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Study on Early Warning System for Debris Flow and Landslide in the Citarum River Basin, Indonesia

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Abstract. Study on debris flow early warning system at Citarum river basin is very beneficial in development of technology for monitoring and prediction of debris flow where linkages between rainfall intensity, area of high susceptibility of landslides and debris flow occurrences have been carried out. Three possible approaches of early warning system have been studied. The first approach is based on map of high susceptibility of debris flow and landslides which is function of slope stability, soil characteristics, land cover, geological formation, and the rainfall intensity duration thresholds. This early warning is qualitative and it only informs people the areas of high susceptibility of landslides and the amount of thresholds rainfall which people should be aware. The second approach that is based on prediction of debris flow and landslide from a distributed sediment yield transport model. The third approach is based on a real time monitoring of hydrological network and soil or ground movement monitoring in the areas of high susceptibility of debris flow-landslide and real time predicting of debris flow and landslides based on sediment yield transport model. This approach is quantitative and warning can be informed to the people few hours ahead.

Keywords. Landslide, debris flow, rainfall threshold, early warning, sediment, Citarum.

1. Introduction

Debris flow and landslides cause loss of life and millions of dollars in property damage almost annually in Indonesia. As the population increases and the social society become more complex, the economic and social cost of landslides will continue to rise unless there is a significant intervention and action.

Geographical location of Indonesia causes most of the islands have experiencing high rainfall and numerous active volcano. Having those characteristics, Indonesia is very susceptible to various natural disasters such as flood, debris flows, droughts, earthquake, tsunami, volcano eruption, and landslides. Indonesian Government has several policies and pays an effort to mitigate disaster by developing structural and non structural measures. Early warning system is one of the way of nonstructural measures to reduce it.

The flood, debris flow and landslide are hit the Citarum River Basin almost in every year and caused extensive damage. Watershed erosion is also a serious problem in the upper river basin where hillsides are steep. The soils derived from volcanic tuff are easily erodible and prone to landslides. Hydrologic characteristics have been changed by land degradation, as a result, flood, debris flow and landslides are very frequent during the rainy season. Therefore, there was an urgent need to devise countermeasure against frequent disasters. "Such big downpours could be a blessing and solve water scarcity problems. But on the other hand, it would lead to more natural disasters such as floods and landslides, especially if there is no effort made to replant the forest area around upstream Citarum so as water can be retained.

Establishment of debris flow warning system in areas where linkages between rainfall intensity and debris flow occurrences have been identified. It will help to mitigate a disaster. Development of early warning system of potential debris flows will be very beneficial to reduce lost of life, economic and social lost. A study on early warning system in Citarum river basin will be very beneficial for development technology in monitoring, predicting debris flow by development of spatial and temporal model and producing map of susceptibility landslides.

3. Study Area

The Citarum River Basin is the largest river on the West Java Island. It drains a watershed area of around 6,080 km². The 269 kilometers Citarum River originates from Wayang Mountain with elevation of 2198 m above mean sea level south Bandung. In the first 25 km, the river follows a steep slope of 0.033 then flows onto the middle part of the basin with slope of 0.0033 starting at Bandung for another 169 km. In the lower parts, the river meanders across an alluvial plain for about 75 km before reaching the Java Sea. There are three cascading reservoirs in the Citarum river basin namely Saguling, Cirata and Jatiluhur. Those reservoirs are built not only for generating hydropower, water supply for irrigation, industrial and domestic but also uses to regulate and traps of sediment. Geomorphology of the catchment consists of volcanic cone, Tuff, Tuff Sand, Lapili, Breccia Aglomerat, Breccia and lahar, Breccia, Lava Andesit, Tuff Breccia lahar, and lava, Andesit and Dasit.

Based on the record of Meteorology and Geophysical Agency, the Citarum river basin has an average monthly temperature of 22.8 to 26°C, average annual rainfall of 1500 – 4000 mm, average monthly wind speed about 2 to 9 knot and average monthly evaporation rate between 120 mm to 150 mm. The population in the Upper Citarum basin is rapidly growing with the urban area expanding around Bandung, the capital of West Java, and disasters are frequent during the rainy season. In the upper Citarum river basin, there are 12 river sub basins which have very steep slopes flow into upstream of Citarum River.

3. Debris flow and Landslide

Debris flow can be triggered by a variety of mechanism such as heavy rain, rapid runoff at hilly slope, flash flood, erosion, rapid melting of volcanic debris during eruption, and earthquake. Two triggering thresholds that relate to different
time scale need to be considered namely an antecedent rainfall threshold and a storm intensity duration threshold. These two thresholds can indicate a different level of potential hazards.

Landslide failures typically occur on steep slope and when rainfall infiltrates through a block of soil. The block of soil gradually saturates, pore water pressure increase and the shear strength decrease. Landslide problems can be caused also by land mismanagement, particularly in mountain, like upstream Citarum river basin.

In areas burned by forest and brush fires, a lower threshold of precipitation may initiate landslides. Land-use zoning, professional inspections, and proper design can minimize many landslide, mudflow, and debris flow problems. A complete prediction of the debris flow and landslides process would include assessments of “where”, “when”, and “how big”. There are some factors which usually controlling landslide namely, rainfall, earthquake, volcanic eruption, vegetation, geological condition, and morphology / slope.

The characteristics of landslides in Indonesia can be categorized into first is slow movement, creeping and no casualties but large damaged area and second is rapid movement with rock, earth, debris flows, causalities and large numbers of damaged. From data collection, there are some typical landslides disaster occurred in Citarum river Basin.

1. Landslide disaster in West Java at Cikalong Wetan, Bandung District on 16 September 2003 that caused at least 13 peoples feared dead and 7 homes have been swept away by the landslide.
2. Landslide disaster in West Java at Cililin, Walahir village on 21st of April 2004 that caused at least 15 people dead, 43 houses collapsed and heavy damaged, 60 Ha of paddy fields and more than 70 goats have been swept away by the landslides. This landslide mainly is due to very steep slope, high weathering products, land-use changed and high intensity of rainfall.
3. Landslide disaster in West Java at Leuwigajah, Cimahi on 21 February 2005 that caused at least 123 people are dead and 70 homes have been flattened by the landslide.
4. Landslide disaster in West Java at Rongga, Bandung District on 3 March 2005 that caused at least 2 people dead and a car overturned and dragged along the river.

4. Rainfall thresholds
To know when debris flows are likely to occur, an antecedent rainfall and storm intensity duration thresholds needs to be developed. Landslides data from 2003 to 2005 and rainfall data monitoring are used as references to develop rainfall thresholds in Citarum river basin. Observations of daily rainfall at the closest stations to landslide locations are selected. The rainfall distribution patterns of Bandung monitoring station are used to distribute the daily rainfall data of the selected rainfall stations. The plots of the relationships between rainfall durations and cumulative rainfalls at each date of landslides occurred can be constructed and analyzed for developing rainfall thresholds and the results can be seen in Figure 1. This rainfall intensity duration thresholds can be used to indicate different levels of potential hazards. A lower threshold identified a rainfall level below which significant debris flow hazards are considered unlikely, and above which debris flow are likely. An upper threshold represents a rainfall level above which abundant debris flow large enough to destroy infrastructures.

Fig.1 Rainfall Thresholds for Citarum River Basin

5. Landslide Susceptibility

The second step is creating a susceptibility map for the upper Citarum River basin from the geology, soil, slope, land use and rainfall information. The susceptibility maps were created through an iterative process from two kinds of information. Firstly is statistic map which consist of land use map, geology map, slope map, distribution of soil movement...
and landslide image. Secondly is dynamic map which consist of rainfall map, earthquake, or eruption of volcano. By analyses of GIS, those maps can be superimposed so that the resultant maps of relative susceptibility represent the best estimate generated from available inventory data can be produced.

Figure 2 and 3 show landslide susceptibility maps based on slope angle and multiple inventories of aspects such as geology, land use soil, slope and rainfall by using statistical analysis.

Assessing Dynamic Map of Slope Stability Using Factor of Safety

Engineering geologists often use the relationship between shear stress (the component of stress that operates in the downslope direction, \( \tau \)) and shear strength (the properties that resist shear stress, i.e., cohesion + normal stress (\( S \))) to carry out a slope stability analysis. The ratio of shear strength to shear stress is called the factor of safety (\( FS \)). For modeling shallow landslides the simplified case of a planar failure on an infinite slope is generally accepted and the FS is calculated by Equation 1 (Ward et al., 1981). When this ratio is greater than 1, shear strength is greater than shear stress and the slope is considered stable. When this ratio is close to 1, shear strength is nearly equal to shear stress and the slope is unstable.

\[
FS = \frac{2(C_s + C_i)}{\gamma_s dsin(2\beta) \tan \phi}
\]

where:

\[
L = \frac{q_o}{\gamma_s d} + m\frac{\gamma_{sat}}{\gamma_s} + (1-m)\frac{\gamma_{sat}}{\gamma_s}
\]

in which \( C_s \) and \( C_i \) is the effective root and soil cohesion; \( \phi \) is the effective angle of internal of soil; \( d \) is the soil depth; \( \beta \) is the slope; \( q_o \) is the vegetative surcharge, \( \gamma_{sat} \) is the weight density of soil at field moisture; \( \gamma_s \) is the weight density of water; \( \gamma_w \) is the weight density of saturated soil; and \( m \) is the relative saturated depth (thickness of saturated zone divided by soil depth). Most of are can be spatially variable but it is assumed that only \( m \) is time-varying, therefore, the factor of FS is a function of \( m \). Assuming that the value of every term in Equation 1, except for \( m \), is known or can be estimated for each local area/grid, a critical relative saturated depth for a grid \( m^* \) can be determined, where \( m^* = FS^{-1}(1) \) (see Equation 1 and Equation 2).

A physically-based distributed hydrological and erosion-sediment transport model has been developed to determine the dynamic of soil moisture, runoff hydrograph and sedimentation graph. Debris flow as a dynamic spatial of water, sediment, and rock movements which is influenced by characteristics of soil moisture condition, slope factor, and root factor is incorporated and combined with the physically-based distributed runoff and erosion-sediment transport model. This model was used as tool to predict \( m^* \) at any locations inside study site.

![Conceptual Model](image)

**Fig. 3** Methodology of dynamic potential landslide occurrence prediction.

\[
m^* = \left\{ \frac{1}{\gamma_s \beta \tan \phi} \cdot \frac{2C_s}{\gamma_s dsin(2\beta)} \cdot \frac{q_o}{\gamma_s d} - \gamma_{sat} + \frac{q_o \tan \phi - \gamma_{sat} \tan \phi}{\tan \beta} \right\} (2)
\]

As further stage, the failure condition for each grid square can be written in terms of the time-varying relative saturated depth: for \( m^* < m^* \), the slope is safe; and for \( m^* > m^* \), the slope is unsafe. Figure 4 shows map of the critical relative saturated depth by considering the effect of uncertainty in model parameters.

![Map of critical relative saturated depth](image)

**Fig. 4** Map of critical relative saturated depth based on (a) upper limit (b) and lower limit of model parameter values at the Cisangkuy Sub-catchment, Upper Citarum river basin.


**Conceptual Model**

As first stage of the model development, physically-based distributed sediment runoff model has been developed to determine hydrologic and sediment yield components generated from any temporally-spatially varied rainfall event or continuous rainfall data input (Apip et al, 2008). The modeling approach is deterministic, physically-based, empirical, spatially distributed and dynamical in time. Dynamic spatial of water movements, erosion patterns and sediment rates can be predicted at any location inside the catchment as well. The concept of physically-based distributed sediment runoff modeling is shown in Figure 5. A sediment transport algorithm is newly added to the rainfall...
runoff model. Sediment runoff simulation can be divided in two parallel phases: runoff generation and soil detachment.

Fig. 5 Schematic diagram of the physically-based distributed sediment runoff model within grid-cell scale.

Splash and flow erosion models as well as sediment transport models are incorporated to the distributed rainfall-runoff model (Morgan et al, 1998). It includes multiple sources of soil erosion, namely soil detachment by raindrop (DR) and hydraulic detachment or deposition driven by overland flow (DF). Soil detachment processes at interrill and rill are implicitly simulated as raindrop splash and flow detachment respectively. The erosion and deposition rates are calculated as a function of the hydraulic properties of the flow, the physical properties of the soil and the surface characteristic. The detachment by raindrop DR is a function of the energy imparted to the soil surface by the individual drops. The basic assumption of this model is that the sediment is transported and yielded when overland flow occurs. The transport capacity of overland flow also has to be specified to simulate sediment transport processes.

Fig. 6 Hydrologic-shallow landslide conceptual model.

Integrating this model with the shallow landslide information (spatial water, sediment, and rock movements) will allows to predict the sediment yield, landslide development map, and real time prediction (see Figure 6).

7. Real Time Early Warning System

The general procedures and activities required for the flood / debris flow forecasting and warning operations can be seen in Figure 7. All activities should be undertaken promptly utilizing the most up-to-date and reliable data and information. The Citarum River Flood/Debris Flow Forecasting and Warning System (FDFWS) should comprise a computerized database system for the storage/retreival of hydro-meteorological data, the computerized debris flow forecasting procedure and ground movement data. The latter will be linked to the database storage system only to the extent that it will access data in continuous real-time which is being transferred from the established hydro-meteorological network of rainfall, water level monitoring sites and ground movement monitoring sites.

Fig. 7 The activities required for the flood / debris forecasting and warning

Conclusions

1. Debris flow and landslides always occur every year in upper Citarum River Basin. The upper Citarum River basin has high susceptibility of debris flow-landslides.
2. Landslides occurred in the areas which have high to medium of susceptibility and triggering by heavy rainfall above 70 mm with rainfall duration of 5 hours.
3. Map of susceptibility and rainfall thresholds have to be modified regularly if there are any significant changes in catchment or climate characteristics.
4. The distributed model which incorporate flow and sediment yields movement, soil hydraulic properties, root factor, and slope stability is prepared for predicting debris flow and landslides.
5. A continuous early warning system is required to be developed in upper Citarum river basin.

References

Forecasting Landslides and the Associated Risk to the Population of Italy

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Abstract. Italy has a long history of landslides and of related catastrophes. In Italy, landslides occur every year in response to meteorological and geophysical triggers, causing extensive economic damage and casualties. The Italian Department of Civil Protection, an Office of the Prime Minister, and IRPI, a research institute of the Italian National Research Council, are designing a prototype system for the quasi-real-time forecasting of rainfall induced landslides in Italy. The system is based on two components: (i) a set of empirical rainfall thresholds for the possible occurrence of landslides, and (ii) a synoptic (small scale) assessments of landslide hazard and risk to the population in Italy. The two system components will be combined to form a national landslide warning system. To determine the geographical distribution of landslide hazards and risk in Italy, existing catalogues of historical damaging landslide events and of landslide events with human consequences are used, together with synoptic thematic and environmental data. Existing rainfall thresholds for the possible occurrence of landslides are validated, and new rainfall thresholds are determined using catalogues of rainfall events that have or have not resulted in landslides, and detailed records of rainfall measurements. When established, the system will exploit real-time rainfall measurements from a dense network of rain gauges, quantitative rainfall estimates obtained from a network of weather radars, and quantitative rainfall forecasts obtained from advanced numerical weather forecasts.

Keywords. Landslide, hazard, risk, rainfall threshold, civil protection, Italy.

1. Introduction

In Italy, landslides are widespread and recurrent phenomena. Historical investigations have revealed that, in the 1148-year period between 860 and 2007, 1,474 single or multiple landslides have caused at least 13,534 deaths and 2,735 injured people, in 10% (829) of the 8,102 Italian municipalities. In the period from 1950 to 2007, landslide mortality was higher than the mortality caused by any other natural hazard, including earthquakes, floods and volcanic activity (Guzzetti 2000; Salvati et al. 2003; Guzzetti et al. 2005; Glade et al. 2005). Natural landslides in Italy are caused primarily by meteorological triggers, chiefly intense or prolonged rainfall and subordinately rapid snow melt, and by geophysical triggers, including earthquakes and volcanic activity.

The Italian Department of Civil Protection (DPC), an Office of the Prime Minister, has the responsibility to protect individuals and communities from natural and technological hazards, including landslides. To accomplish this challenging task, the DPC performs multiple actions, including monitoring water discharge and precipitation from a dense network of measuring stations, and issuing daily meteorological warnings based on numerical weather forecasts. The DPC is also involved in the determination of landslide hazard and risk at different geographical scales.

In 2007, the DPC asked IRPI, a research institute of the Italian National Research Council, to design a prototype system for the quasi-real-time forecasting of rainfall induced landslides in Italy. The system is based on two main components: (i) a set of empirical rainfall thresholds for the possible occurrence of landslides, and (ii) a synoptic zoning of landslide hazard and risk in Italy, based on historical landslide information and small scale environmental data.

2. Rainfall thresholds

Determining the amount of precipitation that can result in landslides is a challenging task of scientific and societal interest (Guzzetti et al. 2007). The problem is complicated by the fact that the pattern and intensity of rainfall varies with time, driven by natural and human induced environmental variations and changes in climate. For rainfall-induced landslides, multiple investigators have attempted to establish thresholds for the possible initiation of failures. Thresholds may define the rainfall, soil moisture, or hydrological conditions that, when reached or exceeded, are likely to trigger landslides (Reichenbach et al. 1998a; Guzzetti et al. 2007).

