of permeability test location as shown in Fig. 6. To get the best results and to allow good contact between soil and ceramic cup, tensiometers was installed 24 hours before the data measurement. Data were taken from all tensiometers to understand the spatial variation of soil matric suction around permeability test site.

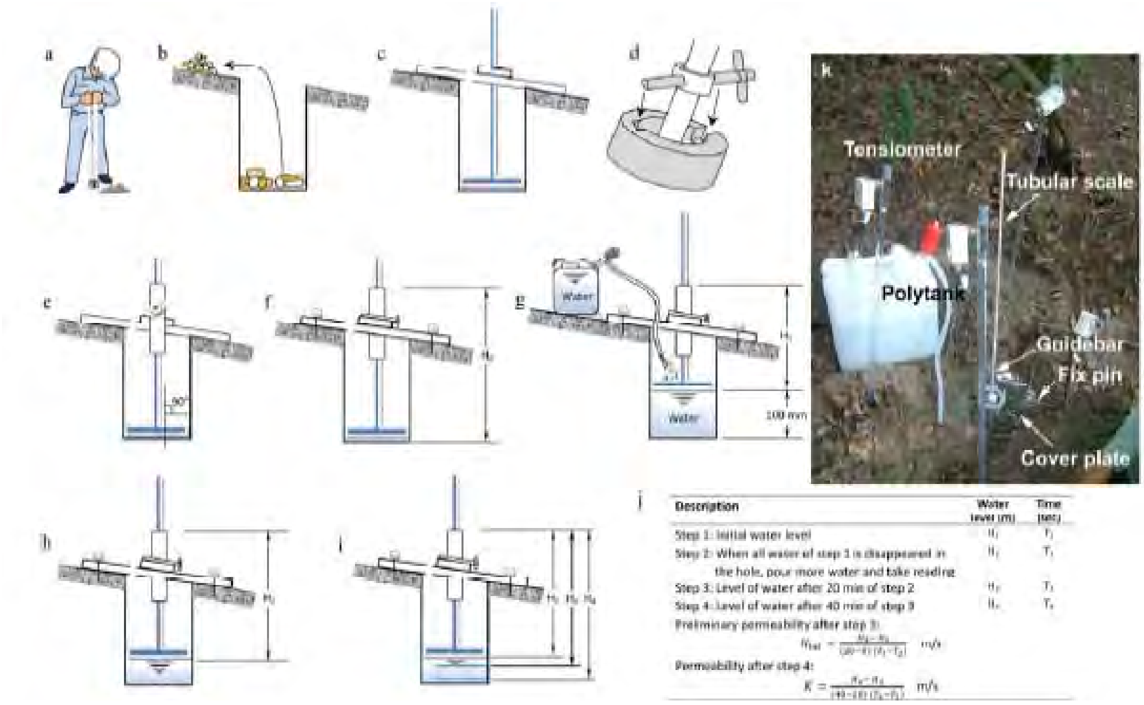

Fig. 5: Schematic illustrations of the Hasegawa in situ permeability tester and permeability determination method. 'a' and 'b' demonstrate process of excavation of hole and removing of any coarse materials on the base. Installation of tubular scale with float plate is illustrated in 'c', 'd' and 'e'. 'g', 'h' and 'i' demonstrate data measurement procedure. Short description of procedure and method of calculation of permeability is shown in 'j'. Permeability test at location B3 is shown in 'k'.

During the permeability test, data of matric suction were also taken at 5 min interval to get the information on the changes in matric suction during water percolation. However, noticeable changes in matric suction were not evident throughout the permeability test period since the soil permeability was relatively low.

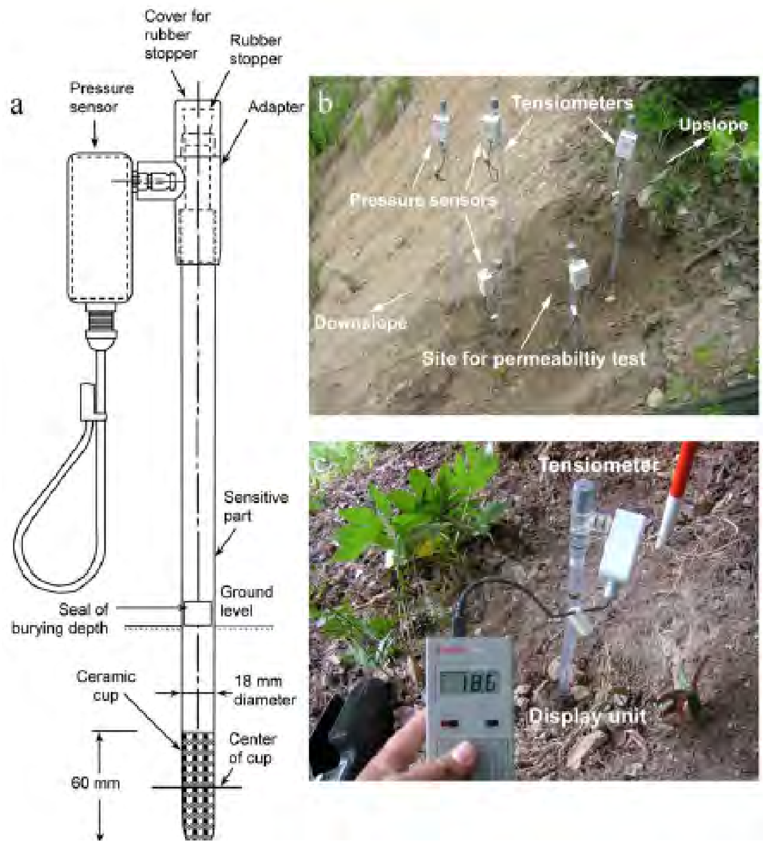

Fig. 6: **a. Schematic drawing of tensiometer, b. Installation of tensiometers in location A4 (scar of landslide 'A'), c. Display unit and matric suction reading at location B2.**

4.1.4 Vegetation and root study

The role played by vegetation in improving slope stability is well recognized and comprehensive reviews are found in the literature (Gray et al. 1980; Greenway 1987; Morgan and Rickson 1995; Howell 1999a and 1999b; Schmidt et al. 2001). Vegetation enhances slope stability via root reinforcement. Therefore, root reinforcement is also considered in this study because penetrating vertical taproots and sinker roots provide additional stability to the slopes.

A certain amount of research has been done on the forested hill slopes (Wu et al. 1979; Sidle 1991; Kondo et al. 2004; Nghiem et al. 2004; Kubota et al. 2004) to understand hill slope stability and root cohesion. Kubota et al. (2004) has mentioned that slope having deciduous broad leaves and mixed forest can have effective cohesion value in between 4.9 kN/m^2 and 6.8 kN/m^2. Kondo et al. (2004) has developed equations to estimate root cohesion with the help of root diameter and tensile strength.

The root shear resistance force (S_r) can be expressed on the basis of equilibrium of acting force (Waldron 1977, Wu et al. 1979, Kondo et al. 2004) as follow. Detail calculations is available in Kondo et al. 2004 and Fig. 7 was used for parameter illustration.

$$S_r = t(\sin\theta_r + \cos\theta_r \tan\phi) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Where, θ_r is inclination angle and ϕ is internal friction angle.

Eq. 1 suggests that if internal friction angle is given, the root shear-resistance (S_r) can be derived with the value of tensile strength and inclination angle (θ_r) of root. Hayashi (1998) has proposed the relationship of tensile strength (t) and the diameter (d) of root at the breaking point with a regression equation as follow.

$$t = k_b d^{2.03} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Where k_b is a correlation coefficient and its value is used according to plant species (Kondo et al. 2004). Wu et al. (1979) has proposed that the angle of θ_r usually range in between 48 degrees to 72 degrees. Thus, for n roots of per square meter, root cohesion c_r can be expressed as follow (Kondo et al. 2004).

$$c_r = \sum_{i=1}^{n} S_{ri} \quad \text{.............................} \quad (3)$$

From Eq. 3, vegetation root characteristics at landslide scar were recorded in the field. The number of roots per sq. m in the landslide scarp were counted. A total of five locations were selected for the root density measurement. In each location, diameter of plant root at landslide scarp was noted. A total of 40 records of root diameters were measured from five locations in both landslides 'A' and 'B' and used to calculate root tensile strength as per the Eq. 2. Minimum value of θ_r, described by Wu et al. (1979), were also considered in the calculation and apparent root cohesion values at locations A and B were determined and used in stability analysis.

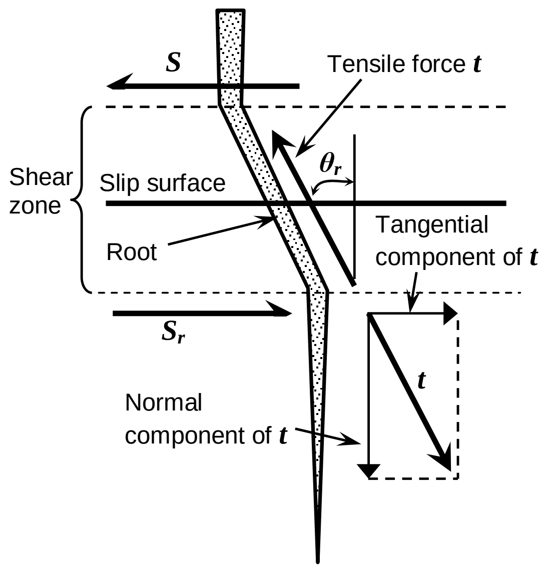

Fig. 7: Reinforcement mechanism in soil with root (modified after Kondo et al. 2004)

4.1.5 Experimental investigations

Soil samples collected from all five locations in both landslides 'A' and 'B' were investigated in the laboratory for index property identification, clay mineralogy, and shear strength parameter. All index parameters were determined in the laboratory along with the grain size distribution. Direct shear tests were conducted in undisturbed as well as reconstituted soil specimens. The soil sample was reconstituted with sieved material finer than 2 mm to eliminate large fragments that could have altered the measurements. All tests were performed in saturated and drained conditions. The applied normal pressure ranged between 100 and 300 kN/m^2. X-ray diffraction analysis was employed to identify the constituent clay minerals in the soil. The powder method was used for the X-ray diffraction analysis, in which all particles were crushed into fine powder and constituent minerals were identified using X-ray diffraction patterns. The ethylene glycol treatment method was used to confirm the expansive minerals of low values of 2θ (less than 15^o, θ is angle of incidence of x-ray).