Review of the literature reveals that rainfall thresholds for the possible initiation of landslides can be physically-based or empirical (Crosta and Frattini 2001; Aleotti 2004; Wieczorek and Glade 2005; Guzzetti et al. 2007, 2008). Physically-based thresholds are models linking – through infiltration – rainfall pattern and history to slope stability/instability conditions. Empirical thresholds are obtained statistically, studying past rainfall events that have or have not resulted in landslides. Empirical thresholds can be classified based on (Guzzetti et al. 2007): (i) the extent of the geographical area for which they were defined, and (ii) the type of rainfall measurement used to establish the thresholds. Based on their geographical extent, rainfall thresholds can be loosely subdivided as global, regional, or local. The rainfall measurements most commonly used to determine the empirical thresholds include rainfall duration (D), rainfall intensity (I), and the total event rainfall (Guzzetti et al. 2007).

Guzzetti et al. (2007, 2008) have compiled a world-wide catalogue of empirical rainfall thresholds for the possible initiation of landslides. The catalogue lists more than 190 thresholds, of different types, including intensity-duration (ID) thresholds (http://rainfallthresholds.irpi.cnr.it). Fig. 1 portrays 25 rainfall ID thresholds for the possible initiation of landslides proposed for Italy, or for selected areas in Italy. These empirical thresholds can be used to forecast the possible occurrence of landslides, based on rainfall measurements, estimates, and quantitative forecasts.
To design and implement the prototype system for the quasi-real-time forecasting of rainfall induced landslides in Italy, existing empirical thresholds will be validated (Fig. 1), and new thresholds will be defined. The new rainfall thresholds, chiefly of the intensity-duration (ID) and normalized-ID types (Guzzetti et al. 2007, 2008), will be established using innovative and objective statistical techniques, an historical catalogue of rainfall events that have or have not resulted in landslides, and detailed records of rainfall measurements. When established, the prototype system will exploit: (i) rainfall measurements obtained from a dense and homogeneous network of rain gauges comprising more than 1500 measuring stations, (ii) quantitative rainfall estimates obtained from a network of ground-based weather radars installed or networked by the DPC, and (iii) quantitative rainfall estimates obtained from meteorological satellites and advanced numerical weather forecasts.

3. Landslide hazards and risk in Italy

Risk analysis aims to determine the probability that a specific hazard (e.g., a landslide) will cause harm, and it investigates the relationships between the frequency of the damaging events and the intensity of their consequences. Determining landslide risk for an entire nation is a difficult task, chiefly because of the complexity and variability of the landslide phenomena, and the lack of relevant and accurate, spatially distributed, thematic and environmental data. In Italy, information exists on historical damaging landslide events (Guzzetti et al. 1994; Cardinali et al. 1998; Reichenbach et al. 1999b; Guzzetti and Tonelli 2004), and on historical landslide events with human consequences (Guzzetti 2000; Salvati et al. 2003; Guzzetti et al. 2005b,c). This information, and small-scale thematic and environmental data, can be used to attempt the determination of landslide hazards and risk at the national (synoptic) scale.

The map shown in Fig. 2 portrays the geographical distribution of 749 municipalities in Italy that have experienced fatal landslides in the 108-year period from 1900 to 2007. Shades of gray show different numbers of casualties, including deaths, missing persons, and injured people.

To establish the prototype system for the forecasting of rainfall induced landslides in Italy, the existing catalogues of historical damaging landslide events and historical landslide events with human consequences were exploited to determine landslide hazard and risk. For this purpose, a simplified version of a probabilistic model proposed to determine landslide hazard at the catchment scale was adopted (Guzzetti et al. 2005a, 2006). The simplified model ascertains hazard/risk as the joint probability: (i) of the spatial (geographical) probability of landslide events (i.e., “where” landslides are expected), and (ii) of the temporal probability of landslide occurrence (i.e., “when” or “how frequently” landslides are expected). Two separate models were prepared. The first model forecasts the occurrence of all damaging landslide events (landslide hazard model), and the second model predicts landslide events that may result in casualties (landslide risk to the population model). To determine hazard and risk, the municipality (an administrative and political subdivision) was selected as the mapping unit of reference.

To prepare the hazard/risk models, two lists of damaging landslide events and of landslide events with human consequences in Italy were prepared. The two lists were extracted from the existing historical catalogues to cover the 52-year period from 1950 to 2001. For modeling purposes, the individual lists were further split in two sub-sets: (i) a
model training set covering the 41-year period from 1950 to 1990, and (ii) a model validation set covering the 11-year period between 1991 and 2001.

To obtain a quantitative estimate for the temporal probability of landslide occurrence (i.e., “when” or “how frequently” landslide events are expected), the average recurrence of landslide events in each municipality was determined. This was obtained dividing the total number of damaging landslide events (or the total number of landslide events with casualties) in each municipality by the time span of the investigated period (41 year period from 1950 to 1990). Next, the recurrence time of damaging landslide events (or of landslide events with casualties) was assumed constant, and a Poisson probability model was adopted to describe the temporal distribution of the events. Finally, the exceedance probability of having one or more damaging landslide event (or one or more landslide event with casualties) in each municipality was computed for different periods, from 1 to 20 years.

The spatial probability of damaging landslide events and of landslide events with human consequences (i.e., “where” landslide events are expected) was obtained through multivariate analysis of synoptic thematic and environmental information (explanatory variables), including lithological, soil, and climate data, and a set of morphometric variables obtained from a 90 m × 90 m digital elevation model. As the dependent variable, the presence or absence of damaging landslide events (or of landslide events that have resulted in casualties) in each municipality was used. For this purpose, the landslide modeling sets were used, i.e., the lists of damaging landslide events and of landslide events with human consequences in the period from 1950 to 1990. The map shown in Fig. 3 portrays the modeled spatial probability of damaging landslides.

Fig. 3 The map shows spatial probability of landslide occurrence (landslide susceptibility) prepared for damaging landslides, obtained through discriminant analysis. Shades of gray indicate classes of landslide susceptibility.

The temporal and spatial prediction models were tested to evaluate the degree of model fit and the model prediction skills. First, the degree of model fit was ascertained preparing contingency tables, four-fold plots, and ROC curves. Next, the model prediction skills were determined using independent landslide information not used to construct the models, i.e., the landslide validation sets covering the 11-year period between 1991 and 2001. In the prototype system for the forecasting of rainfall induced landslides in Italy, the temporal and spatial models will be used both separately and in combined form.

Conclusions

The Italian Department of Civil Protection (DPC) and IRPI, a research institute of the Italian National Research Council, are designing a prototype system for the near-real-time forecasting of rainfall induced landslides in Italy. The system exploits existing and new rainfall thresholds for the possible occurrence of landslides, and synoptic landslide hazard and risk zonations. The zonations adopt the municipality, an administrative and political subdivision, as the mapping unit of reference, and were obtained through multivariate statistical modeling of historical landslide information, and small-scale thematic and environmental data. When operational, the system will be used by the DPC to issue daily national and regional warnings for the possible occurrence of rainfall induced landslides, based on precipitation measurements, estimates, and numerical weather forecasts. The system will contribute to mitigate – through prevention – the risk posed by rainfall induced landslides in Italy, with emphasis on the reduction of the risk to the population.

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Reichenbach P, Guzzetti F, Cardinali M (1998b) Map of sites historically affected by landslides and floods in Italy, 2nd edition. CNR GNDCI publication n. 1786, Rome, scale 1:1,200,000
A Simple Risk Evaluation Method for Earthquake-induced Landslide Based on Geomorphological and Geological Factors-Case of Mid-Niigata Prefecture Earthquake in 2004, Japan

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Abstract. Based on previous study of landslide occurrence ratio and causative factors for landslides induced by the Mid-Niigata Prefecture earthquake, a simple risk evaluation method for earthquake-induced landslide is proposed. In this study, 727 pre-existing landslide sites and 55 earthquake induced landslides occurred within pre-existing landslide sites were analyzed. According to the analysis of the relationship between causative factors and landslide occurrence ratio, a score ranging 0-2 is determined for the factors, marked 10 in max, and 0 in minimum. And to evaluate the landslide occurring risk, the total scores are marked 3 ranking, Rank1 includes those landslides scored 0-4, Rank2 is 5-7 and Rank3 is 8-10. The landslide occurrence ratio is 4% in Rank1, 8% in Rank2 and 17% on Rank3, respectively. The results have implication to the landslide mitigation measures with the consideration of possible earthquake.

Two earthquake-induced landslides are discussed in the present paper, one is the Toge-shiotani River downstream landslide, and another is the Uragara landslide. These two landslides occurred within pre-existing landslide topographies, under different evaluating scores, may indicate the local conditions influence the landslide initiating.

Keywords. Earthquake-induced landslide, Mid-Niigata Prefecture earthquake, Susceptibility

Introduction
Mid-Niigata Prefecture earthquake occurred on October 23, 2004, with a magnitude of Mw6.6, a depth of 13km, in the Chuetsu region, Niigata Prefecture, northern Japan (Japan Meteorology Agency, 2004). Triggered by the earthquake, a large number of landslides occurred in the Chuetsu region, caused serious damage to the mountainous regions.

Several studies have been conducted to date on the landslides induced by the Mid-Niigata Prefecture earthquake, revealed characteristics such as type classification (Oyagi et al, 2005), geological and geomorphological features (Chigira, 2005; Sekiguchi and Sato, 2006; Yagi et al., 2007), movement characteristics (Moriwaki et al, 2005) and descriptions of individual landslides (e.g. Hasbaator et al., 2006). Hasbaator et al., (2008) had revealed several geomorphological and geological factors, such as longitudinal convexity, largest erosion depth, lower part inclination of landslide slope within geomorphological context, and sandstone, alternation of sandstone and mudstone, and non-reverse dip slopes structure within geological context responsible to landslide occurrence by the Mid-Niigata Prefecture earthquake. The purpose of this study is to propose a simple evaluation method for earthquake-induced landslide, based on the result of Hasbaator et al., (2008). In this paper, we also describe two landslides induced by the Mid-Niigata Prefecture earthquake, the Toge-shiotani River downstream landslide (Toge-shiotani in short) and Uragara landslide, and discuss the differences between them.

Study area and method
The study area was located in Higashiyama hills of central Chuetsu region, Niigata Prefecture, northern Japan (Fig.1). The study area is characterized by low hilly terrain with the elevation of 300-700m above see level. The Imo River, in its basin where the most intensive slope failures occurred by this seismic event, is flowing from north to south through the hilly terrain. The geological characteristics of the study area are broad distribution of Neogene to Quaternary sedimentary rocks and NNE-SSE oriented axes of active folds (Yanagisawa et al., 1986; Kobayashi et al., 1991).

The mainshock hypocenter of the Mid-Niigata Prefecture earthquake occurred in southern Higashiyama hills, and a series of strong aftershocks occurring in Higashiyama hills and Uonuma hills. Most of the slope failures induced by the earthquake were distributed in the Higashiyama hills.

In this study, we focused on the pre-existing landslides and earthquake-induced landslides in the study area (Fig.2), which were interpreted using aerial photographs as well as DEM taken before and after the earthquake. We analyzed 727 pre-existing landslides, 55 earthquake-induced landslides which occurred within the pre-existing landslide...
Based on the results of Hasbaator et al. (2008), several geomorphological and geological factors are related to the landslide occurrence by the earthquake event. Therefore, given these geomorphological and geological factors responsible to occurring of the earthquake-induced landslides, we determined critical values for these factors (Table 1) according to the relationship of landslide occurrence ratio and values of these factors. As a result, 1-3 classifications were determined under the landslide occurrence ratio changing with the values of each factor. We set score of 0-2 for these factors (Table 1), so as to evaluate the landslide occurring risk induced by earthquake. Because some factors showed that there were almost no landslide occurred under a value of the factor, so we determined 0 score for this cases. According to the score and landslide occurrence ratio, we also determined susceptibility ranking of those landslides distributed in the study area.

Result
Based on the critical values and score of the geomorphological and geological factors, we analyzed both pre-existing landslides and earthquake-induced landslides in study area. The relationship of the score distribution and number of landslide, landslide occurrence ratio is shown as Fig. 3. The landslide occurrence ratio increases with the total scores, except there is uneveness in score from 4 to 6, that ratio is up to 30% at total score of 10. The ratio has an unclear tendency from score from 4 to 6.

We set 3 susceptibility ranks to evaluate the landslides, according to the landslide occurrence ratio and total scores. Here, Rank 1 is corresponding to score 0-4, Rank 2 is 5-7 scores, and Rank 3 is 8-10 scores, respectively (Fig. 3).

Susceptibility ranking and landslide occurrence ratio is shown as Fig. 4. Total of 11 landslides occurred in 275 pre-existing landslide sites in the Rank 1; the ratio is approximately 4%. In Rank 2, 28 landslides occurred in 356 pre-existing landslide sites, the ratio is approximately 8%, and the landslide occurrence ratio is up to 30% in Rank 3, 16 landslides occurred in 96 pre-existing landslide sites in this rank. As shown in Fig. 4, the landslide occurrence ratio of susceptibility ranks increasing from 4% at Rank 1 to 8% at Rank 2, and up to 17% at Rank 3.

As a result, the Rank 3 landslide sites are mostly concentrated in Imo River basin, showed the tendency of landslide occurring by this earthquake.

### Table 1: Critical value and score of causative factors

<table>
<thead>
<tr>
<th>Factors</th>
<th>Score</th>
<th>2</th>
<th>1</th>
<th>0</th>
</tr>
</thead>
<tbody>
<tr>
<td>G.C.</td>
<td>Sandy mudstone, alt. Ss and MS</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>G.S.</td>
<td>non-reverse dip slope</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>L.C.</td>
<td>exceeding 0.8</td>
<td>0.6~0.8</td>
<td>under 0.6</td>
<td></td>
</tr>
<tr>
<td>L.E.D.</td>
<td>exceeding 90m</td>
<td>50~90m</td>
<td>total score =0 if under 50m</td>
<td></td>
</tr>
<tr>
<td>L.P.I.</td>
<td>exceeding 30 degrees</td>
<td>10~30 degrees</td>
<td>total score =0 if under 10 degrees</td>
<td></td>
</tr>
</tbody>
</table>

Two examples of earthquake-induced landslides and discussion

Landslide risk evaluation result shown that the landslide occurrence ratio is 30% at score of 10 (Fig.3), and about 17% at susceptibility Rank3 (Fig.4), the result giving a tendency of landslide occurring induced by earthquake. The result shown that more landslides of susceptibility Rank3 is distributed in the Imo River, may be due to the active folds and sandy sedimentary rocks or alternation of sandstone and mudstone. The Toge-Shiotani landslide is an example of these landslides. However, there are landslides occurred by the earthquake in the study area, even the susceptibility Rank is 1. The Uragara landslide is an example of low rank but initiated by the earthquake.

Table 2 Comparison of the Toge-shiotai river downstream landslide and Uragara landslide

<table>
<thead>
<tr>
<th>Factors</th>
<th>Toge-shiotani river downstream</th>
<th>Uragara</th>
</tr>
</thead>
<tbody>
<tr>
<td>Value</td>
<td>Score</td>
<td>Value</td>
</tr>
<tr>
<td>G.C.</td>
<td>Sandstone</td>
<td>2</td>
</tr>
<tr>
<td>G.S.</td>
<td>Dip-slope</td>
<td>2</td>
</tr>
<tr>
<td>L.C.</td>
<td>0.7</td>
<td>1</td>
</tr>
<tr>
<td>L.E.D.</td>
<td>92</td>
<td>2</td>
</tr>
<tr>
<td>L.P.I.</td>
<td>39.7</td>
<td>2</td>
</tr>
<tr>
<td>total</td>
<td>9</td>
<td></td>
</tr>
</tbody>
</table>

Toge-Shiotani River downstream landslide

This landslide is located on the right bank of a tributary of the Imo River, on an east faced slope. Landslide initiated within the pre-existing landslide slope, with about 220 m long and 120 m wide, moved about 60 m and blocked the river at its toe area (Photo.1, Fig.5). The bedrock of the landslide slope primarily consists of sandstone, with striking NNE-SSW, and dipping to east about 20 degrees, so the slope showing dip-slope structure. According to borehole investigation, the moving soils consist of fractured weathered sandstone, and the bedrock consists of sandstone, and the slip surface was estimated forming in the boundary of weathered sandstone and fresh or weakly weathered sandstone (Yuzawa Sabo, 2007). According to the score determination as described above, the total score of the landslide slope is 9 (see Table2).

Uragara landslide

Uragara landslide is located on a northwest faced slope along the left bank of the Asahi River, a tributary of the Shinano River. The Uragara landslide induced by the Mid-Niigata Prefecture earthquake occurred within pre-existing landslide topography. The landslide is 105 m long and 60 m wide, and the displacement is about 40 m (Photo.2, Fig.6). The landslide slope is composed of debris, weathered mudstone, and the bedrock is composed of mudstone. The bedrock is striking NE-SW, dipping 20 degrees to NW, and the landslide slope showing dip-slope structure. According to the geological investigation (Niigata Prefecture, 2005), the moving soils consist of debris, heavily weathered mudstone, and the bedrock consists of mudstone interbeded with sandstone. The slip surface was estimated to formed in weathered mudstone or at the boundary of weathered mudstone and fresh mudstone.

According to the causative factors of the landslide slope, the score of this landslide is 4 (Table 2), significantly lower than that of the Toge-Shiotani landslide. These two landslides shown different characteristics according to the geomorphological and geological conditions (Table 2), and the Toge-Shiotani landslides was expected to occur while the Uragara was not expected to occur, according to the score. But, as a fact, both the landslides induced by the Mid-Niigata earthquake, even the score of the Uragara

Photo.1 A panorama view of the Toge-shiotani River downstream landslides. Drawn from ortho photos taken immediately after the earthquake.