4.1.6 Hydrological and stability analysis

Hydrological and stability analysis were performed to analyse the variation of pore water pressure and slope instability in the Mineyama landslides. The modelling of transient pore water pressure and slope stability has done with finite element method (FEM) based computer applications namely SEEP/W and SLOPE/W (GeoStudio, 2005). Modelling parameters has obtained from both field and laboratory investigations as well as from the literatures. In SEEP/W and SLOPE/W, the model code is based upon the equations of motion and mass conservation. Both saturated and unsaturated flows in soil were simulated using a modified version of Darcy's law. For unsaturated soil conditions, the hydraulic conductivity function of a soil is described by the relationship between water content and pore water pressure (Fredlund and Rahardjo, 1993). In case of transient flow, the hydraulic head is no longer independent of time and volumetric water content changes with time. Thus, Richard's equation is suitable for describing transient flow (Richards 1931; Fredlund and Rahardjo 1993) as follows:

$$\frac{\partial}{\partial x}\left(k_x \frac{\partial H}{\partial x}\right)+\frac{\partial}{\partial y}\left(k_y \frac{\partial H}{\partial y}\right)+q=\frac{\partial \theta}{\partial t} \quad \text{......................(4)}$$

where H is total head, k_x is hydraulic conductivity in the x-direction, k_y is hydraulic conductivity in the y-direction, q is applied boundary flux, θ is volumetric water content, and t is time.

The state of stress and soil properties influence the change in the volumetric water content of soil, and for both saturated and unsaturated conditions, the state of stress is usually described in the form of $\sigma- u_a$ and u_a-u_w (Fredlund and Morgenstern 1976, 1977; Fredlund and Rahardjo, 1993). The parameter σ is the total stress, u_a is the pore air pressure, and u_w is the pore water pressure. SEEP/W assumes that the pore air pressure remains constant at atmospheric pressure during transient flow. Hence, $\sigma- u_a$ does not influence the changes in volumetric water content; rather, the stress variable u_a-u_w is responsible for changes in volumetric water content. When considering u_a is considered as a constant variable, the change in volumetric water content solely depends on the pore water pressure changes. As a result, the change in volumetric water content can be related to a change in pore water pressure and Eq. 4 can be written in the form of the soil-water characteristics function (relation between the volumetric water content and the negative pore water pressures) as follows (Fredlund and Rahardjo, 1993; Tsaparas et al. 2002):

$$\frac{\partial \theta}{\partial t}=m_w^2 \frac{\partial u_w}{\partial t}=m_w^2 \rho_w g \frac{\partial H}{\partial t} \quad \text{...................... (5)}$$

where m_w^2 is the coefficient of volumetric water change with respect to a change in negative pore water pressure, i.e. soil matric suction (u_a-u_w) and is equal to the slope of the soil water characteristic curve, ρ_w is the density of water, and g is gravitational acceleration.

Combining Eqs. 4 and 5 gives

$$\frac{\partial}{\partial x}\left(k_x \frac{\partial H}{\partial x}\right)+\frac{\partial}{\partial y}\left(k_y \frac{\partial H}{\partial y}\right)+q=m_w^2 \rho_w g \frac{\partial H}{\partial t} \quad \text{...................... (6)}$$

Eq. 6 is the final form of the governing equation of water flow through the unsaturated soil and it is an explicitly non-linear equation. SEEP/W uses finite elements to solve Eq. 6. To obtain the numerical solution of Eq. 6 in SEEP/W, it is necessary to provide the permeability function (permeability with respect to water versus negative pore water pressure), soil-water characteristic curve, boundary flux, and initial hydraulic head in the course of defining the problem.

5 Results of investigations

5.1.1 Geology and Geomorphology

As already discussed under the heading study area, the main rock type in the landslide area is bronzite-andesite and it is moderately weathered. The rock is dark grey in colour and gives strong metallic sound when hammer or when two rock pieces struck together. The rocks on the landslides scar are extensively fractured. Poorly defined sheet joints are also noticed on the scar of landslide A, however, at scar of landslide of B, colluvium soil cover is predominant.

Length of the both landslide slope is less than 200 m and both slides are in similar elevation, and have the same types of vegetation, rocks, and soils. The upslope is relatively rockier than the central and lower part of the slope. Geomorphologically, both slides are situated on topographic hollow or zero-order basin. Slope curvature strongly suggests that the both landslide scars are situated on the highly susceptible zone of rainwater accumulation.

5.1.2 Geotechnical characterization of soils

The stratigraphy at the selected sites on Mineyama landslides includes homogenous residual soil and bedrock bronzite andesite. In both Mineyama landslides, failure surfaces were found at the contact between the residual soil and moderately weathered bedrock. The grain size distribution curves of the soils are shown in Fig. 8. Unified soil classification of the soil shows that the soil type of the selected sites is mainly silty clay of medium plasticity with sand and gravel. The soils from all the five locations have more than 50% fine materials. Their grain size distribution curves are more or less the same for finer portion. The in situ permeability tests revealed that Mineyama landslides have permeability in the range of 10^{-7} m/s to 10^{-8} m/s. The in situ matric suction measurements suggest that increase in rainfall causes a decrease in soil matric suction. A total five cycles of measurements were taken between June 26 to July 15, 2007. A maximum soil matric suction of 61.4 kPa and a minimum 0.6 kPa were measured at different locations of the investigated area during the period. During one of the investigation periods, typhoon 0704 (named as Man-Yi) hit the sites and the soil matric suction fluctuations in the soils were recorded. Abrupt change in soil matric suction and moisture content were observed with frequent rainfall during investigation period (Fig. 9).