Photo.2 A panorama view of the Uragara landslides. Drawn from ortho photos taken immediately after the earthquake.
landslide is only 4. This result may be indicating that the landslide occurring affected by local conditions, such as groundwater, material strength in individual landslide sites.

However, we did not mention the earthquake ground motion, an important factor to influence the landslide occurring. We will conduct further analysis including these factors.

According to Hasbaatar et al. (2008), more landslides occur within pre-existing landslide sites, with some measurable geomorphological and geological factors, in the case of Mid-Niigata Prefecture earthquake. Our study gave a simple landslide risk evaluation method according to geomorphological and geological factors. The results have implication to the landslide mitigation measures with the consideration of possible earthquake.

Earthquake-induced landslides have been an important problem in recent years, especially after the 2004 Mid-Niigata Prefecture earthquake, such as Sichuan earthquake 2008, China, and Miyagi-Iwate Inland earthquake 2008, Japan. Therefore, evaluating the landslide risk is an important problem, because it could provide important information to the management of landslide in local level. However, the result is an example of regional landslide susceptibility, but not for occurring prediction for individual landslide.

Conclusions

Based on the critical value of causative factors responsible for the Mid-Niigata Prefecture earthquake, 2004, we proposed a simple landslide risk evaluating method for earthquake-induced landslide.

1. The landslide occurring rate is increasing from score 6, and up to 30% at the score of 10;
2. Landslide susceptibility rank classification shown that Rank1 is approximately 4%, Rank2 is about 8% and the Rank3 is up to 17%.
3. The fact of landslides occurred by the Mid-Niigata Prefecture earthquake shown that this method provided a tendency of landslide occurring, but not for predicts individual landslide occurring by an earthquake.

Acknowledgment

We extend our thanks to the Yuzawa Sabo Works Office of the MLIT, Japan, and the Sabo Branch of Niigata Prefecture, Japan, for providing information regarding landslides induced by the Mid-Niigata Prefecture earthquake.

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The First World Landslide Forum, 2008, Tokyo
Influence of Rainfall in the Behavior of Residual Soils

Carlos José Bacca Bautista (The University of Tokyo) · Taro Uchimura (The University of Tokyo) · Jou Fukuda (The University of Tokyo) · Wang Lin (Chuo kaihatsu corporation)

Abstract. Landslides in residual soils occur frequently in many places around the world after rainfall periods causing many deaths and material losses. Regardless of the efforts that have been made to improve slope safety, many issues related with this kind of phenomena still have not been fully understood. According to the previous experiences it have been observed that many landslides produced in weathered soils slopes have presented shallow slip surfaces above the ground level and under unsaturated conditions.

In situ investigation related to the slope response due to rainfall and water infiltration had indicated that suction is reduced by infiltration of water which can provide a triggering factor to initiate instability of a slope. Although some studies have described the effect of intensive rainfall in residual soils under partially saturated conditions, few have studied about the effect of consecutive repetitive rainfall events in the accumulation of deformations, under constant stresses and its effect in the safety factor. Additionally few researches have been concentrated to study this problem in term of critical deformations under constant shear stress as occur in real cases.

The present research attempts to study the rainfall effect into shallow landslides considering the influence of successive rainfall events into the suction soil parameters and the accumulation of deformations under constant shear stress. To study this phenomena, water content sensor and electronic inclinometers have been placed in a weathered slope at Kobe city Japan in order to monitor these two parameters. According to preliminary field information, it has been identified that after every rainfall event a considerably increment in the water content have been found. In addition, during consecutive rainfall periods changes in the matric suction into the soil have been found. Nevertheless, critical parameters cannot be defined due to the absence of a failure condition in the slope.

In order to understand adequately this phenomenon, until failure, an extensive laboratory program is proposed. For this attempt modified triaxial apparatus will be used to simulate adequately the field conditions. This machine specially modified to measure unsaturated conditions is composed by a ceramic disk, an internal pore pressure sensor and a double cylinder cell to measure volume change in the specimen. To simulate the field stress paths, a constant anisotropic test will be performed while the suction into the sample is changed. During the test axial strains and volumetric strains will be monitored until achieve the failure. Finally taking into account the similarity of the accumulation of deformations under constant stress state, these phenomena will be compared with the creep failure which currently is under research.

Keywords. Shallow landslide, infiltration, monitoring, precursor stage of landslide

1. Background

Most of the traditional slope stability methods frequently use equilibrium methods where the water is only considered under the ground water level under saturated condition and positive pore pressure. Recently, some researches have started to consider the importance to study the change in the saturation ration as a triggering factor in slope stability. Studies about the unsaturated soil conditions show the importance of suction forces in the soil resistance. In the particular case of shallow landslides many field studies have demonstrate the accumulation of water in the surface layer (perched water table) of weathered soils as a triggering factor of shallow landslides (Dai Fuchu et al.1999;Nishigaki and Tohari.2000, A.g.Li.2005). Therefore, in this surface layer the saturation ratio is constantly changing due to wetting and drying cycles presenting different values of suction into the soil. Studies under unsaturated condition have demonstrated that the decreasing of suction into the soil can produced failure under unsaturated conditions (Yoshida et all. 1991). Although these studies have been done, the influence of repetitive rainfall events has not been totally studied. In this way the accumulation of displacements under constant applied shear stress after many wetting to drying cycles have not been considered has a critical factor.

However, some recently researches about creep in soil have been studying the effect of accumulation of displacement has a triggering factors into the soils (Tatsuoka et all. 2008). Although, these both phenomena have been studied separately, this new creep concept could be applied to study the shallow landslides as combined way.

2. Site description

To study the behavior of the shallow layer in residual soils a hillside in Kobe was selected (Figure 1).This zone was chosen specially for three reasons. First, the presence of a typical weathered granite soil, second, for the evidence of shallow landslides in the zone and third for the existence of additional instrumentation placed into the area.

The selected slope site is located in the Senjon-Tami Valley close to a natural stream course (Figure 1). The place is cover by typical natural vegetation and trees. The average slope angle is about 40°.As is shown in the Figure 2 some previous shallow landslides as occurred in this place.
3. Soil description

According to the in situ visual characterization, the shallow material corresponds to a discomposed granite.

The particle-size distribution of the material is shown in the Figure 3. The bulk density, the specific gravity and the void ratio are shown in the Table 1.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity</td>
<td>2.54</td>
</tr>
<tr>
<td>Bulk density (g/cm³)</td>
<td>1.86</td>
</tr>
<tr>
<td>Void ratio</td>
<td>0.76</td>
</tr>
</tbody>
</table>

Table 1. Soil properties

According to the regional geology this place is composed by plutonic rock specially granite type. In order to characterize the shallow material, some samples were taken. The residual soil from the granite weathering is basically identifying as light brown, silty clay sand. The Figure 4 shows a digital photograph taken from the electronic microscopes using and amplification lent of 175X. This photograph shows the general microstructure of the shallow material.

5. Instrumentation system

In order to monitor the behavior of the slope, water content sensor and electronic inclinometers were placed. The distribution of instrumentation within slope is shown in the Figure 5. The places chosen were located around the previous shallow landslides with the purpose of study the behavior of undisturbed material. Figure 5 show the chosen instrumentation points which are named with letters A, B, C, D. For every instrumented point, water content sensors and electronic inclinometers were installed.

The water sensors were pushed inside the soil at depth of 30 cm after open a small hole into the soil. On the other hand, the inclinometers sensors were located inside the internal collector unit which was previously tied to a metal bar sank into the soil at 40 cm of depth (Figure 7). The electronic inclinometer used in this study has the possibility to measure the angle of rotation for three axes, between the range of -30° and 30°. In the present instrumentation, the X axis corresponded to the inclination on the slope direction. It is important to underline that the negative values measured for this device in the X axis will correspond to the inclination at the forward slope direction. The schematic diagram about the instrumentation is shown in the Figure 6. To transfer the information between the measuring point and the central base, each sensor unit was connected by wireless communication system to a laptop located in the bottom part of the slope. Into this computer a cell phone antenna was placed with the aim of transfer the information to the central collector located at the University of Tokyo. This wireless system was developed by...
the University of Tokyo as a research project with the goal to obtain easily real-time data in instrumented slopes. The general procedure is shown in the Figure 7.

Fig 6. General installation of water sensors and inclinometers.

![Image of water sensors and inclinometers]

Fig 7. Instrumentation procedure at Senjon Tami slope

6. Partially field monitoring result

The information contained in this paper corresponds to the period from April 17 to June 22, 2008. The data was taken every 10 minutes during the elapsed time period. Figure 8 shows the variation of water content for every instrumented point. By examining the information about the water content into the soil (Figure 8), the following observations are made:

- It is evident the response of the water content into the soil after every rainfall event for all the instrumented points.
- The increment of water content for every point differs for different points. The maximum increment was identified for points B and D, where the average increment was approximately 40%. Nevertheless, in the other places the average increment was about 20%. These differences could be attributed to changes in the landform which could allow the accumulation of water.
- After every rainfall event a decrement of the water content is observed during the dry period. It is interesting to highlight a general tendency in this recuperation curve. The amount of recuperated water content will depend on the duration of the dry period.
- During the period between May 12 and June 5 2008 five successive rainfall events were registered. In this period an accumulation in the water content into the soil is observed.

On the other hand, Figures 9 shows the inclination variation in the slope and its relation to the water content for a typical case (Point B). In this information, negative values correspond to the inclination in the forward slope direction. Respect to the results is important underline the evident sensitivity of inclinometers, represented in the consecutive fluctuation in the inclinations values. However, the general tendency in the inclination values was taking into account to the analysis. To highlight this tendency, a wide line was drawn over the data. Additionally, to analyze the behavior after successive rainfall events, shadow rectangles on the graphs were placed.

![Image of inclination variation]

Fig. 8. Variations of the volumetric content into the soil at Senjon Tami slope from April 17 to June 23, 2008.

Fig. 9. Variation of the inclination in the slope and its relation to the water content.

By analyzing the inclination information the following observation are made:

- A small increment in the inclination value is identifying during every rainfall events.
- Despite of the sensibility of the sensors and the small
inclinations measured, accumulations of displacements during the rainfall period for all the instrumented points were found. This increment is higher in the period between May 12 and June 5 2008, where five consecutive rainfall events occurred.

- During the instrumentation period, small deformations were observed. This means that no limit values were reached.

6. Future studies

In order to observe the soil behavior during the saturation process and identify the critical values for shallow landslides, a laboratory triaxial test is projected. For this propose a modified triaxial machine for unsaturated condition is suggested in order to adequately simulate the conditions in field. This machine essentially is composed by a ceramic disk, an internal pore pressure sensor and a double cylinder cell specially to measure unsaturated condition and volume change into the specimen. To simulate the field stress paths, a constant anisotropic test will be performed while the suction into the sample is changed. The mean propose of this test will be to analyze the behavior of the soil in terms of deformation after a successive changes in the soil suction and define a appropriated failure criterion in terms of deformations.

Conclusions

The partial results of this investigation show that:

1. The field data shows a relationship between the water content and development of deformations into the soil.
2. After consecutive rainfall events an evident accumulation of deformations is observed in the soil mass.
3. During the present instrumentation period, small changes in the inclination for every control point have occurred. However, it is important to study the behavior of the type of soil under different wetting and drying condition in order to identify the shallow mechanics of failure, common in residual soils.
4. A laboratory investigation is proposed in order to analyze the behavior of residual soils under different wetting and drying cycles maintaining a constant shear stress as occurred in real slope conditions. In addition, the accumulation of displacement in the soil material will be taking into account in order to define critical displacements in monitoring systems.

References


Landslide Incidence in the Limpopo Province, South Africa

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ABSTRACT: Landslides are a serious geohazard in the Limpopo Province of South Africa, as many densely populated rural communities live in close proximity to areas of steep relief, which are susceptible to slope instability. The region has experienced both historical and recent landsliding. This paper presents the findings of landslide inventorisation activities conducted in the province over the past year, and also explores a case study, i.e. a distinctive ancient landslide which formed Lake Fundudzi. This study has facilitated the production of the first landslide inventory map for the Limpopo Province. The map will display landslide occurrence and distribution throughout the province, and has the potential to mitigate development problems and promote a safer living environment, thus impacting positively on the local communities.

Keywords: discontinuities, Lake Fundudzi, landslide, Limpopo Province, rainfall.

1. INTRODUCTION
The Limpopo Province is the northern-most province in South Africa and borders Mozambique, Zimbabwe and Botswana (Fig. 1). The Council for Geoscience (CGS) National Geohazard programme identified the Limpopo Province for landslide inventorisation, as landslides are a major geohazard in the province and rural communities are becoming increasingly vulnerable.

Many countries around the world have realized the enormous impacts of these geohazards and have developed programmes for landslide inventory and susceptibility mapping. A Landslide Susceptibility map for Southern Africa was compiled in 1985 by Paige-Green, based on climate, geology, geomorphology and past landslides associated with roads and railways. The map was later revised, in 2004, by Paige-Green and Croukamp. The CGS has undertaken to map each province in more detail. This project is the second of its kind, following a similar project in the KwaZulu-Natal Province.

This paper presents the findings of the investigations conducted in the province over the past year, and assesses the ancient landslide case study of Lake Fundudzi. The primary objectives of this study were the following: 1) to add to the current body of knowledge of catastrophic landslides in South Africa; 2) to produce a landslide inventory map for the province; and 3) to infer the probable cause and failure mechanism of the Fundudzi landslide, based on the available data.

Landslides have a negative impact on human as well as natural environments and on socio-economic development. Therefore the “identification of landslide hazard areas is important for site selection and development planning of housing and infrastructure facilities within the landslide prone area” (Iwao et al., 2002). This study has facilitated the production of the first landslide inventory map for the Limpopo Province, displaying landslide occurrence and distribution throughout the province and has the potential to mitigate development problems and promote safer living environments.

Figure 1: Limpopo Province and South Africa locality map.

The inventory process for this project has now been completed and forms the basis of this paper. A distinctive landslide lake, examined during the inventory process, is also presented here as a case study. The published literature on Lake Fundudzi is limited. However, from available information it was inferred that the process of slope toe erosion by the Mutale River (van der Waal, 1997) and the presence of major structural discontinuities (faults and joints) were important controlling factors for tension crack initiation at the crest of the slope and the consequent mass-wasting event.

2. PHYSIOGRAPHY
The geology of the Limpopo Province varies from Palaeo-Archaean mafic, ultramafic and felsic extrusives to Mesozoic sedimentary rocks and flood basalts (1:1000 000 scale RSA Geological Map series, 1984). The topography varies from relatively flat areas to mountainous terrain. Limpopo is an area of mixed grassland and trees, generally known as bushveld. Three distinct mountain ranges were the focus of this project; these being clearly identifiable from the slope class map (Fig. 3). These ranges are the (i) south-eastern Drakensberg and Strydpoortberg mountains, which consist predominantly of Archaean granite and gneiss; (ii) the northern Soutpansberg and Blouberg
mountains, which are highly block-faulted rocks, consisting of a volcano-sedimentary sequence of mainly basaltic lavas and quartzites, tilted to the north-northwest at an average of 20°. Lastly, (iii) the western Waterberg mountain range, which consists predominantly of sandstone with minor basic lavas at the base of the Waterberg Group (Brink, 1981; Barker et al., 2006). The province has an extensive network of roads, creating a large number of road cuttings. Many cuts are poorly designed, resulting in numerous slope instabilities. The climatic conditions of the study area are characterised by summer rainfall, with an average annual precipitation of 620 mm. The northern and eastern areas are subtropical, with hot and humid summers (SA Weather Service, 2008). Mist is frequent in the mountain regions, and winters are mild and mostly frost free.

3. METHODOLOGY
The methodology implemented consisted of a desk study, encompassing a literature review; stereoscopic examination and interpretation of panchromatic aerial photo pairs and digitally enhanced aerial images. The colour 3D space imagery of Google Earth 2008 were also reviewed. Questionnaires were created and sent out to communities in the province, responses indicated areas that had been affected by recent landslides.

The desk study was followed by intensive field work in the region in order to investigate suspected landslide areas identified during the desk study. Field work included ground as well as aerial inspections using a light fixed-wing aircraft. All the acquired information was used to compile the preliminary provincial landslide inventory map, using ArcGIS software (Fig. 3).

A geomechanical survey was also conducted on the host rock walls of the Lake Fundudzi landslide and included a discontinuity survey as well as Point Load and Schmidt hammer tests, following ISRM suggested methods (1981) and ASTM standards (2000).

4. RESULTS
4.1 Landslide inventory and susceptibility mapping
The inventorisation process identified over 700 recent and ancient landslide events. These events were classified according to the abbreviated landslide classification scheme of Cruden and Varnes (1996), and are summarised in Table 1. Five types of movement were identified, namely: falls, topples, slides, flows and undifferentiated landslides.

These events vary in size, with the majority being rockfalls. Most of the landslides in Limpopo Province are a result of one or more factors, i.e. steep slopes, discontinuities, high rainfall, weathering and/or human intervention. These conditions increase shear stresses and decrease the shear strength of the materials.

Many slope instability events tend to occur along road cuttings in the province: poorly constructed mitigation measures have done little to prevent these events (Fig. 2). Eye-witness accounts obtained from locals during field investigations in the province, indicate that the majority of recent landslide events had occurred in February 2000 after particularly heavy and intensive rainfall. The Limpopo Province experienced its worst floods in living memory in February 2000 owing to cyclone Eline. As a result, flooding and rainfall induced landslides on man made as well as natural slopes caused loss of life (101), damage to houses, infrastructure ($166 million) and livestock losses (Limpopo Provincial Disaster Management Unit, 2000).

<table>
<thead>
<tr>
<th>Type of movement</th>
<th>Number of landslides</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fall</td>
<td>570</td>
</tr>
<tr>
<td>Topple</td>
<td>4</td>
</tr>
<tr>
<td>Slide</td>
<td>53</td>
</tr>
<tr>
<td>Flow</td>
<td>60</td>
</tr>
<tr>
<td>Undifferentiated</td>
<td>24</td>
</tr>
</tbody>
</table>

The first landslide inventory map of the Limpopo Province has been compiled (Fig. 3). The location and distribution of events were mainly located in the steeper mountainous areas of the province; these being areas where slope angle is greater than 12 degrees. These landslides fall within areas defined by Paige-Green (1985) as areas where “instability may occur”.

4.2 Case Study: Lake Fundudzi landslide lake
The Lake Fundudzi landslide is a large translational rock slide or rock avalanche situated in the Mutale River valley of the Soutpansberg mountains. The Lake Fundudzi landslide occurred tens of thousands of years ago and blocked the course of the eastward flowing Mutale River (Trevor, 1926; Janisch 1931; van der Waal, 1997). According to Costa and Schuster (1988) Lake Fundudzi is classified as a type 2 landslide dam in terms of material deposits on the valley floor. The slide occurred in the northward dipping quartzite of the Fundudzi Formation of the Soutpansberg Group.

The crest of the landslide scarp is approximately 450 m in length, 220 m above the lake, and has a slope failure plane surface area of approximately 17 ha (Fig. 4).
The calculated volume of rock that slid down the failure surface is estimated at 10 to 15 million m$^3$, while rock blocks travelled a distance of up to 700 m across the valley floor.

The largest blocks have a diameter between 7 to 10 m. The failure slope dips approximately 60º to the south into the valley, while bedding planes dip an average of 20º to the north (Janisch, 1931).

Barker (1979) mapped two NW-SE and SW-NE intersecting faults which form the failure lines of each flanking buttress. Several minor rock slides and falls are still active and noticeable on the failure side of each buttress, and were also noted by Janisch (1931). Tension cracks were also observed at the crest of the failure slope.

The results for Point load tests on the Fundudzi Formation quartzite reveal UCS ranges between 26 MPa and 118 MPa, which classify as moderately strong to strong according to ISRM (1981). Schmidt hammer results reveal that the approximate uniaxial compressive strength is 60 MPa, which classifies as strong, according to Miller (1965). The rock is well jointed and the discontinuity survey identified three major joint sets. The sets have been numbered as J1 to J3 and are orientated SW-NE, WNW-ESE and N-S; respectively. The main failure surface of the Fundudzi landslide best correlates with J2.

5. DISCUSSION

The landslide inventory for the Limpopo Province has identified hundreds of landslide events, these have mainly occurred in the steeper mountainous terrain, often as a result of sustained heavy rainfall. Some of these events, especially those of February 2000, have resulted in loss of life and damage to the natural and human environment. The project has the potential to assist the Department of Works, Disaster Management and Roads and Transport in Limpopo Province with mitigation measures, given that slope
instabilities are evident along many national, provincial and
district roads.

With regard to the Lake Fundudzi landslide, pre-existing
conditions such as the joints, fractures and faults made the
rock mass vulnerable to instability. The stream erosion by
the Mutale River led to oversteepening of the slope (this in
agreement with van der Waal, 1997), thus reducing its
overall stability, and accelerating tension crack formation at
the crest of the slope resulting in the failure. The
Soutpansberg Mountains also experience high rainfall and
as rainfall-induced slides are common in the province, this
was also a probable trigger to failure. Available historical
records of seismic data indicate a low level of seismic
activity for the region, but may also have acted as an
additional trigger at the time.

6. CONCLUSION

The study has facilitated the production of a first
landslide inventory map for the Limpopo Province. The
study has also shown that the rainfall-induced landslides of
February 2000 had a major negative impact on rural
communities. It is concluded that future landslides, initiated
by extreme rainfall events, will continue to take a high toll
if further research, proper management and mitigation
measures are not implemented. The Inventory map will
facilitate the compilation of the first landslide susceptibility
map for the province, which is still to be produced. This
map has the potential to mitigate development problems
and promote safer living environments.

Proper design of cuttings and slope protection methods
should be followed when constructing roads and railways,
as most existing failures seen along roadways are caused by
improper slope design.

7. ACKNOWLEDGMENTS

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and CGS for funding and supporting this project.

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Characteristics of Landslides and Landslide Prediction Maps by a Probabilistic Prediction Method in Korea

Yong-chan Cho (KIGAM, Korea)

Abstract. This study analyzed characteristics of landslides induced by intensive rainfall during the monsoon season in Korea. The study also suggested a method of landslide prediction mapping using a probabilistic prediction method of landslides. Based on the method, this study constructed regional and local prediction maps of landslides in Gyeonggi province and Gangwon province in Korea. The maps show locations of high landslide potential and their probabilities.

Keywords. Landslide predicting map, probabilistic prediction,

1. Introduction
The landslides induced large losses of lives and much damage of private and public properties. According to a statistical data issued by NEMA in 2006, there have been average annual human deaths as many as 47.2 people induced by landslides in Korea since 1976. It is 22.3 percent of total human deaths by all kinds of natural hazards occurred in Korea. Moreover, the percentage of human death by landslides to that of all natural hazards in 2006 reaches up to 36%. Considered with these data, landslides are severe natural hazards in Korea, especially during summer season. Therefore, there have been increasing societal needs to reduce landslides hazards and damages in the community. Therefore, it is necessary to determine landslide potential areas and to develop landslide prediction techniques and damage mitigation techniques. In order to meet these needs, the following studies were performed; 1) examination and systematization of influential factors on landslides; 2) computation of weighting values of the factors; 3) establishment of hazard index; and 4) production of landslide probability map.

The study area are the Gyeonggi province and Gangwon province in the Korea, where many severe landslides occurred after heavy rainfall. More than 2,600 landslides were surveyed.

2. Types of landslides
Eight nine percents of the landslides in the study area were grouped into the translational flow. Most of these slides were initiated as translational flows at the head parts and were changed into debris flows toward the center and toe of the slides. Translational flow showed a type of non-circular failure which the sliding plane formed a planar shear plane. The landslides in this area are also mainly caused by intense rainfall that suddenly increases pore pressure and abruptly reduces the shear strength of the soil (Canine 1980).

The debris flows usually took place along V-shaped valleys on mountain slopes. Translational slides initiated on relatively planar mountain and gradually changed into debris flows composed of a mixture of coarse rock fragments and fine soil material.

In case of the landslides in the study area, 75 % of the landslides are less than 60m in length, among which about half are less than 30m. More than 80% of the slides are narrower than 10m in width. All the landslides cut below the 1 m thick surface material. On the other hand, the maximum size of landslides is more than 250 m long in northern Gyeonggi area. These features are due to the steeper slopes in the northern Gyeonggi area. Although most of the landslides are quite small in size, they destroyed a lot of property and human lives suddenly and unexpectedly. The reason why there have been much damage by small landslides in Korea is that many residential areas are located near the bottom of mountains or along mountain valleys (Kim and Chae 1998).

3. Probabilistic prediction of landslide
Many studies have been done to determine the factors inducing landslides and to predict the susceptibility of landslide occurrence. However, most of these studies determine the susceptibility using subjective and qualitative methods. In particular, many landslide susceptibility maps were made with a relative index of susceptibility, not with a quantitative one. However, this study tried to determine the influential factors of landslides and to develop a quantitative probability index of landslide occurrence using statistical methods.

Chae et al.(2004) suggested probabilistic prediction model of landslide. But that model contains two categorical factors, especially grain size distribution(USCS) is so difficult to calculate the probability. This study used a modified logistic regression model for a probabilistic prediction of debris flow on natural terrain(Eq. 1; Cho et al., 2007). The modified model dose not contain grain size distribution factor that were used in the previous model and secured higher reliability of prediction than that of the previous one. The modified model is composed of lithology, two factors of geomorphology, and three factors of soil property. Verification result shows that the prediction reliability is more than 90%.

\[
\text{Logit}(p) = -9.3670 + 0.2129 \cdot \text{Angle} + 0.7690 \cdot \text{DryDensity} - 0.0052 \cdot \text{Elevation} + 0.4248 \cdot \text{Lithology} + 0.1777 \cdot \text{Permeability} + 0.555 \cdot \text{Porosity}
\]

(1)

The regression coefficients of logistic regression can be regarded as the relative weighting values of each factor. Substitution of logit value into Equation 2 calculates the probability of landslide occurrence at a position.
\[ P_s = \frac{1}{1 + \exp\{-\logit(p)\}} \quad \text{or} \quad P = \frac{\exp\{\logit(p)\}}{1 + \exp\{\logit(p)\}} \] (2)

The importance of the above method is that quantitative weighting values are assigned to the influential factors.

4. Landslide probability map

The probabilistic map of landslides was made using GIS tools on the basis of the statistical analysis. The distinctive point of the maps is that they were composed of quantitative probabilistic index of landslide occurrence. The index was calculated using a very scientific and reliable method, not by a subjective and qualitative one. The probability of landslides makes it easy to understand the meaning of the hazard index.

For the construction of landslide prediction maps, detailed field survey was performed over 1,500Km² area in Gyeonggi Province and 1,574Km² area in Gangwon Province. Based on the field survey data and laboratory test results for geologic properties of soil, 30 sheets of landslide prediction maps were drawn up in Gyeonggi Province in both digital maps and printed maps. Among 30 sheets, 10 sheets were drawn up in the scale of 1:25,000 in regional areas (Fig. 1). 20 sheets were drawn up in the scale of 1:5,000 to predict detailed probability of landslides in the vicinity of large cities (Fig. 2). For Gangwon Province, 10 sheets were published in the scale of 1:25,000.

Verification of the applicability of the suggested method shows that it has more than 90% of the correctness. For further studies, the method needs to be modified to include more various geologic conditions, vegetation as well as rainfall effect. Moreover, a representative logit equation to cover Korean peninsula has to be developed.

Conclusions

Most of the landslides are induced by intensive rainfall during the rainy season in Korea. The landslides are typical translational slides at the triggering position, and then, changed into debris flows as they move down slope. The topography at the triggering position strongly controls the frequency and magnitude of landslide on the basis of the downslope change in landslide type.

The influential factors of landslides were selected using the logistic regression analysis. The factors can be grouped as geomorphologic and soil properties. The geomorphologic factors include elevation and slope angle, whilst the soil properties are dry density, porosity, lithology and permeability. The method assigns quantitative weighting values to the influential factors, which makes it possible to set up a quantitatively accurate index of landslide probability.

The probabilistic prediction map of landslides in the study area was made using statistical methods...
and GIS tools. The distinctive mark of the maps is that they are based on a quantitative probability index of landslides calculated using a scientific and reliable method, not by a subjective and qualitative one. The hazard index makes it easy to understand the meaning of the landslide susceptibility.

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Mitigation of Geohazards on the Onshore Pipeline Route through the Makarov Mountains, Sakhalin II Project

Konstantin Samarkin Dzharsky (Inzhzaschita LLC, Sochi City, Russia), Neil O'Donnell (Sakhalin Energy Investment Company Ltd)

Abstract. Sakhalin II project is the largest oil and gas project currently being carried out within the territory of the Russian Federation. The project Client is “Sakhalin Energy Investment Company”, and Phase 2 of the project involves construction of onshore oil and gas pipelines over a 782km long corridor, running from the north end of the island to the LNG plant and export terminal at the south end of the island. “Starstroi” LLC (Krasnodar City) was appointed as the EPC Contractor for the onshore pipeline construction. This paper explains the design philosophy applied in mitigating risk from landslides to the pipeline route through the Makarov mountains section of the route.

Keywords: risk evaluation, landslide monitoring, planning procedures for engineering protection, mitigation.

1. Survey of landslide areas

Preliminary geological and feasibility studies were carried out along the pipeline route in 1998. In 2003 and 2004 “Rosstroizyskania” FSUE (Moscow) and “Inzhzaschita” LLC (Sochi City) carried out further studies which focused on the geohazards present in the section of the pipeline route through the Makarov mountains. This section, approximately 170km long, runs through low but difficult mountainous terrain with relief inclinations varying between 15 and 50 degrees. The feasibility studies revealed that this section of the pipeline route, through the Makarov mountains, involves complex geology and geomorphology. This factor, along with the fact that Sakhalin Island lies in a seismically active area, required that special attention had to be given to mitigating the geohazards which presented potential risk to the pipelines and their corridor.

Mitigation of the following geohazards was taken into consideration when implementing the design of the pipeline route: marsh areas, flooding and ground saturation, suffusion (internal erosion), surface erosion, lateral erosion and scouring of watercourses, solifluction; talus, creeps and landslide processes; active neotectonic faults.

A separate study and design process was carried out for the sections of the pipeline route which crossed neotectonic fault locations over the whole length of the route, and is not considered in this presentation which focuses mainly on erosion and landslide geohazards. 183 areas were initially identified as exposing the pipelines to unacceptable geohazardous risk (if not mitigated against), and as a result some 38% of the original route identified in the 1998 feasibility study was relocated.

Design of engineering protection to the pipeline against geohazards was based on detailed investigation of geological structure, and identifying the type and ongoing development of geohazardous processes. In addition, design had to take into account environmental issues and the environmental impact that the pipeline construction would have on the immediate vicinity.

2. Assessment of Risk

The Russian standard document SNiP 11-02-96 “Engineering surveys for construction. Fundamentals” defines engineering risk as “potential loss (social, economic, etc.) from the impact of natural and man-made processes”. For the purposes of assessing technical and economic aspects of design, geohazards were classified according to the following categories of engineering risk:

Category I (low) – low-active erosion processes (potential activation during construction), solifluction, mudflows, landslides up to 1.5 – 2.5 m thick
Category II (medium) – erosion processes which are naturally active, flowing landslides and block landslides up to 4.0 m thick
Category III (high) – flowing landslides and block landslides more than 4.0 m thick, mudflows, landslide slopes with significant spread and steepness.

The table below shows correlation between these categories of engineering risk used on the project and the construction regulations and technical requirements of the Russian Federation.
The primary objective of the project is to embed the pipes into the ground at a location and depth where they would not be subjected to significant displacement during the lifetime of the operational period, taking seismic, hydrogeological and hydrometeorological factors all into account. Design was carried out in compliance with the requirements of SNiP-II-7-81 “Construction in Seismic Regions” and of European Code 8, Chapter 5.

During the design phase, special attention was paid to the following:

a) slope stability calculations performed by “Geoproject” LLC (Krasnodar city) taking into account the specified initial seismicity that was provided by Institute of Marine Geology and Geophysics, Far East division of Russian Science Academy.

b) pipeline crossings over mudflow and avalanche areas taking into account the data provided by Far East Geology Institute, Far East division of Russian Science Academy, Sakhalin subsidiary. The pipelines were designed to be embedded below the level of maximum theoretical mudflow washout level and any zone of dynamic impact of avalanches. Permanent overground facilities (block-valve stations, helicopter sites, permanent access roads, etc.) were designed to be located beyond the boundaries of mudflow and avalanche impact.

c) pipeline crossings of abandoned coal workings of the 1930s – 1940s, taking into account the data received from GPR survey performed by “Geotech” LLC (Moscow) in 2006.

d) engineering measures to protect against bottom scour and lateral erosion at the crossing of streams and rivers within the pipeline corridor. Where necessary and appropriate, gabion wall structures were designed and constructed taking hydrological and hydraulic data into account.

In the Makarovs, difficult terrain conditions and poor physical and mechanical soil properties at some locations resulted in extensive earthworks (up to 1.3 million m$^3$), which had negative geotechnical and ecological consequences as follows:

- change in the stress condition of slopes, generating activation of landslide processes.
- change in surface and ground water flows, with further development of linear and sheet erosion processes, and in some cases, mudflows.
- impact on ecosystem, with significant increase in erosion processes.

Engineering protective measures were designed to mitigate or minimize the negative consequences described above. This included management of permanent spoil locations to ensure they were not located on or near:

- landslides or potential landslide slopes.
- heads of erosion gullies.
- areas subject to flooding.
- mudflow or avalanche risk areas.
- natural ground water discharge (springs).
- areas of landscape and ecological value.