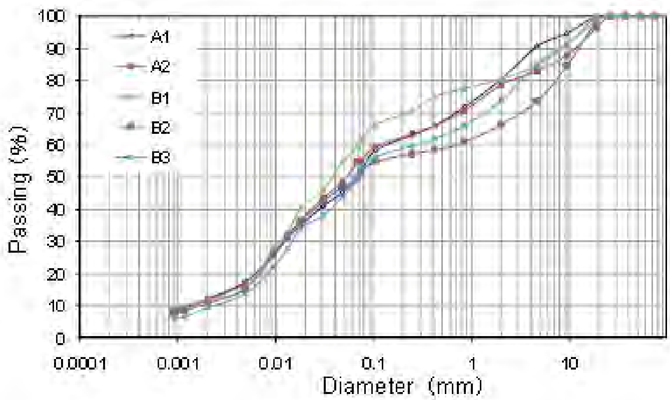

Fig 8, Grain size distribution curve of soils in selected sites

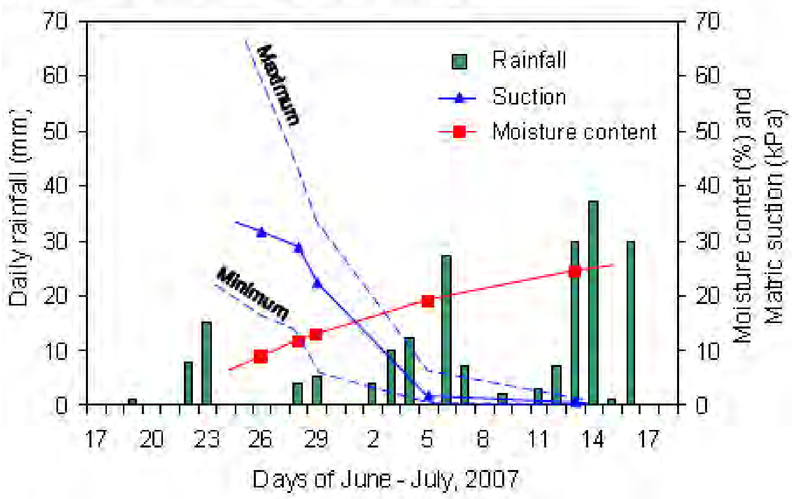

Fig. 9: Variation of matric suction and moisture content of soils at Mineyama landslides site during typhoon rainfall of 2007. Average matric suction and moisture content are used in this plot. Maximum and minimum value ranges of matric suction are also shown in graph.

From Eq. 3, root cohesion of the Mineyama landslides site has also estimated. The soil data obtained for both the landslides is given in Table 1. The data obtained were used for hydrological and stability analysis. Cohesion and friction angle were estimated by direct shear test and root cohesion estimation method (Eq. 3), volumetric water content in saturation and unit weight were estimated in the lab. Soil permeability was measured in field. These parameters were used in hydrological and stability analysis.

The results of the x-ray diffraction tests are provided in Table 2. From the X-ray diffraction patterns, it is clear that the main constituent minerals of the samples are quartz, feldspar, metahalloysite, smectite, and illite.

Table 1. Geotechnical properties of Mineyama landslides (A and B).

Landslide	Average slope angle (^{o})	Landslide scarp length (m)	Effective cohesion * (kN/m^2)	Unit weight* (kN/m^3)	Effective friction* angle (^{o})	Volumetric water content (%)	Hydraulic conductivity (m/s)
A	25.5	46	4.4	13.1	21.5	45	1.167×10^{-7}
B	26.5	25	5.2	13.7	19.4	45	1.111×10^{-7}

*These values are mean values and factor of safety distribution via sensitivity calculation was performed during stability analysis.

Table 2. Result of X-ray analysis of soils collected from Mineyama landslides site

Landslide	Quartz	Feldspar	Metahallosite	Orthopyroxene	Smectite	Illite
A	+++++	++++	++	++	++	+
B	++++++	+++	++	++	+	+
Abundance: ++++++ high + low						

6 Hydrological and slope stability modelling

6.1.1 Model parameters

For the hydrological and slope stability modelling in GeoStudio (2005), both laboratory and field data were used to assess the hydraulic properties of the residual soils. The thickness of the residual soil layers were selected as per the field measurements. In Mineyama landslides, thickness of soil layer range from 0.7 m to 1.8 m. Both landslide affected slopes were modelled by adopting slope angles measured in the field. Taking reference of the landslides scarp, 25 m up slope and 25 m down slope length was used in modelling along with total length of scrap as shown in Fig. 10. Locations of the scarps were fixed from GPS data and field measurements. Contact between soil and rock was considered to be geometrically planar, having soil layers parallel to the ground surface. The geotechnical parameters presented in Table 1 were used in the modelling.