Permanent spoil tip areas were prepared by clearing all organic growth, benching any slopes greater than 10°, and

Table 1 - Correlation of categories of engineering risk used on “Sakhalin-2” project and construction regulations and technical requirements of the Russian Federation

<table>
<thead>
<tr>
<th>Risk degrees of development of geological processes during economic exploration of territories under construction</th>
<th>Categories of hazard of processes</th>
<th>Categories of complexity of engineering – geological conditions</th>
<th>Categories of engineering risk</th>
<th>As per SNiP 11-02-96 “Engineering surveys for construction. Main regulations” engineering risk is “loss probability (social, material, etc.) from the impact of natural and industrial processes”</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Low.</strong> Engineering – geological and hydrological conditions are non-complex. Local measures of engineering protection from geohazard are required.</td>
<td>Mild hazardous</td>
<td>I (non-complex)</td>
<td>I (low) – low active erosion processes (activation under construction is possible), solifluction, landslides-earthflow up to 1.5-2.5 m thick.</td>
<td></td>
</tr>
<tr>
<td><strong>Medium.</strong> Engineering – geological and hydrometeorological conditions are complex; significant development of hazardous processes under man-made impact. Combined engineering protection (2 – 3 processes) is required within limited area.</td>
<td>Hazardous</td>
<td>II (medium complex)</td>
<td>II (medium) – erosion processes are naturally active, flowing landslides and block landslides up to 4.0 thick.</td>
<td></td>
</tr>
<tr>
<td><strong>High.</strong> Extremely complex engineering – geological, hydrometeorological and seismic conditions. Overall protection from combination of interdependent catastrophic and hazardous processes.</td>
<td>Extremely hazardous</td>
<td>III (complex)</td>
<td>III (High) – flowing landslides and block landslides more than 4.0 m thick, mudflows, potentially landslide slopes of significant extension and steepness.</td>
<td></td>
</tr>
</tbody>
</table>
installing basal drainage where required. Tip construction included 90% compaction in 300mm layers, with external slopes not exceeding 1V : 1.5H; slopes in excess of 11m were benched and drained, and surface vegetation restored as quickly as possible to improve stability.

3. Engineering protection - design and application

Mitigation design was carried out to ensure that risk to the pipelines and the pipeline corridor from lateral erosion, washout during floods and snowmelt, mudflows, avalanches and landslide processes was reduced to a practical minimum for the lifetime of the pipeline. Generally, mitigation philosophy for the three risk categories described above was that engineering protection would be provided as follows:

**Category I** - on completion of the main construction.

**Category II** - during construction

**Category III** - prior to construction

Mitigation design and construction was carried out in six interrelated stages as follows;

**Stage 1.** Section-by-section analysis of geological conditions and intrinsic geohazards along the route, determination of typical design areas and those which required a site-specific detailed design. In total, 472 areas were identified that required typical engineering solutions. In addition, 20 areas of categories II and III engineering risk were identified which required site specific detailed engineering analysis and design. The identified geohazardous areas were marked on topographic plans, Scale 1:1000 (67 pages), and registered in the Master Geohazard Register (MGR), indicating station and references to the pipeline alignment sheets, with a description of each geohazard and the proposed design solution.

**Stage 2.** Prior to carrying out detailed design of engineering protection, technical specialists from “Sakhalin Energy”, “Starstroi” and “Inzhzaschita” carried out additional field investigations, including excavation of trenches in order to confirm actual ground conditions following right-of-way construction.

**Stage 3.** Preparation of typical drawings of engineering protection measures for the pipeline route. In total, 24 sets of typical drawings were developed by specialists from “Starstroi” LLC and “Inzhzaschita” LLC. These typical drawings have been developed and implemented throughout the whole route of the onshore pipelines on Sakhalin II project.

**Stage 4.** Allocation, section-by-section, of the typical drawings to the oil and gas alignment sheets, including determination of the scope of construction work involved.

**Stage 5.** Preparation of site-specific detailed designs for particularly complex areas which were identified with engineering risk categories II and III in Stage 1. Maximum use was made of the technical solutions provided in the typical drawings, with adjustments made to suit the site specific condition.

**Stage 6.** On completion of the main construction works and excavation of pipe trench, and before installation of pipe in the trench, the trenches and the Right-of-Way (ROW) were inspected for compliance with the work schedule. Table 2 shows the scope of construction work completed by 1st September, 2008.

Table 2. Summary of engineering protection works carried out along the whole route of the onshore pipeline “Sakhalin-II” project (according to the data from “Starstroi” LLC)

<table>
<thead>
<tr>
<th>Types of work</th>
<th>Units of measurement</th>
<th>Q-ity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cutting of sliding soil</td>
<td>million of m³</td>
<td>1.3</td>
</tr>
<tr>
<td>Drainage arrangement</td>
<td>m</td>
<td>86,200*</td>
</tr>
<tr>
<td>Installation of drainage conduits</td>
<td>m</td>
<td>44,000*</td>
</tr>
<tr>
<td>Erosion protection of slopes</td>
<td>he</td>
<td>670</td>
</tr>
<tr>
<td>Gravity Retaining wall, including reinforced soil</td>
<td>m</td>
<td>1,750</td>
</tr>
<tr>
<td>Bank protection by means of rip-rap</td>
<td>m</td>
<td>68,800</td>
</tr>
<tr>
<td>Bank protection by means of gabions and Reno mattresses</td>
<td>m</td>
<td>34,500</td>
</tr>
<tr>
<td>Mudflow conduit</td>
<td>unit</td>
<td>5</td>
</tr>
<tr>
<td>Technical remedial works of right-of-way</td>
<td>he</td>
<td>3,360</td>
</tr>
<tr>
<td>Biological remedial works of right-of-way</td>
<td>he</td>
<td>3,360</td>
</tr>
</tbody>
</table>

*including fault crossings

The results of the inspections are recorded in geotechnical inspection reports for the pipeline route. These reports record and analyze the as-built condition at each location, describing trenches and pits textually and graphically, providing photographs and logs of in-situ materials, as well as a geotechnical assessment of the efficiency of the engineering protection work as constructed, along with conclusions and recommendations.

Stage 6 works are currently in progress. There are over 110 geotechnical reports on site visits completed to date. Additional independent geological and technical inspections of the works are being carried out by consultants “Scott Wilson Ltd” (London, Great Britain) on behalf of Sakhalin Energy. Photographs 1 to 7 show examples of the engineering protection measures and reinstatement work implemented on site (not included here).

4. Landslide monitoring

Monitoring of hazardous geological processes has been carried out during construction and will continue in the operations phase. Monitoring systems have been installed in 54 landslide locations. Procedures, specifications and the technical philosophy of geohazard monitoring were developed by “Starstroi” LLC with the participation of
“Sakhalin Energy Investment Co”. Field and office works were performed by “Inzhzaschita” LLC and “Avers-1” LLC (Yelizovo).

The main objective of monitoring of geohazardous areas is to foresee and thereby prevent possible failures and deformations which would damage the pipelines and accessory facilities. Final selection of areas to be monitored took into consideration the immensity, intensity and direction of the potential geohazard, the engineering protection measures undertaken during construction, and the potential for the geohazard to develop further due to seismic activity, erosion or other natural processes during the life of the pipelines.

Monitoring during the course of construction included continuous geological inspection, by means of route observations, description of excavations and exposures created during construction, comparison of survey information and factual data. Monitoring work included installation of geodetic bench mark and visual monitoring networks (peg arrays), boreholes with inclinometers and piezometers (produced by SysGeo of Milan, Italy) on active landslides and potential landslide slopes. A full topographical survey of each landslide area has been carried out and mapped at a scale 1:1000.

Monitoring locations vary in size between 2 and 12 hectare - the number of bench marks for the base geodetic network is generally 2 to 5 units, the number of array pegs 9 to 27 units, inclinometer boreholes 1 to 2 units, and piezometer boreholes 2 – 4 units. Instrument boreholes are 10 to 18 m deep depending on geological conditions, and the spacing between the array pegs vary from 20 to 35m (see Figure 2, Layout of monitoring network installation).

Monitoring of the geological environment during the Operations phase should include regular periodic visual inspection along the pipeline route (at least once a month), taking readings of the monitoring network instruments and ground bench marks on active landslides, and observation and recording of any erosion development. Instrumental monitoring of bench marks and inclinometers has been carried out continually during the construction phase since July 2006.

The results of observations are summarized into a special database, and reports issued following each reading. These reports indicate areas of potential activation of each geohazard, and give necessary recommendations to the operations team.

5. Conclusion

Geohazards on the project have been identified using several techniques, and mitigation has been designed and constructed so that risk to the pipelines have been reduced to ALARP level. Where geohazards have the potential to develop due to seismic or natural processes, inspection and monitoring systems will provide an early warning system which will allow the operations team to respond well before any risk to the pipeline is manifested.
Experimental Study on Pre-failure Creep in Soils Induced by the Generation of Pore Water Pressure by Means of Ring Shear Test

Ekaterina Georgieva (Kyoto University, Japan)  ·  Hiroshi Fukuoka (Kyoto University, Japan)

Abstract. This study examines the characteristics of the creep movements preceding the final failure of landslides triggered by single rainfall event. 13 tests on Silica sand-bentonite mixture samples have been conducted by means of ring shear apparatus. Pore water pressure (PWP) generation has been simulated by applying backpressure. As a result the response of the soil under backpressure and constant shear stress $\tau$ has been investigated. The tests are representation of landslides triggered by single rainfall event.

Keywords. Ring shear test · Creep · Pore water pressure generation

1. Introduction

Every year heavy rains trigger landslides of different scales all over the world. Some countries having rainfall seasons or which are attacked by devastating hurricanes and typhoons are suffering mostly of disastrous slope failures that claim human lives, properties and infrastructure and require high cost recuperative activities. Municipalities and geoscientists are both interested in limitation of the risk for people and in preparing adequate evacuation plans when a certain area becomes prompt to occurrence of landslides due to extreme downpours. Understanding of the triggering mechanism and the pre-failure processes in the slopes due to the raising groundwater level are of primal importance when the time of failure is to be foreseen.

The creep movements in the soils have been attracting the attention of the researches since few decades. It is the creep phases having distinctive patterns of time-related displacement and speed - acceleration characteristics which gave a new aspect of the knowledge on the pre-failure behavior of the soils and new approaches in making more precise predictions about when a landslide is going to occur are being developed.

2. The concept of creep in soils

Based on real landslide events and experimental data many researches have found out that in many landslides the failures are preceded by accelerated trend of displacement. It is associated with crack growth, soil particle rearrangement and shear surface evolution.

Creep basically is accepted as time-dependent deformations. In 1965 Saito developed a method for predicting the time of failure of a slope based on creep rupture. Fukuzono conducted tests on artificial slopes with simulation of rainfall. Based on the results he proposed a graphical method to forecast failure time by a plot of reversed (reciprocal) velocity versus time (Fukuzono, 1985). Fukuzono also found that increment of the logarithm of acceleration was proportional to the logarithm of velocity of surface displacement immediately before catastrophic failure. This relation is represented by the equation:

$$\frac{d^2x}{dt^2} = A\left(\frac{dx}{dt}\right)^a$$  \hspace{1cm} (1)

where $x$ is surface displacement, $t$ is time and $A$ and $a$ are constants.

Further investigations have been done by Tuchiya and Ohmura (1988, 1989) who developed a model showing the sliding surface enlarging. Creep has been studied by direct shear box test by Matsukura (1984) and Mizuno et al. (1985), since ring shear test has been used for the same purpose by Araiba (1994), Chiyonobu (1998) and Minamitani (2007). In his research Minamitani has found that a linear relation between $A$ and $\alpha$ exists and also that they do not remain unchangeable during the creep.

3. Objective of the current study

13 tests with different initial conditions of soil properties and stresses have been conducted to investigate the role of the raising pore water pressure on the pre-failure movements in the soil and their parameters. Close similarity to natural conditions has been sought. The parameters $A$ and $\alpha$ which control the Fukuzono’s equation describing the tertiary creep as well as preliminary displacement patterns were to be analyzed.

4. Initial conditions

Similarly to the approach used by Jurko (2007) the simulation of field conditions was derived from an infinite slope, taking into consideration the static conditions acting along the sliding surface and the desired inclination. The total normal stress and the shear stress corresponding to the needed inclination are calculated by the following equations:

$$\sigma_0 = \gamma \cos^2 \theta$$  \hspace{1cm} (2)

$$\tau_0 = \gamma H \sin \theta \cos \theta$$  \hspace{1cm} (3)

where $H$ is the thickness of the soil overlaying the sliding surface, $\gamma$ is the average unit weight of the soil within the sliding mass, and $\theta$ is the slope inclination.

The sample represents a soil block at a certain depth and inclination of the slope while ground water level is rising, due to a heavy rainfall. (Fig. 1)
5. Sample characteristics

All samples for the 13 tests were mixtures of silica sand No.8 and bentonite clay. In 6 of the samples the content of bentonite was 5% and the 7 had bentonite content of 10% by weight. The mixture including 5% bentonite clay by weight is non-plastic and LL and PI are not available. SS8 with 10% bentonite has liquid limit 46% and plastic limit 35.6%. The silica sand consists mainly of sub-angular and angular quartz. Its grain size distribution shows uniformity with the grain size being between 0.001 and 0.4 mm with the dry weight of the fine grains being over 70%. The bentonite consists mostly of smectite minerals which are responsible for its high adsorption abilities and swelling. Full saturation was easily obtained and the pore water pressure generated during consolidation was dispersing slowly. Also during the application of the shear stress there was PWP generated easily.

6. Experimental procedure

Choice of the ring-shear apparatus

An intelligent, undrained, dynamic-loading, torque-controlled ring shear apparatus has been used for the tests. It is the 5th version of a series developed and improved by Sassa et al. (2003) in Disaster Prevention Research Institute (DPRI), Kyoto University. A schematic diagram of its main parts and principles for its running are shown in Fig. 2. DPRI5 has some valuable features which make it appropriate for our experiments:

It can support for unlimited period conditions of constant total normal and shear stress.

It allows undrained tests.

It allows unlimited shear displacement.

It has speed and torque control of shear displacement.

It has a precise monitoring and recording system for PWP, shear resistance and shear displacement.

Technical parameters of DPRI No5:

Shear box:
- Inner diameter 12cm
- Outer diameter 18cm
- Max possible height of sample 115mm
- Shear area 141.37cm²

Preparation of the samples and test procedures

1. Setting the sample and saturation.

The samples are mixed after the sand and the clay have been oven-dried at 105°C for at least 24 hours. The dry deposition method (Ishihara, 1993) has been used and the maximum possible initial height of the sample was applied. All the tests have been conducted at full saturation conditions. For checking the degree of saturation BD parameter test was proposed by Sassa (1988). For obtaining high BD value (equal or more than 0.95) the sample has been firstly introduced by CO2 at low rate for 1 hour so that the air closed together with the sample goes out from the shear box. Thereafter the soil mixture was saturated with de-aired water for at least 12 hours at low rate ensuring that all pores are filled with water.

2. Consolidation to the desired total normal stress

3 different values of total normal stress have been applied in different tests – 150, 200 and 300kPa. Consolidation was allowed to take place until the generated PWP disperses and no significant changes in the height of the sample are observed. Usually that was a period of at least 24 hours. When OCR of 1.5 and 2 had to be attained the unloading was allowed for another 24 hours.
3. Application of the initial shear stress

The applied initial shear stress varied as to represent the slope inclination in combination with the established total normal stress. Few different inclinations were used for the tests ranging from 17 to 35 degrees. The application of the shear stress has been done in drained conditions in steps and by gradually decreasing rate to diminish as much as possible movements in the soil and to prevent failure during this procedure. After the desired value has been established few hours have been waited until any slight movement ceases and PWP becomes 0kPa.

The first 3 steps are preparation stages – in the end of the third one the sample is fully saturated, consolidated and set under the desired shear stress, thus presenting the soil block in the slope.

4. Increasing PWP by application of backpressure

The upper outlets from the shear box were connected to the backpressure system of the triaxial apparatus of the laboratory of RCL, DPRI. Schematically it is presented in Fig. 3. The backpressure was applied by rate of 25kPa per hour in all the tests. No drainage was allowed during that phase. It continues until the sample fails.

Test results

A and α: Summary of the parameters A and α obtained in the tests is given in Table 1.

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<tr>
<th>Test No</th>
<th>Clay content</th>
<th>Total normal stress (kPa)</th>
<th>Inclination (deg)</th>
<th>OCR</th>
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<td>0.023</td>
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</tbody>
</table>

Table 1 Summary of the test results for A and α

It was found out for both mixtures that when the corresponding inclination is the same A and α increase with the total normal stress being bigger. (Fig. 4) (exception for the value of α was the test with 300kPa normal stress). Also both A and α tend to be higher for 10% Be mixtures in the pairs of identical tests for both mixtures (Fig. 4) only one exclusion was observed in the tests with 200kPa total normal stress and 27deg inclination, where A for 10% Be mixture is slightly smaller compared to 5% Be. Another clear tendency was found for the OCR series (Fig. 5) – both parameters decrease with OCR being higher. In 6 of the tests conducted under shear stress corresponding to 20 deg, linearity in the relation between A and α is apparent (Fig. 6). Test 13 only shows exception from the tendency, due to the low value of α.

Fig. 3 System for application of backpressure at the last phase of the test

Fig. 4 Example of the dependency of the parameters A and α on the total normal stress at inclination of 20deg

* Triangles for 5%Be, diamonds for 10%Be.

Fig. 5 Dependency of the parameters A and α on the OCR – both show good linear relation with the OCR value
An interesting picture has been observed when the graphical presentation of all data for A and α was performed and different inclination has been outlined through using different colors. Based on the conducted tests it is quite apparent that the results for any other inclination lay in an independent linear function (Fig. 6).

**Creep patterns.** 5 different creep patterns have been observed (Fig. 7). Tests 7, 8, 10 and 13 with 10% Be did not show any significant preliminary movement and failure occurred suddenly (a). In test 3 preliminary movements took place for 45 min but their rates and total shear displacements were negligible (b). In tests 2 and 9 creep of constant rate was observed for 105 and 25 min respectively ending with the acceleration at failure (c). In tests 1, 4, 5, 11 and 12 there was a common pattern of the preliminary movement – at certain time a movement with big acceleration took place followed by quick full stop and later shear displacement was renovated in slow constant rate ending with quick acceleration and failure (d). Tests 6 showed combination of the last two patterns – after the initial quicker displacement the movement did not completely stop but continued with slow constant rate to the failure (e).

It is important to mention that the samples with 10% bentonite responded with a certain delay to the backpressure. There was a period of 20 to 40 minutes before the pore water pressure sensors showed change. After that the pore water pressure increased to the applied rate – 25 kPa per hour. Also the same samples after failure generated additional PWP so even the removal of the backpressure didn’t lead to self stop of the shearing. In the case of 5% bentonite the response to the backpressure was more or less immediate and after the failure occur when the backpressure was stopped and drained conditions were allowed, the pore water pressure dispersed quickly and the shearing stopped without the need of decrease of the shear force.

**Conclusions**

It was found that the parameters A and α are dependent on the clay content, the initial total normal stress and the OCR. Some relation was also observed in dependency from the inclination of the simulated slope. Totally 5 different creep patterns were distinguished, 2 of which dominating (a and d patterns in Fig. 7).

**References**


Living with Landslides and Awareness Raising: Case Examples from Laprak Landslide, Gorkha, and Aaula Rockfall, Myagdi, Nepal

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Abstract. The Himalayan Range, the highest and the youngest mountain range in the world, is very fragile and delicate. Due to the inherent geological, physiographic and climatic conditions, as well as excessive human interventions, the Himalayan region is quite vulnerable to mass movements, floods and earthquakes. Regular movement of the geological formations causes a number of natural events which turn into disasters when the people living near the events are not prepared for them. Landslides are one of the major geologic hazards in Nepal. Though we cannot avoid the occurrence of landslides, we can reduce or minimize their effects through timely taken suitable preparedness measures and thereby can save the lives and property. Many people are still living in the vulnerable areas, mainly villages in the foothills, in Nepal where they are not prepared to cope with the landslides and in consequence, Nepal faces a loss of significant number of human lives and property every year by the landslides alone. People living with landslides or under the threat of possible landslides are not prepared to cope with the landslides for many reasons. Many people are not prepared because of their ignorance, while in many cases, it is not possible to do so at their own level and requires help of the Government (Landslide experts). Laprak village, situated in Laprak Village Development Committee in Northern Gorkha District in Central Nepal and lies at some 46 KM North of Gorkha Bazaar. It can be reached after a 3 days walk from Gorkha. It is located at latitude 28°13' 4.8" N and longitude 84°48' 12.7" E and is situated at an elevation of 2100 m (6930 feet) from the mean sea level. The Laprak Landslide is active since its first occurrence in 1999 and the nature seems to have warmed enough to take care of it as early as possible. Approximately 3900 people, 98% of them Gurungs and 2% of them Dalits, reside in 520 households within the village. As the result of extremely heavy rainfall (340 mm in 24 hours), the Laprak landslide began moving on 3 July 1999 washing away 1 woman in addition to destroying 5 houses, and about 2 hectares of cultivated land along sides of the Chhelong gully. The landslide has moved repeatedly since then, especially during unusually wet years, and poses a continuing threat to the village (figure 1). Another danger situation has been assessed at Aaula village in Myagdi in West Nepal where a great risk has been imposed to some 150 households by the rock falls from the weathered rock mass of Bhirkateri cliff above the settlement. First time in September 2006, some parts of the weathered rock mass fell down & some rubbles rolled down to the village. Luckily it occurred on the day time and people managed to escape from their houses. One house was completely damaged. Since then, some portions of the weathered rock mass is falling down from time to time, especially during monsoon (figure 2). Though the locals toiled hard to fill some 500 gabions to build the security wall in winter of 2007 & 2008, it has been badly damaged recently (August 2008) by big rubbles disintegrated from the weathered cliff and now the gabion wall seems to be ineffective for large boulders. The ground has sunk and deep crevasses are formed in the rock mass. The larger masses of the weathered rock are likely to fall down at any time under the effect of heavy rainfall or an earthquake.

Keywords. Himalayan range, Nepal, Landslides, Rock fall, vulnerable places, Lack of preparedness, Awareness

1. Introduction

Every monsoon (June – September), news about landslides (slump, mud flow, debris flow, rockslides, rock fall ) and losses they bring, has been common for Nepalese people. But if we take precautionary measures in time, we can save lives of people & minimize the losses. Laprak landslide is situated in Laprak village in Laprak Village Development Committee, a remote place in Northern Gorkha District in Central Nepal and lies at some 46 KM North of Gorkha Bazaar. It can be reached after a 3 days walk from Gorkha. It is located at latitude 28°13' 4.8" N and longitude 84°48' 12.7" E and is situated at an elevation of 2100 m (6930 feet) from the mean sea level. The Laprak Landslide is active since its first occurrence in 1999 and the nature seems to have warmed enough to take care of it as early as possible. Approximately 3900 people, 98% of them Gurungs and 2% of them Dalits, reside in 520 households within the village. As the result of extremely heavy rainfall (340 mm in 24 hours), the Laprak landslide began moving on 3 July 1999 washing away 1 woman in addition to destroying 5 houses, and about 2 hectares of cultivated land along sides of the Chhelong gully. The landslide has moved repeatedly since then, especially during unusually wet years, and poses a continuing threat to the village (figure 1). Another danger situation has been assessed at Aaula village in Myagdi in West Nepal where a great risk has been imposed to some 150 households by the rock falls from the weathered rock mass of Bhirkateri cliff above the settlement. First time in September 2006, some parts of the weathered rock mass fell down & some rubbles rolled down to the village. Luckily it occurred on the day time and people managed to escape from their houses. One house was completely damaged. Since then, some portions of the weathered rock mass is falling down from time to time, especially during monsoon (figure 2). Though the locals toiled hard to fill some 500 gabions to build the security wall in winter of 2007 & 2008, it has been badly damaged recently (August 2008) by big rubbles disintegrated from the weathered cliff and now the gabion wall seems to be ineffective for large boulders. The ground has sunk and deep crevasses are formed in the rock mass. The larger masses of the weathered rock are likely to fall down at any time under the effect of heavy rainfall or an earthquake.

Keywords. Himalayan range, Nepal, Landslides, Rock fall, vulnerable places, Lack of preparedness, Awareness

2. Geology:

2.1 Laprak Landslide: The landslide has a ridge line above its head and the snow fed Raizo river at its toe. The average gradient of the slope is 26°. It appears to be part of a much larger ancient landslide complex of unknown age and depth,
which may or may not involve bedrock. Soil thickness in the area of active land sliding varies from 5 to 15 m. Bedrock consists of mica schist, phyllite, quartzite, and gneiss with foliation dipping steeply towards the northwest. The Main Central Thrust (MCT), a major fault capable of generating large earthquakes, lies within several hundred meters of the village. The bed rock is anticline at the eastern part of the village and syncline towards the main settlement and again anticline at the western edge of Chhelong gully. The colluvial mass is deposited at the syncline part, where the settlement is located and is under the influence of active mass movement.

Figure 2: (2a) weathered rock mass. (2b) Aaula village

2.2 Aaula Rockfall: Aaula village is situated at the foot of Bhirkateri cliff (figure 2b). The cliff consists of Gneiss and Phyllite rock. The exposed rock mass at the top of the village is highly weathered and there is a fault passing through the rockmass. Further there are many cracks and crevasses in and around the weathered rock mass. It appears to be part of a tectonically active part, which might be suffered further from the past earthquakes before it started to fall down. The surface water enters into the rock fissures and increases the pore water pressure thus is creating a favourable condition to slide the rock. There are some trees (jungle) above the village but unfortunately there is hardly any trees in the area, where the rubbles roll down. This has further eased rock falls to reach to the village and has increased more threat.

3. Characteristic Description

3.1 Characteristics of Laprak Landslide

According to the classification of the Landslides (Varne 1987), Laprak landslide falls under the complex type (combination of rotational, translational and flow). It is still active and moving downward. There are several local slides within the area. Laprak landslide is 1500 m long and 20 – 270 m wide. The landslide has maximum width below the western side of the village along Chhelong gully where the landslide is active every monsoon (figure 4). Several tension cracks up to 60 cm wide are appeared in and around the village. Beginning with the translational landslide at the head scarp, the slide turns into the debris flow as it travels few hundred meters down. The debris flows in the Chhelong gully has invited further landslides along the adjacent loose soil deposits of the gully, particularly below village, thus rotational slides (at the right bank of the Chhelong gully) and translational slides (at the left bank of the Chhelong gully) are present. At the top of the village, a rotational slide with a big tension crack of some 200 m long is appeared and the tension crack has extended up to the School area. However, the rotational slide at the top of the village is not expanding since the first occurrence in 1999 and it is so perhaps because of the existence of high permeable soil (gravelly sand) in this area. But every monsoon rainfall has triggered further landslides along Chhelong gully, mainly at its both sides below the village, where the area is dominated by silty sand.

Figure 3: (3a) detached Rock mass, (3b) security (Gabion) wall

3.2 Characteristics of Aaula Rockfall

The Aaula village is located facing south. The direct sun heat and excessive rainfall are contributing to disintegrate the highly weathered rock mass above the Aaula village. Further the foliation of rock mass is parallel to the slope and has steep slope gradient (45°). A big rock mass of 30 m long and 20 m high is detached from its parent rock mass creating a deep fissure along the joint (figure 3a). The detached mass is not only highly weathered but has also several cracks on its
surface. Some portion of it fell down in 2006, luckily the mass broke into several small pieces of rubbles before it reached down to the village.

**Figure 4**: Active part of Laprak Landslide

### 4. Causal factors

#### 4.1 Causal factors for Laprak Landslide

The combination of weathered rock, steep slope, high intensity of monsoon rainfall, parallel foliation of bed rock, excessive human interventions (mainly heavy cultivation all the year round and deforestation) might have caused the Laprak Landslide. The main factor that triggered the Laprak landslide appears to be the high intensity of rainfall. A total 341.8 mm rainfall was recorded on the day (3rd July 1999), which is the highest rainfall intensity ever recorded in the area in a day (Figure 5). There is a great possibility that earthquakes of even small magnitude, may trigger further landslides in Laprak and particularly the existing houses are very prone to collapse during the earthquake.

**Figure 5**: Rainfall pattern in Laprak

#### 4.2 Causal factors for Aaula Rockfall

The combination of weathered rock mass, steep slope, monsoon rainfall, parallel foliation of bed rock might have caused the Aaula Rockfall. Besides, there is a great possibility of rock fall in the event of earthquakes.

### 5. Present condition

#### 5.1 Laprak Landslide

The Laprak landslide since occurred first in July 1999, is still active and continuously expanding, especially during the monsoon periods. In particular, the soil along the sides of the Chhelong gully is moving and the houses at the vicinity of this gully are more vulnerable than those on other parts of the landslide. There are also many large tension cracks from 35 to 60 cm wide and 20 to 200 m long. The landslide is 20 to 270 m wide and 1500 m long. New fissures (tension cracks) are developing in and around the village. No prevention work has been implemented or done to date to control the landslide.

#### 5.2 Aaula Rockfall

The Aaula Rockfall ever since started first in September 2006 is still active. Rock fall took place this monsoon also (10 August 2008). This time the security wall (Gabion wall) saved rubbles to reach to the village but it is heavily damaged and may not work for similar rock falls in the future. If similar mass splits again in next time, then it may reach to the village and cause severe damage.

### 6. Effect of the landslide

#### 6.1 Laprak Landslide

Most of the existing houses in Laprak village are suffering from the significant floor and wall cracks. Many foundations show such signs of differential settlement as bulging and tilted walls (Figure 6). Because the houses are built with unreinforced dry stone walls, there is a possibility of more death or serious injury if the walls collapse as a result of ground movement (either because of continued landsliding or future earthquakes). People are spending sleepless nights during the monsoon time because of the fear of possible landslides.

#### 6.2 Aaula Rockfall

One house was damaged by the rock fall in September 2006. Though the Gabion wall (which is built at the top of the village to check the rubbles from coming down to the village) was under minor damage by rock falls every monsoon, it has been seriously damaged by the rock fall this year (August).

### 7. Awareness Raising among villagers

Once vulnerable areas to Landslides are identified, people living in the affected area should be made aware about the possible threat. Some landslides are continually active for many years and never seem to recover. People should understand that for deep failures & major slides, preventive measures become more expensive and usually ineffective & should be avoided as far as possible. At least, people should be made aware about the safe exit before the landslide turns into disasters. There are, as of today, no exclusive educational courses or training programmes on Landslides for the community people. The local people in Laprak and Aaula have been informed about the threat posed by the possible landslide by the author himself and other experts. Also a couple of reports have been submitted to the relevant...
Government offices for the immediate action to control the landslide. Unfortunately all efforts are in vain. Neither local people are doing something nor the Government has started doing anything. This is true that though most of the local people are aware and worried about the possible landslide, most of them are compelled to live in these villages. They don't have any alternative except living with landslides. For others, they don't bother about landslides because of ignorance. They believe that landslides are God's act. It is interesting to mention here that villagers of Aaula had sacrificed 7 billy goats last year in the name of God praying for the stoppage of Rock fall. Under such circumstances, it is difficult to raise an awareness in the community at an individual level and for that, a fixed program from the Government level should be implemented. Many things can be done at the community level to thwart any untoward incident. Training for locals, specially for teachers and students, may help to create an awareness in the community effectively and quickly. Different warning systems can be used to make aware about the impending disaster based on different factors. For example, daily rainfall or rainfall threshold can be used to warn the people prior to the landslides.

8. Preventive measures
The nature is giving enough alarming to take note of the possible disasters in these areas. The landslide issues should be addressed as early as possible both from the community level and government level.

8.1 Preventive measures for Laprak Landslide
Stabilizing the existing Landslide in Laprak need structures (for example, deep reinforced concrete piers) or deep horizontal drains, which would be extremely expensive and impractical given the available resources. If such measures are not applied or maintained properly, moreover, there is a high probability that landsliding will continue. Small scale civil structures such as retaining walls not tied into bedrock, shallow drains, or bio-engineering solutions are likely to have little or no effect on the deep landslide in Laprak. Unless structures are embedded into stable bedrock, which would be virtually impossible given the existing resources and manual excavation, they will not stabilize the deep landslide. Therefore following measures are recommended.

a. Regular monitoring of landslide movement is essential. Although monitoring should occur throughout the year, it is particularly important during each monsoon season. The monitoring program should be designed and implemented under the supervision of qualified engineers and geologists.
b. Existing houses that are affected (either cracked or tilted) by the landslide should be dismantled and replaced with lightweight houses as shown in figure 7. The geology and soils of Laprak are not suitable for the existing houses, which have heavy but weak stone walls. It is my opinion that the proposed lightweight houses will save lives in the event of a future earthquake or continued landslide movement.
c. Gabion check dams at different locations along the Chhelong gully (at least at 5 locations), surface drains, sub-surface drains in the entire slope should be built to improve shallow slope stability. These measures may not help to stop the deep landslide, but they will help to reduce damage and loss of cultivated fields to the shallow landslides and erosion that are common around Laprak. For sub-surface drains, gabion bolsters are effective.
d. The tension cracks at the crown of the Landslide should be sealed with stone and mud, and those areas planted with trees.
e. Suitable vegetative plantation should be done in the entire slope of Laprak.

8.2 Preventive measures for Aaula Rock fall
a. The Gabion walls (security walls) should be built at different locations (at least at 5 places) along the path of rock fall between the village and cliff.
b. The steep slope above the village should be reduced by making terraces so that it helps reduce the velocity of rolling rubbles.
c. The water which is directly seeping into the cracks at the joint between the detached rock mass and its parent rock should be diverted to safer areas by putting geo-membranes and sealing off the cracks.
d. Regular monitoring of rock fall is essential.

9. Conclusion
Not only people in the above mentioned villages are living with Landslides in Nepal, surely there are many such areas which are unnoticed. Since certain class of landslides could be predicted and most landslide disasters could be averted by the prepared communities; landslide awareness, education and training should become integral part of landslide disaster mitigation strategy. Blending of landslide education with educational curricula, institutionalization of specialized education on landslides by research and conduct of community awareness programmes deserve high priority. It is high time to identify such vulnerable places at the earliest and discuss among people in the communities to make them aware and to address landslide issues on time in order to make this globe a better & safe living place in the days to come.

References:
Abstract. There are more than 6,000 rivers and streams in Nepal. Most of them flow from north to south, generally with high velocity due to high river gradient. Most of the big rivers originate from the Himalayan range that is covered by perpetual snow. As the topography of the country is steep, rugged and high-angle slope with complex geology, very high intensity of rainfall during the monsoon season causes flood, landslide and debris flow. Landslides are the most destructive type of disasters for engineering structures in Nepal. Landslide is a common problem in Nepal and hence a proper site study is necessary for the infrastructure development. Three quarters of the total land area of the country is hilly and many villages are situated on or adjacent to the unstable hill slopes. News about landslides, thereby damage & losses it brings, has been common for Nepalese people every monsoon. Many projects have been washed away by landslides. Most landslides in Nepal are rainfall triggered Landslides. More than 80% of the annual rainfall takes place during the monsoon season and more than 90% of it occurs within three months (July – September). Annually a considerable number of engineering infrastructures are either partially or fully damaged by landslides alone in the Lesser Himalayan region (700 – 2000 m altitude) of Nepal. Lack of proper study of the site for possible landslide occurrence is often found the main reason for these disasters. If we take precautionary measures on time, we can save both lives and property & thus can minimize the effect of landslides. Regular movement of the geological formations causes a number of natural events which turn into disasters when the people living near the events are not prepared for them. We can not stop these natural events but we can reduce or minimize their effect through suitable preparedness measures. Many people are still living in the vulnerable places where there is a threat of potential landslides. Besides, due to the rugged terrain and fragile geology, construction of engineering infrastructures is extremely difficult and equally challenging. A proper geotechnical study and application of suitable preparedness (remedial) measures in time may lessen Landslides’ risk and save both lives and property.