The volumetric water content and the hydraulic conductivity functions were obtained from curves derived using similar grain size distributions function provided in GeoStudio (2005) by adjusting the saturated water content and permeability values to the actual measured values. The main failure events occurred on 20th October 2004, and it is difficult to describe the initial pressure distribution prior to the rainfall events because almost all days of October have considerable rainfall and no data were available for pore pressure variation. Thus, the simulation has performed considering typhoon rainfall of 19th and 20th October of 2004.

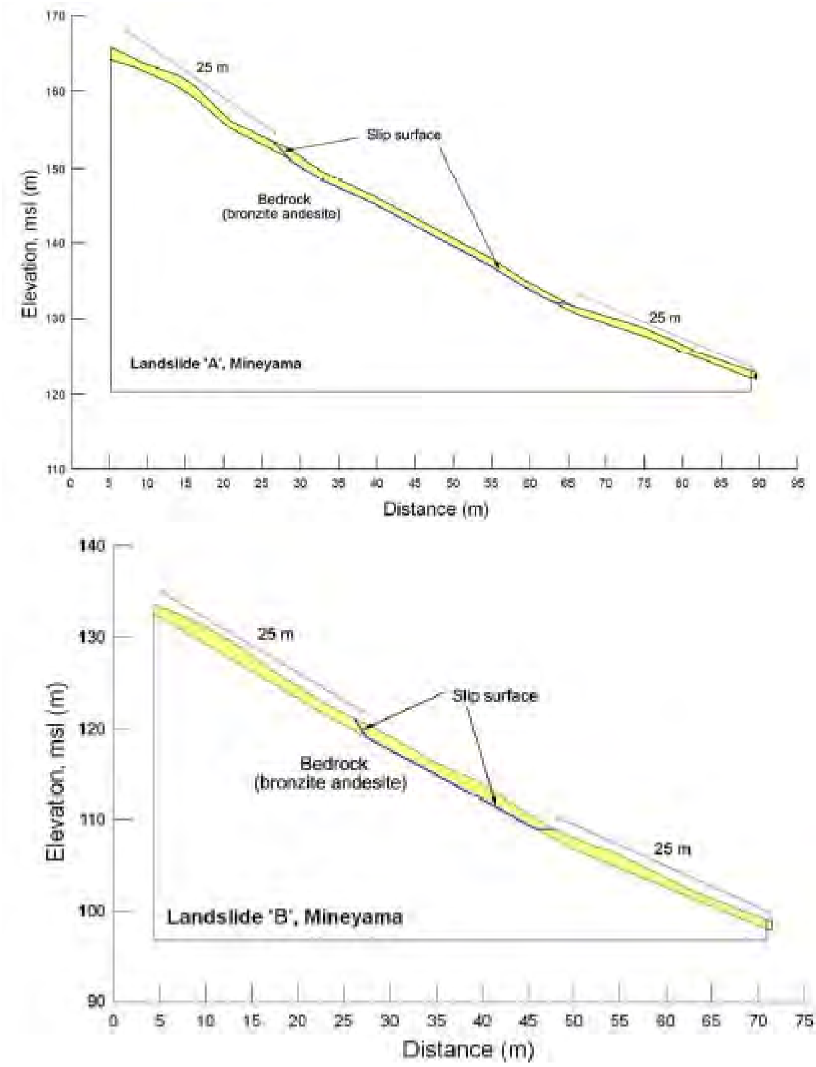

Fig. 10: Slope geometry of landslide 'A' and landslide 'B' locations at Mineyama hillock

Rainfall started at 4:00 hours on the 19th October. The time and the date were set as a start up point, and the initial water table has defined above the bedrock with a maximum negative pore pressure of 61.4 kPa. For the boundary conditions, a transient flux function with values equal to the hourly rainfall intensity for 19th and 20th October of 2004 has applied to the nodes along the ground surface. A null flux condition has assigned to the upslope and down slope vertical faces of the model. In the case of the down slope vertical face, potential seepage face review option and the infinite elements was also selected to extend the actual right edge to infinity in the positive x-direction, to avoid an unnatural impermeable border, and minimize any side effects. Null flux boundary has also imposed at the lower boundary of both simulations. The complete layout of model is shown in Fig. 11. The homogeneous residual soil above the bedrock was considered as single layer for modelling. A null flux condition was assigned to the upslope, down slope and lower boundaries of soil layer.

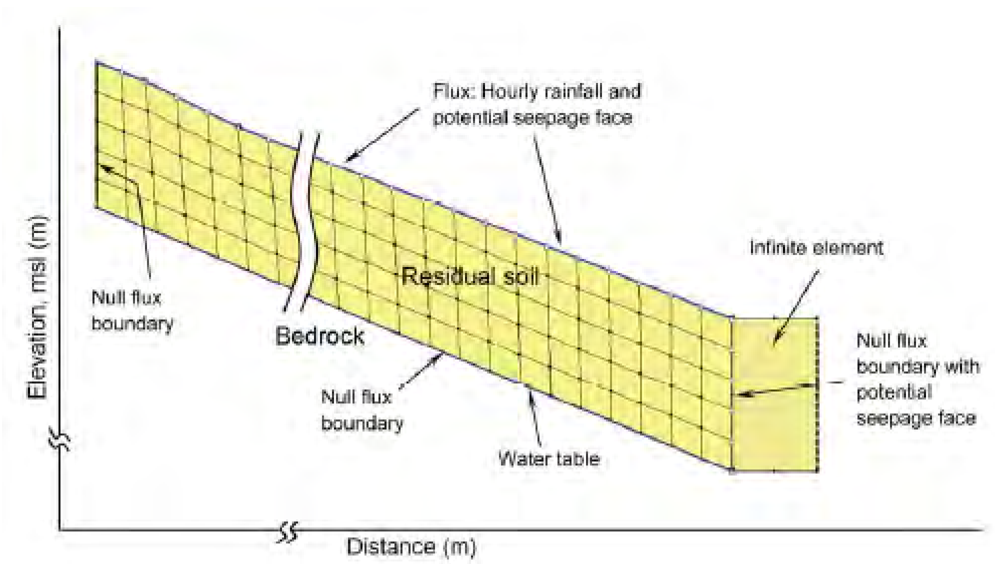

Fig. 11: Schematic illustration of finite element description of slope geometry.

Rate of evaporation was not considered in this modelling and it is an important limitation of this study. In reality, the actual evaporation or evapotranspiration rate from the surface is a function of vegetation cover, soil moisture, and sunlight hour. SEEP/W v.4 of GeoStudio can handle an evaporative flux only by defining a negative flux along the ground surface. Thus, to incorporate evaporation as a negative flux in the seepage analysis, an average evaporation flux rate was applied along the surface during the preliminary phase of analysis. The result of pore pressure reduction, however, was very unrealistic in comparison to the field problems. Other researchers have had similar experiences during seepage analysis (Gasmo et al. 2000; Tsaparas et al., 2002). In general, during extreme typhoon rainfall events negligible evaporation takes place, since this study is focused on the changes in pore water pressures during extreme typhoon rainfall events, evapotranspiration is not a significant Factor Likewise, in the earlier phase of simulation, to ascertain the infiltration process of rain water, a Green-Ampt solution for unsteady rainfall model developed by Parsons and Munoz-Carpena (2000), was used to ascertain the infiltration process at the Mineyama landslides site. Gampt v0.3 follows the method of Chu (1978), Mein and Larson (1973) and Skaggs and Kaheel (1982) to calculate infiltration for unsteady rainfall using the Green and Ampt (1911) equation. Results of Gampt v0.3 suggested that there was no surface ponding period during the rainfall of 19th and 20th October 2004 at the Mineyama landslides site. Thus, surface runoff was not considered in this model and whole rain was assumed to completely infiltrate through soil. Two simulations (for locations A and B) were done. For the simulations, the typhoon rainfall event of 19th and 20th October of 2004 was divided into 2700 time steps of 1 min length (total 45 hours) and seepage into the soil were simulated using SEEP/W v.4 of GeoStudio.

6.1.2 Result of seepage analysis

From the seepage simulation, the values of pore water pressure in both selected slopes were usually high in October 20th 2004, the date of failure. Pore water pressure development was usually transient because of the permeability of soil and potential seepage faces on the slope. Pore water pressure variations in 19th and 20th October of 2004 at the slip surface of both landslides are shown in Fig. 12. The distribution of the water table varies in both simulations. In both simulations, rainfall of 4:00 to 20:00 hours on the 19th October was found to be responsible to saturate soil for several hours. As a result, although there was no rainfall between 20:00 hours of 19th October and 7:00 hours of 20th October, seepage dissipation and continued increment of pore water pressure were observed at many nodes along slip surfaces during that period. Consequently, rainfall between 7:00 hours and 14:00 hours of 20th

October increased the pore water pressure to a maximum of 9 kPa in some nodes of Landslide ‘A’ and around 12 kPa in some nodes of landslide ‘B’ (Fig. 12).