Key Words: Nepal Himalaya, Landslides, Extreme rainfall, Rugged terrain, Steep slope, Infrastructure development, Geotechnical study

1. Introduction
The fragility of the Himalaya is mainly attributed to inherently weak geologic condition, extreme relief, very steep and rugged topography, strong south Asian monsoon (90% annual rainfall occurs within 3 months of the year). Thus the construction of engineering structures in Nepal Himalaya is very challenging. The present trend in Nepal is that most small scale engineering projects (micro hydro projects, trail suspension bridges, water supply schemes, schools, rural roads, etc.) are constructed either without any geological study or carried out by those who have inadequate geological knowledge. In consequence, most of them have been the victim of natural events, usually by Landslides (figure 1).

2. Mass Movements: The different type of mass movements (figure 2) that are responsible for causing damage to infra-structures projects and current practice of treatment in Nepal Himalaya are discussed below.

2.1 Falls
Different type of falls eg., earth fall, rock fall, debris fall are
frequent in hilly areas of Nepal Himalaya. The rock fall can be treated in most cases but the rock slide is like a landslide and may become difficult to stabilize. There are several cases where infra-structures have been damaged either partially or fully by these fall events. Falls generally occur along steep slopes. The material is detached from the parent one usually by weathering and jointing. Sometimes earthquakes and slope un-stability also cause falls, thereby damage infrastructures. Falls may involve sliding or flow.

**Treatment:** Stabilization work is carried out only at those sites where treatment is possible. For earth / debris fall, series of gabion check dams are constructed and the water is diverted by surface and subsurface drains to the safer location. For rock fall, gabion retaining walls are built. Besides, combination of vegetative turfing and jute & coir netting are also used to control falls.

### 2.2 Topple

It is a type of fall. It generally involves pivoting or forward motion of rocks, debris or soil. Generally it is difficult & expensive to stabilize the topple falls. Hence such sites are usually avoided for treatment.

### 2.3 Rotational landslide

Rotational slide is that form of failure which occurs along a distinct more or less semi-circular or curved shear slip surface (spoon shape), and usually occurs in shales, mudstones and clays (Homogeneous mass). Here the rupture surface is curved concavely upwards and the slide movement is rotational about an axis parallel to the slope. Rotational slides usually have a steep scarp at the upslope end and a bulging "toe" of the slid material at the bottom of the slide. Rotational slides may creep slowly or move large distances suddenly. Rotational slides usually develop after prolonged rainfall in Nepal Himalaya. It is very difficult to control the deep Rotational slide and is quite expensive too.

**Treatment:** It usually consists of a combination of slope dressing, surface and subsurface drainage and provision of restraining structures (such as soil nailing, bolting, and anchoring). Mostly soil anchors, horizontal gravity drains, surface drains, toe protection walls (retaining wall), sealing of surface cracks, use of live plants (vegetative turfing) are used to control the Rotational slide. Retaining wall (cement masonry / Gabion wall) application at the toe of the landslide area are quite common in roadside slopes (figure 3).

### 2.4 Translational landslide (Plane failure)

Translational Slides are those in which the moving material slides along a more or less planar surface. Translational slides occur on surfaces of weaknesses, such as faults and bedding planes or at the contact between firm rock and overlying loose soils. Translational slides may creep slowly or move large distances rather suddenly. An intense rainfall following few days of antecedent rainfall usually develops plane failures. The shallow deposit above the bed rock slips down due to the loss of interface shear strength.

**Treatment:** It usually consists of a combination of slope dressing, surface and subsurface drainage and provision of retaining structures (gabion wall, breast wall). Use of live plants (vegetative turfing), jute core & netting are used to control this slide. In special cases, such as for example at Kaligandaki Hydro Electric Project, soil nailing followed by shotcreting and stone pitching are in use (figure 4). Shotcreting of the slope surface is recommended to protect the slope from infiltration of rainwater as well as from direct impact of the intense rainfall and runoff to prevent excessive erosion. However shotcreting is expensive, hence its use is limited to big projects only in Nepal.

### 2.5 Debris Flows

Debris flow (figure 5) is a common landslide in Nepal Himalaya. It is usually associated with the high intensity of rainfall. Debris Flows (also called debris torrents) are movements in which loose soils, rocks and organic matter combine with entrained water to form slurries that flow rapidly downslope or within a stream channel. Generally in steep slopes, they become more powerful & dangerous as they move down further.

**Treatment:** Management of surface and sub-surface water comes under the first priority to control all type of landslides. Gabion Check dams at suitable locations are usually built to control debris flows in Nepal. Sometimes concrete check-dams are also built, especially when there is excessive quantity of debris flow with big boulders (figure 6).

### 2.6 Flash floods

Landslide adds enormous load to the streams and rivers, causing flash floods downstream (figure 7). These are not common but can cause damage to a greater extent. Flash floods emerge out of a landslide mass and they become more
destructive when they flow through a narrow channel. We can prevent our structures from it by constructing our structures far from the river banks and by allowing sufficient freeboard for crossing structures such as bridges.

2.7 Avalanches

Though avalanches are very rare in lesser Himalaya (700 - 2000 m) in Nepal, they are destructive in nature. If there is a heavy snow fall in the mountains, then avalanches occurs. We should treat avalanches as mud flows or debris flows and similar engineering measures (as applied in debris flows) are applied to protect structures from the avalanches.

2.8 Complex Landslides

When there are more than two different types of landslides in the slope movements, it is called complex Landslides. Stabilization of complex landslides is usually difficult and expensive. Krishnabhir Landslide which lies on the main Prithvi Highway at 70 KM west of Kathmandu, was a famous complex landslide in Nepal during the period 2001 - 2004.

**Treatment:** A typical model of Bio-engineering technique is successfully applied at Krishnabhir landslide. Surface drainage, sub-surface drainage, concrete and gabion check dams, Toe wall (cement masonry retaining walls) in combination with vegetative turfing and live plants are used to stabilize the Krishnabhir Landslide (figure 8)

3. Bio-engineering Mitigation Measure

In Nepal, bio-engineering measures to control the landslide events are in use since a decade ago. There are many successful examples throughout the country. The use of living plants either alone or in combination with small scale civil engineering structures (Gabion wall, check dams, surface drains, Retaining walls, etc.) or non living plant material for reducing the shallow seated instability and controlling erosion on slope is called bio-engineering. It is very popular in Nepal, especially along road side landslide stabilization and peripheral landscaping works. It is cost effective and involves no high tech. The main advantage in bio-engineering technique is that civil engineering structures (Gabion wall,
retaining wall, check dams) function very well for the first few years. Then their strength slowly decrease with time, while live plants gain strength with time and they perform very well in the end when Civil engineering structures become almost functionless with time. Thus the combination of these two make a perfect solution for long-term slope stabilization. A live plant can perform all engineering function required to stabilize a slope.

<table>
<thead>
<tr>
<th>Problems on the landslided slope</th>
<th>Engineering Function</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water enters into slope or liquefy the slope material</td>
<td>Armour Function</td>
</tr>
<tr>
<td>Materials roll down the slope</td>
<td>Catch Function</td>
</tr>
<tr>
<td>Loose state of materials</td>
<td>Reinforce Function</td>
</tr>
<tr>
<td>Outward and downward movement of slope</td>
<td>Support Function</td>
</tr>
<tr>
<td>Slip of overlying layer</td>
<td>Anchor Function</td>
</tr>
<tr>
<td>Accumulation of water</td>
<td>Drain Function</td>
</tr>
</tbody>
</table>

Figure 9: (9 a) Before treatment, (9 b) After treatment

Figure 10: (10 a) Brush-layering of live plants at a roadside landslip, (10 b) the landslip site 5 months after the treatment

4. Conclusion

Technological innovations in Nepal in the area of Landslide Control is still on primitive stage. A proper investigations of landslides is essential, since it is not possible to design an effective mitigation system without proper understanding of the slope problems. Many development schemes implemented without carrying proper geological and geotechnical investigation due to shortage of money or time or due to other constraints. In order to avoid or minimize the landslide hazard, a proper geological and geotechnical investigation is essential prior to implementation of any development scheme. First of all, proper investigation method of landslide should be selected according to the need, which will save time and money too. A landslide calamity may be avoided or at least minimized by applying appropriate remedial measures or set remedial measures at the initial stage of development scheme.

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References:
Mechanism and Application of New Steel Check Dam in Debris Flow Prevention

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Abstract. Japan topography, one of the steepest topography in the world combining with frequency earthquake and heavy rain triggered many debris flow those annually caused serious lost of dead and economic. The debris flow is often formed as a torrent of water and soil rock in rainy season or deposit soil of a slope failure that rapidly move to downstream and possibly damage infrastructure, environment etc. Photo 1 shows the debris flow occurred by Iwate-Miyagi Nariku earthquake that destroyed a small hotel at the downstream and caused five dead.

Many types of steel check dam for debris flow prevention have been constructed from 70’s year. This paper will explain the type of main steel check dam and will discuss about the application of steel check dam.

Keywords. Steel check dam, debris flow

1. Type of the main steel check dam

In Japan, there are two kinds of structure of steel check dam, open and close dam. Open dam is divided 2 types, Slit dam and Debris breaker. Open dam is usually structured by steel pipe. It has the open floodway that was designed to capture the largest gravels following a suitable factor. Open dam is very permeable that reduces water level and pressure of soil water acting upon the dam and thus increases dam stability (Photos 2 a and b). Debris breaker like a tunnel (Photo 2 c), it is lower but longer along the flow of river compared with slit dam.

There are many kinds of close dams such as Cell dam (Photo 3b), Frame dam and the newest one, Soil cement wall (Photo 3a). Soil cement wall is structured by mixture of soil rock; recycle material and cement. It is new environment solution having mechanism similar to that of a filled dam.

The circle walls that is structured by steel plate, steel sheet pile or steel panels and is filled by soil rock form cell dam (Photo 3b). Frame dam is constructed by steel frame and is filled by gravel or rock.
2. Application of check dam

Photo 4 shows Slit B dam captured debris flow and debris wood. Photo 5 shows Debris breaker captured mudflow in Sakurasima volcano, Kagoshima prefecture.

Photo 3a Soil cement wall

Photo 3b Cell type dam

Photo 4 Slit B dam captured debris flow (downstream side)

Photo 5 Debris breaker captured mudflow (upstream side)
Selection Method of Emergency Measures for Snow Avalanche in Japan

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Abstract. Examples of emergency measures for snow avalanche were collected to develop selection method. The examples were classified according to the active/passive control and manual/machinery work. Finally a flowchart was made as selection method of emergency measures.

Keywords. snow avalanche, emergency measures, selection method

1. Introduction
In the winter 2005-06, large snowfall led to a number of avalanche events, house collapses, traffic closures and village isolation in Japan. Hence we started a research on avalanche risk assessment method in heavy snow to overcome the future extreme snowfall. The aim of this study is to develop a selection method and to publish a manual of emergency measures for snow avalanche. Here we introduce the emergency measures for snow avalanche in Japan.

2. Methods
We collected examples of emergency measures from administrative offices and constructors in heavy snow area by questionnaire and interview. 60 cases were gathered and past emergency measures were also reviewed\(^1\), \(^2\). Figure 1 shows flow of the method.

3. Results
3.1 Examples
We classified the cases according to the object and method (Figure 2). Here we illustrate typical examples.

3.1.1 Active control – Cornice control
Snow cornices around ridge and top of the slope have a possibility to trigger the snow avalanche. Removal of cornices is carried out by the following ways:

- Manual handling – manual cornice removal when heavy construction equipments are difficult to access. Attention should be paid to safety of workers not to slip down and avalanche triggering by the removed snow blocks (Figure 3). Wire rope and bucket truck are also used particularly for the road facilities to prevent snow dropping (Figure 3).
- Machinery handling – machinery cornice removal at the slope less than 25m in height. Cornices can be cut by not only backhoe bucket but also a steel plate hoisted by a crane truck (Figure 4). This is an efficient method but have limitations on the access and the slope length.
- Manual and machinery handling – cornice removal using both manual and machine at the relatively long slope (Figure 5). In addition, following conditions require this method: first, change of the slope angle prevents visual observation of the upper part of the slope from the machine; second, the arrangement of the avalanche control structures have a danger

Figure 1 Flow of research on emergency measures for snow avalanche.

Figure 2 Classification of emergency measures. Numbers of collected examples are indicated.
to scrape against the machine (Figure 5).

Explosive control – cornice removal by detonating explosives in the starting zone. One benefit is a short-time control of the large mass of snow that is out of reach of the machine. Another one is the snow on the avalanche truck can be controlled at the same time when the collapsed snow flows down with entraining the snow. However, the use of the explosives is strictly restricted in Japan; long-time application and detailed planning are required. To overcome this situation, non-explosive demolition agent has recently applied to the avalanche control (Figure 6).

Figure 3 Cornice control by manual handling using shovel (left) and wire rope (right, broken line).

Figure 4 Cornice control by machinery handling using excavator (left) and crane truck (right).

Figure 5 Cornice control by manual and machinery handling.

Figure 6 Cornice control using explosives. Plant charge (left) and examples of explosion using non-explosive demolition agent (right).

3.1.2 Active control – slope stabilization
As a method of the slope stabilization, combination of boot packing and stepped terraces has been reported. This involves workers walking along the contour line and making terraces from the top to bottom (Figure 7). It is efficient to fill the snow in the crack and tread them for preventing the full-depth avalanche.

3.1.3 Passive control – construction of defensive structure with snow
If the avalanche disaster is susceptible beside the long slope that the starting zone is difficult to access, construction of defensive structure is effective as the avalanche measures. Under the emergency condition, temporal structure using snow is one of the choices due to the low-cost, high-workability and natural removal after the winter. Construction is carried out by following ways:
3.1.4 Passive control – construction of defensive structure with construction materials

Installation of construction materials is also valid as the defensive structure. Concrete blocks and large sandbags are used when the uphill space is narrow or the amount of expected avalanche volume is small (Figure 10).

3.1.5 Avalanche detection and warning system

Example of wire sensor as a detection system is reported. One sensor was installed on the snowy arrester in the avalanching slope. The sensor was connected to signal below the arrester; if the snow avalanche cut the wire, the signal will turn red (Figure 11).
3.2 Selection method
We made a flowchart so as anybody can take proper emergency measures. In the flowchart, condition of the avalanche slope (e.g. accessibility, working space, slope length) is asked and then proper method can be indicated.

Conclusions
We collected the existing examples of emergency measures for snow avalanche and classified them according to the active/passive control, type of measures and manual/machinery work. We also made the flowchart for the selection of the proper emergency measures. These results will be distributed as a manual of emergency measures for snow avalanche.

Acknowledgments
The authors would like to thank Ministry of land, infrastructure, transport and tourism Hokuriku regional development bureau, Prefectural government of Niigata, Nagano and Fukui, and constructors for providing examples of emergency measures.

References

Figure 12 Flowchart for emergency measure selection.
Green Slope, A New Approach of Natural Slope Stabilization in Urban Areas

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Abstract. Due to the issue of industrialization and urbanization, there are very few green slopes remaining in urban area even though these slopes play a very importance role in the daily life of citizens. Fig.1 is the forest map of Pref. Kanagawa, close in SW of Tokyo. The forest is concentrated in the west area (the gray zone on left side of Fig.1), while very few forest in the east area, where Yokohama and Kawasaki cities located. Yokohama and Kawasaki are two of the largest cities of Japan. Fig.2 shows the number of slope failure of Kanagawa prefecture. The east area clearly has the number of slope failure greater than that of the west area. Figs. 3a and 3b shows concrete frame and retaining wall those are the traditional methods and are very common in Yokohama and Kawasaki. These kinds of slope stabilization methods partly clear green, damaged natural landscape, environment and finally was not effective (Fig. 2). Figs. 1 and 2 give us a general view of “the greener area having smaller number of slope failure”. The concept of green slope must be considered in slope stabilization plans in urban areas.

Keywords. Green slope, slope failure, natural slope stabilization, urban environment

1. New approach in slope stabilization

In a natural slope, it is important to prevent against the shallow landslide and slope failures in the weak topsoil. By using steel bars in combining with natural tree root to fix unstable topsoil into the bedrock, the slope stability is increased (see Fig. 4). The reinforcement mechanism of this new soil nail method is similar to that of a natural tree root. Hereby, steel bar is similar to vertical nail, bearing plate is similar to stump and wire rope is horizontal root (see Figs. 5 and 6).
2. **Application in urban area**

Japanese with their traditional cultural often live close to natural forest. The green slopes are thus really needed for Japanese recreation as a special kind of natural resort. The green also play a key role in controlling temperature, air pollution and reducing noise, mental stress…the environmental and society problems occurring very frequency in most of urban areas. The new soil nail is suitable solution that can solve the problems of slope failure while protecting and thus creating the recreations as mentioned above. The application of this method is necessary for the new urban plans concerning slope stabilization in the future.