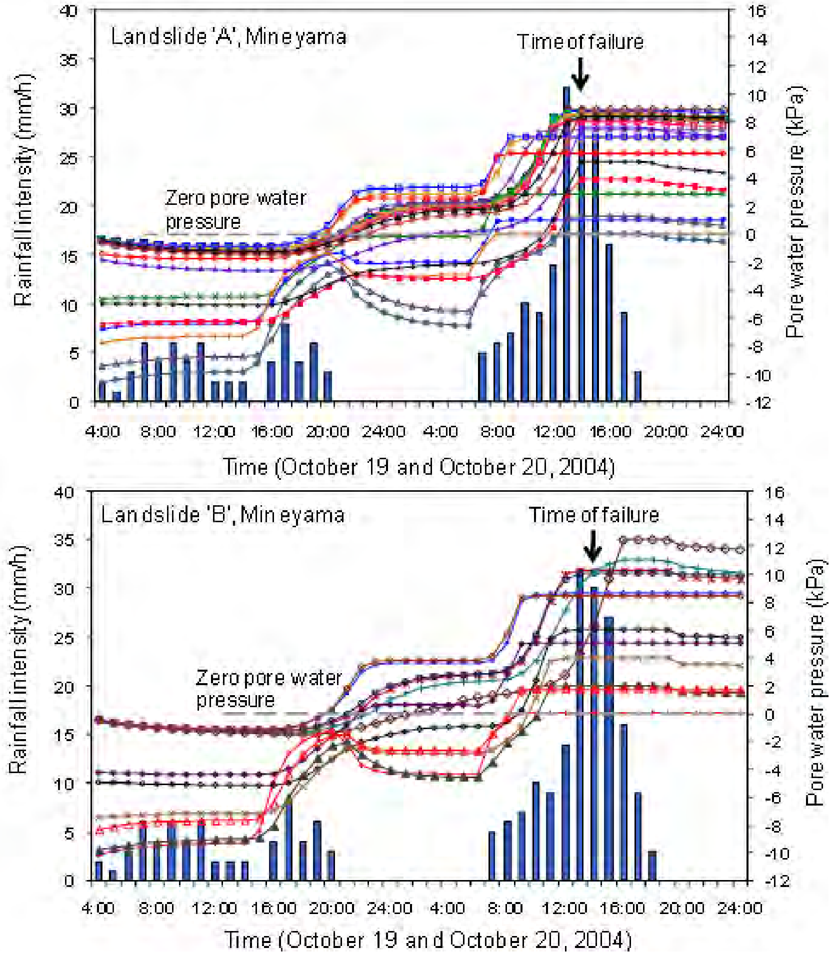

Fig. 12: Pore water pressure variation in 19th and 20th October of 2004 at the slip surface. The location of slip surfaces were fixed as per the field measurement and are shown in Fig 10 for both sites. Curves of low pore water pressure resemble pore water pressure at nodes of higher elevation along the slip surface.

6.1.3 Stability analysis and result

Slope stability analyses were conducted for both landslide locations, using pore water pressures determined at different time steps as input data for a limit equilibrium analysis performed with SLOPE/W v.4 software (GeoStudio 2005). The factor of safety of the soil cover was computed with the Morgenstern-Price method. In both models, fully specified (GeoStudio 2005) slip surface (same as failure condition of 20th October 2004) was used at a surface distance of 25 m from the upslope edge and down slope edge (see Fig. 10). During the stability analysis, critical slip surfaces were optimized with a maximum of 2000 iterations. In the simulation, the fully specified slip surface was also optimized to obtain the most critical value.

The average measured value of unit weight, cohesion and friction angle (Table 1) may vary from the actual values of natural slopes. The measured angle of friction and unit weight are relatively low probably because of organic content (dead roots) on soil. Therefore sensitivity analysis was done in the stability analysis. Sensitivity analysis is a heuristic analysis which examines the dependency of various parameters used in a calculation. Therefore, sensitivity analysis carries out along with stability analysis to assess the variations in the factor of safety with respect to changes in engineering parameters such as cohesion, friction angle, and unit weight. Thus, in the present study, the factor of safety distribution via sensitivity calculation (an option available in Slope/W) was considered to get the factor of safety distribution value. For this purpose, minimum and maximum values of cohesion, friction angle, and unit weight measured in the field were used to estimate value of delta and steps from the mean.

In all simulations, factors of safety decreased abruptly from 7:00 hours of 20th October 2004. At the time of failure (14:00 hours), the factor of safety at landslide 'A' was less than 1. Similarly, for landslide 'B', although time of failure was not known for this landslide, the factor of safety was found to be less than 1 after 14:00 hours and it is congruent with the previously assumed time of failure. The variation of factor of safety in both modelling scenarios is shown in Fig. 13.

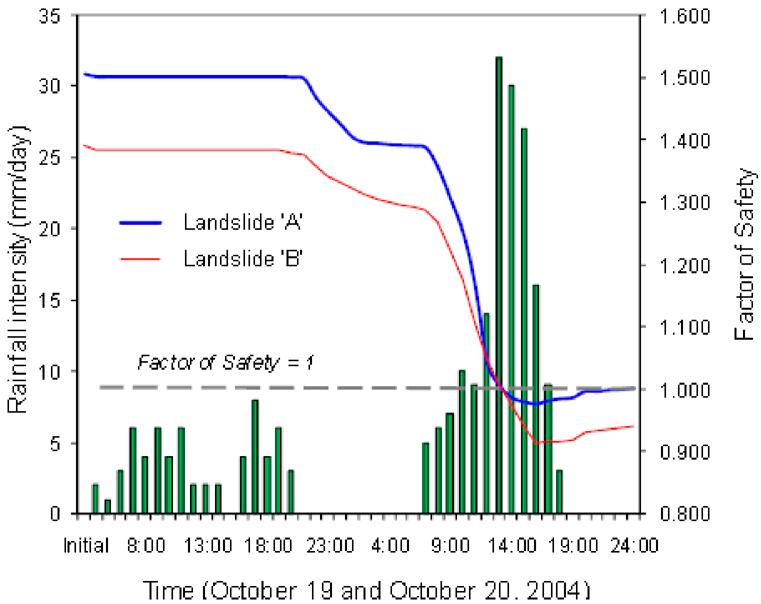

Fig. 13: Change of factor of safety with rainfall in 19th and 20th October of 2004. The seepage dissipation and continue increment of pore water pressure after rainfall of 19th October possesses substantial effect in reduction of factor of safety of slope on 20th October

7 Discussions and conclusions

In this paper, a comprehensive geological, geomorphological, and geotechnical investigation is discussed to understand the contributing parameters involved in landsliding triggered by typhoon rainfall in the andesitic terrain.

The X-ray diffraction study confirmed that the soil consists of quartz, feldspar, metahalloysite and illite as major constituent minerals, along with significant amounts of smectite, and amorphous silica. Smectite was detected in the soil from X-ray analysis. When such swelling minerals are present in slope materials, the slopes are prone to failure. Swelling minerals such as smectites expand when they become wet as water enters the crystal structure and increases the volume of the mineral. During rainfall, they would swell and slopes become prone to failure. Previous studies (Kerr 1972; Bardou et al. 2004; Yatabe et al. 2000) involving the role of swelling clay minerals in activating a landslide have shown that even a small amount of swelling clay minerals in soil greatly affects its strength behaviour. Thus, the presence of swelling minerals in the soils of the study area increase the probability of landslides in the Mineyama hillock.

The residual soil above the bedrock mainly consists of silty clay with fine sand and granules. In situ measurements of permeability of soil revealed that the soil is moderately permeable and have hydraulic conductivity in the range of 10^{-7} m/s. The residual soil is well vegetated by shallow rooted vegetation, is relatively loose, and its bulk density is relatively low. Thus, rainwater can easily infiltrate within the soil. The thickness of residual soil on slopes is also supporting the issue of infiltration and failure. On the scar of the Mineyama landslides, the average soil thickness was 1.2 m. This thickness on uphill slopes (average inclination 28^o to 35^o) of zero order basins is found to be favourable for landsliding.

After understanding of geological and geomorphological setting of the area as well as geotechnical properties of slope materials, seepage modelling and stability analysis were performed. Hourly rainfall data was used in the simulations. Fluctuation of transient pore water pressure with respect to rainfall was observed in the both simulated slopes. The stability analysis of the simulated slopes showed that the factor of safety was less than 1 after 14:00 hours on the 20th October 2004. The reported time of failure by eyewitness account tally with the simulated time. Thus the modelling was significant and representing the real scenario. The transient pore water pressure and factor of safety variations in the andesitic slopes during typhoon rainfall were understood from this modelling. In general, the andesitic slopes are prone to failures due to the significant pore water pressure response to the rainfall, which leads to the considerable loss of soil matric suction and rise of positive pore water pressure under moderately intense rainfall.

Similarly, seepage modelling suggested that the water dissipation period of slope materials in andesitic terrain was comparatively large. As a result, pore water pressure was increased at later stage of first phase rainfall (i.e. 19th October, 2004). Presence of more than 50% of fines (silt and clay) in slope materials supports the lower rate of seepage dissipation. When, the soil permeability with respect to water is low (in the range of 10^{-7} m/s) then the pore water pressures may not change significantly during the rainfall, but they can start increasing towards positive values after the end of the wet period (Tsaparas et al., 2002). Same situation was appeared in the case of Mineyama landslides also and there was not any significant change in pore water pressure during first phase of rainfall. During forty-five hours of typhoon rainfall events, there was a period of 10 hours (in night) of no rainfall. Seepage simulation suggested that the first phase rainfall of 19th October, 2004 was depicting antecedent moisture effect on soil. Consequently, when rainfall started at 7:00 hours of 20th October, 2004, the factor of safety of slope decrease abruptly and failure occurred at 14:00 hours.

The following concluding remarks can be drawn from this study.

- Soil characteristics, low internal friction angle of fines in soil, and the presence of clay minerals were the main contributing parameters for slope failures on Mineyama hillock.
- The clay mineralogy of slope materials was also a major contributing factor for rainfall-induced landslides in the andesitic terrain.
- Results indicate that the simulation of the saturated and unsaturated flow within the soil, using a finite element seepage analysis can provide useful information of the pore water pressure, total head and volumetric water content in response to the different intensity and phases of the typhoon rainfall event.
- The results of this study show that transient pore water pressure distribution appears to be a key triggering mechanism behind rainfall-induced slope failures in residual soil slopes of andesitic terrain. Other researchers in the residual soils of granitic and sedimentary terrains (Rahardjo et al. 1995; Rahardjo et al. 2002, Rinaldi et al. 2004; Dapporto et al. 2005; Dahal et al. 2008a) make similar observations.
- Antecedent rainfall affects landslide stability by reducing soil matric suction and increasing transient pore water pressure. Seepage analysis clearly suggests that antecedent moisture content vigorously enhance the pore water pressure build-up the failure process in the residual soil of andesitic terrain in west Japan. Thus, in the andesitic terrain of western Japan, typhoon rainfall of one or two days is always responsible for shallow slope failures.

8 Acknowledgements

The authors would like to thank Ms. Aika Mino, Mr. Toshiaki Nishimura, Mr. Toru Mimura, Mr. Yasushi Hamada and Mr. Toshiaki Takabatake for their help in the collection of field data. The authors also acknowledge local forest office of Takamatsu City for providing permission to enter the forest for investigation. Dr. J. E. Parsons of NC State University and Dr. R. Munoz-Carpena of University of Florida are sincerely acknowledged for providing free access to Gampt (Green-Ampt Infiltration for Unsteady Rainfall Model) software. Mr. Anjan Kumar Dahal is acknowledged for his technical support during the preparation of this paper. Finally, special thanks go to the two anonymous reviewers for their useful comments that improved the manuscript substantially.

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