![Fig. 4 New approach of natural slope stabilization](image)

**Fig. 4** New approach of natural slope stabilization

![Fig. 5 Relationship between tree and reinforcement parts](image)

**Fig. 5** Relationship between tree and reinforcement parts

![Fig. 6 Outline chart of New soil nail method](image)

**Fig. 6** Outline chart of New soil nail method

![Fig. 7 Applications in Urban area](image)

**Fig. 7** Applications in Urban area

(a) Near the central Yokohama city

(b) Atagoyama, Tokyo
Analysis of Slope Stability Using Grid Net
Bo-An Jang(Kangwon Nation University, Korea) · Hyun-Sic Jang(Kangwon Nation University, Korea) · Bo-Hyun Poong(Kangwon Nation University, Korea)

Abstract. When we evaluate slope stability, we regard the slope homogeneous and evaluate slope stability at the most dangerous portion of slope. However, since conditions and properties of rock mass/soil are different from one location to another within a single slope, slope stability evaluated by present concept cannot represent slope correctly. This also results in over-reinforcement at the portion where reinforcement is not necessary. In order to solve these problems, we suggest cell unit evaluation method in which we apply small rectangular cells in a slope and regard each cell as a single slope. In this method, slopes are classified into soil slope and rock slope depending on materials. Strength of rock, volumetric joint count, spacing of joints, condition of joints, ground water condition and so on are examined and SMR and risk index values are calculated. Finally, all data and results are presented as contour maps.

We apply cell unit evaluation method into cut slope. SMR values estimated by the new method are larger than those by present concept at most portions of slope, indicating that the new method suggested by this research represent slope stability more correctly than methods which were used. This method will prevent over-reinforcement at the portion of slope where reinforcement is not necessary.

Keywords. slope stability, cell unit evaluation, risk index

1. Introduction
Although characteristics of rock mass/soil mass, such as material, strength, degree of weathering and characteristics of discontinuities and so on, are different from one location to another within a single slope, we analyze slope stability assuming the slope as homogeneous. However, characteristics of rock mass/soil mass are different from one location to another within a single slope. Until recently, we measure characteristics of slope at the most critical region and the stability analyzed is regarded as stability of the whole slope. Therefore, the stability is usually underestimated and support measures more than necessary are carried out.

We suggest cell unit evaluation method to solve the problem mentioned above. Slope is divided into many rectangular cells and each cell is regarded as a single slope. Characteristics of rock mass/soil mass and slope stability of each cell is estimated and stability of whole slope is presented as contour map. Support measures can be carried out to only the region necessary if this method is used.

2. Method
The sequence of investigation consists of formation of cells, investigation, data processing, contour map and stability analysis. Slope is divided into equal sized rectangular cells and identification number is assigned to each cell. The size of cells depends on size of slope and number of cells divided. Each cell is regarded as a single slope and site investigation for slope stability is performed for each cell. Cells are classified into soil cells and rock cells depending on constituting materials. Different items for investigation are used in soil cells and rock cells. Orientation of cell, strength of rock, volumetric joint count, orientations of discontinuities, characteristics of discontinuities and geological characteristics of each cell are investigated for rock cell. Depth of soil, characteristics of soil, groundwater condition and failure history are examined for soil cells.

Kinematic analysis, limit equilibrium analysis, SMR classification and risk index analysis are performed for each cell based on data collected by site investigation. Contour maps are drawn for strength of rock, volumetric joint count, depth of soil, risk index and SMR values (Fig. 1).

Fig. 1 Flow chart of Cell unit evaluation method

3. Risk index analysis
Because evaluation systems and the maximum ratings of rock cells are different from those of soil cells, stability of rock cells and soil cells cannot be presented in one contour map. Ratings of cell measured are divided by the maximum ratings and then, ratings of cell can be presented as a percentage to the maximum rating, which is called risk index. Risk index for rock cell is evaluated similar with the SMR method, except that volumetric joint count is used instead of RQD and length of fault is included. Risk index for soil cell is evaluated by
investigating characteristics of soil, groundwater condition, dip of cell, failure history and vegetations (Ministry of Construction & transportion, 2003)

4. Application

Cell unit evaluation method is applied to the cut slope located by local road No. 70 in Korea. The slope is 80m long and 16m high and dip direction and dip are 32° and 45°. The slope is divided into 40 cells using 5m x 5m square cell. 18 cells are rock cells and 15 are soil cells. 7 cells are mixed cells in which both rock and soil are distributed (Fig. 2). Evaluation systems for rock cell as well as soil cell are performed for mixed cells. Kinematic analysis, risk index analysis and SMR method are performed for rock cells and risk index analysis is done for soil cells.

Kinematic analysis shows that failures are possible in 6 cells (Fig. 3). Contour map for SMR value is shown in Fig. 4. The highest value is 88 in cell number 20 and the lowest value is 6 in cell number 1. 10 cells are evaluated as stability class III and 7 cells are evaluated as stability class IV and V which is the most unstable class. Locations of these cells are almost identical with the cells in which failures were possible in kinematic analysis. Figure 5 shows contour map for risk index and risk indexes range from 0.18 to 0.77. Generally, soil cells have higher values than rock cells.

5. Discussion

Slope stability analysis with present concept is generally carried out for the most critical joint set and support measures are conducted based on this analysis. Joint set 1 in the slope investigated is the most critical one and SMR value is 23 which is class IV. According to the suggested supports by Romana(1985), systematic reinforced shotcrete, toe wall and/or concrete and re-excavation should be carried out on the whole slope immediately. However, cell unit evaluation method shows that 3 cells are classified as class V, 4 cells are in class IV and 18 cells are evaluated as better than class III. With this analysis, heavy support measures are required on only 7 cells region and spot or systematic bolting and spot shotcrete are enough on the other regions.

Conclusions

When we analyze slope stability, we measure characteristics of slope at the most critical region and the values analyzed are assumed as representing the whole slope. However, characteristics of rock mass/soil mass are different from one location to another within a single slope, support measures more than necessary are usually carried out. We suggest cell unit evaluation method to solve this problem and applied that method to the cut slope. Slope stability by new method shows quite different result from those by present concept. Because this method visualizes the variation of slope stability very precisely, proper support measures can be carried out on the only regions necessary. Risk index is very useful to visualize the stability of rock mass and soil mass in one contour map.

References


Morphometric Analysis of Landslide-prone Areas in Japan Using LiDAR-derived DEMs

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Abstract. Detailed contour maps/visualized images produced from LiDAR-derived DEMs are powerful tools to locate landslide areas, thereby improving the quality of landslide inventories. However, information of not only locations but also of slope types and activities are wanted in constructing/maintaining infrastructure to mitigate landslide disasters. Since LiDAR measurement can extract the fine surface features associated with slope activities, the DEMs can also be utilized to provide this information. In this study LiDAR-derived DEMs are analyzed to quantify surface features of colluvial and landslide blocks. Two landslide prone areas were selected for analysis, from Kii Peninsula and Fukushima Prefecture in Japan. In the former Mesozoic sedimentary bedrocks are tectonically sheared, while Tertiary tuff dominates in the latter. DEM grid sizes used were 1-m and 2-m, respectively. The eigenvalue ratio, a filter to indicate three dimensional surface roughness, and slope angle were used to characterize slope morphometry. Field investigations were also carried out to examine the relations between filter values and actual surface features, block types, and recent slope activities. Results showed that colluvial blocks were characterized by slope angles that clustered about the angle of repose. In the Kii Peninsula site, a high proportion of eigenvalue ratios between 2.75 and 4 indicated the presence of colluvial slope deposits. In both the study sites, recently active slides were steeper than surrounding areas and contained a higher proportion of cells with eigenvalue ratios < 2.5 (scars and bedrock exposure). The Fukushima site also comprised an active mature slide with an undulating surface. Here, there were a high proportion of cells with an eigenvalue ratio of 4-5.

In this study the DEM-derived eigenvalue ratio and slope angles could characterize slope types, features and their activities. However, the relationships between filter values and field conditions were not the same between the sites, because of differences in the dominant hillslope processes, and cell-size and resolution of DEMs. Therefore supplementary field investigations are always necessary in the routine application of these filters to characterize landslides and slope features elsewhere. Furthermore, a number of different roughness filters may be required in addition to the eigenvalue ratio to help fully discriminate between surface features in other land settings.

Keywords. LiDAR-derived DEM, deep seated landslide, eigenvalue ratio, slope angle

1. Introduction

LiDAR measurement can produce detailed contour maps/visualized images representing the fine-scale geomorphic features associated with activities of deep seated landslides, e.g., cracks and internal scars. While they can contribute to locating and outlining landslide areas, land managers also need information about the condition of landslides for constructing and maintaining infrastructure and ultimately to mitigate landslide disasters. In addition, the appearance of colluvial slopes is sometimes similar to landslides, particularly for gentler slopes located under steep scarps. Because such slopes are occasionally misidentified as landslides, despite the different movement mechanisms involved, developing efficient methods to discriminate between them will contribute to effective slope management.

While field observation is usually the best way to identify slope processes, types, and recency of activity, it is often very difficult to undertake over large regions and is time consuming and costly. Here, it is proposed that LiDAR data analysis can help to quantify surface features associated with landslide blocks at various stages of activity and provide objective clues to their development. So far only a few case studies have been undertaken to appraise recent landslide activities using LiDAR derived DEMs and surface roughness filters. The use of such filters is based on the notion that recently active surfaces become rougher following the appearance of fine surface features (Glenn et al. 2006, McKeen and Roering 2004, Kasai et al. 2008). Because slope surfaces are a reflection of the underlying slope processes, these filters should also be able to discriminate landslides from stable colluvial slopes.

In this study, LiDAR-derived DEMs from two study sites are analyzed to find the relationships that exist between grid-cell filter values and local surface features, and between the spatial patterns of those values and slope types of contiguous blocks of land. The results from sites are also compared and contrasted to examine the influence of
deposits (Figs. 1 and 3). In contrast there was no distinctive spatial pattern of eigenvalue ratios in colluvial blocks at the Fukushima site, because the range for colluvial deposits appears to overlap with other feature types (Fig. 1).

4. Results

The relationships presented in Fig. 1 were obtained by comparing eigenvalue ratios and local surface features. Feature types as represented by the range of values, differ between sites.

At both study sites, recently active slides were steeper than colluvial blocks in their proximity (Sk1, Sk2, Sk3, Sk5 and Sf1, Fig. 2) and contained a higher proportion of cells with eigenvalue ratios < 2.5 (scars and cracked bedrock outcrops) (Fig. 3). The Fukushima site also contained a higher proportion of cells with eigenvalue ratios > 4 (colluvial deposits) (Figs. 1 and 3). In contrast there was no distinctive spatial pattern of eigenvalue ratios in colluvial blocks at the Fukushima site, because the range for colluvial deposits appears to overlap with other feature types (Fig. 1).

3. Methods

LiDAR-derived DEMs of both sites were analyzed with two filters, slope angle and the eigenvalue ratio. Slope angle was chosen simply to express surface forms. The angle for a grid point was derived from the elevations of the adjacent 8 DEM points. The eigenvalue ratio is a roughness filter used to express the degree of diversity of direction of normal unit vectors of adjacent grid cells (Woodcock, 1977), and derived for a 3 by 3 cell window in this study. Lower values indicate rougher surfaces.

In addition to DEM analysis, field investigations were carried out to identify the actual surface features and block types (whether landslides or colluvial slopes). The slope blocks used for analysis: colluvial slopes and landslides at various stages of activity, were outlined based on visualized images/contour maps created from the DEMs and field investigations.

2. Study sites

Two landslide prone areas were selected for analysis, from the Kii Peninsula and Fukushima Prefecture in Japan. In the Kii Peninsula site, tectonically sheared Mesozoic sedimentary bedrocks underlie the area. Average slope angle is 36 degrees. Steep (>45 degrees) erosive slopes are concentrated along the Amano River, which runs central to the site. In contrast Tertiary tuff dominates the geology of the Fukushima site. The site is located by the Surigami-dam reservoir, and slope angles are roughly constant from the reservoir edge to the ridge top. Average slope of the area is 32 degrees. The grid sizes of DEMs used for analysis were 1-m for the Kii Peninsula site and 2-m for the Fukushima site.

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Colluvial blocks were characterized by slope angles that clustered about the angle of repose (Fig. 2). Furthermore, in the Kii Peninsula site, a high proportion of eigenvalue ratios between 2.75 and 4 indicated the presence of colluvial slope
5. Conclusion
The results indicated that the DEM-derived eigenvalue ratios and slope angles could help characterize and identify slope features, types, and their activities at both the study sites. The difference between sites with respect to slope activities and landslide features was likely attributable to underlying rock types. These help determine a site sensitivity to land instability. The cell-size and resolution of DEMs also probably influence the relationships between filter values and land surfaces features. The larger cell size and lower resolution of the DEM at the Fukushima site tended to express topographic relief as if being smoother and more level, such that the same type of surface features have a higher range of eigenvalue ratio, than at the Kii Peninsula site. Hence DEM resolution needs to be taken into consideration in this type of analysis.

In future, more LiDAR measurement and surveys can be expected in landslide prone areas as the cost decreases and technology improves. This study suggests that supplementary field investigations will always be necessary in the routine application of these filters to characterize landslides and slope features elsewhere. Furthermore, a number of different roughness filters may be required, in addition to the eigenvalue ratio, to help fully discriminate between surface features in other land settings.

Acknowledgments
Nara Prefecture and Surikamigawa-Dam office of Ministry of Land, Infrastructure and Transport are very much thanked for providing their LiDAR-data.

References
The Preliminary Study on Landslide Prediction Model in Malaysia

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Abstract
In this paper, we build up the landslide prediction model to provide reliable information of the factor of safety (F) value, which may assist the authority on decision making on how to measure disaster prevention and information dissemination in Malaysia. We apply landslide prediction equation, which is proposed by Dr. Hiramatsu. We modify the equation to suit Malaysian environment and build up the Malaysian landslide prediction model. Our model also provides smooth information dissemination system, which gives information to the rescuers on how to access to the disaster area and directing the population at risk to shelter nearby while landslide happening.

Keywords: landslide, modeling, Disaster Prevention and information dissemination

1. Landslides Issues in Malaysia
Landslides are a recurring hazard in Malaysia. The recent landslides at Sandakan, Sabah and Ipoh, Perak in 2007 were recorded and caused huge loss to the society. The landslide at Gua Tempurung in 2004 affecting the north-south highway was not the first and will not be the last, and it had already happened before close to the very same site in 1994. The collapse of the apartment block Highland Towers at Bukit Antarabangsa in 1993 was a major tragedy with many lives lost, and the same site suffered another landslide a few years later in 2002, fortunately on a much smaller scale. Other incidents include the collapse of the hillside housing development in Gombak (2004), the highway rockslide at Bukit Lanjan in Selangor (2003) and the house-sized boulder rockslide at the highly populated Majestic Heights in Paya Terubong in Penang (2003). Thus, it is very clear that landslides and other ground failures impose many direct and indirect costs to society.

Direct costs include lost of life, the actual damage sustained by buildings and property, ranging from the expense of cleanup and repair to replacement. Indirect costs are harder to measure and include business disruption, loss of tax revenues, reduced property values, loss of productivity, losses in tourism, and losses from litigation. The indirect costs often exceed the direct costs. Much of the economic loss is borne by Federal, State, and local agencies that are responsible for disaster assistance and highway maintenance and repair.

Thus, study on landslide prediction model in Malaysia will certainly give a much better insight into disaster mitigation involving landslides. Better still, with better knowledge on the roles of rainfall, ground conditions, roots and vegetation, some landslides may even be prevented or avoided altogether.

2. Introduction to Landslide Prediction Model
Normally, landslide occurs because of earthquake, volcano activities and rainfall. In Malaysia, heavy rainfall is the cause of landslide. In our study, we apply infinite slope stability landslide prediction model, which is proposed by Dr Hiramatsu, 1992 as in Figure 1, for slopes in Malaysia. We collect the data from sites in Penang, Malaysia and simulate the model through PC based simulation system (Landslide Prediction Simulation System (LPSS)). From the simulation result and site monitoring, we may study on the differences, and modify the equation for better landslide prediction in Malaysia. Furthermore, we may develop landslide prediction model, for the cut slope along the highways and roads, which is suitable in Malaysia climate and environment. We set a site in Teluk Bahang, Penang to monitor the cut slope for obtaining data time by time.

Figure 1. The Stability of The Infinite Slope

The below equation can represent the stability of the infinite slope in Figure 1.

\[ F = \frac{\tau}{\tau_x} = \frac{\frac{c}{\cos \theta} + \left( \gamma_l - \gamma_w \right) \cdot H_1 \cdot \cos \theta \cdot \tan \theta}{\left( \gamma_l + \left( \gamma_w - \gamma_l \right) \cdot H_1 \right) \cdot \sin \theta} \]

where H is soil height, H1 is underground water table, \( \theta \) is slope degree, \( \tau \) is resistance force of the slope, c is sliding force, \( \gamma_l \) = cohesion (soil sticky force), \( \gamma_w \) = internal friction.
angle, $\gamma_s$ = soil unit weight, $\gamma_t$ = saturated soil unit weight and $\gamma_b$ = soil unit weight in water.

Here, by the equation, if this F value is greater than 1, the slope is considered stable. We consider the flow of water in the slope as the below hydrological equation:

$$
\lambda \cdot \frac{\partial h}{\partial t} + \frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} = q_s
$$

$q_x = h \cdot K \cdot I_x$

$q_y = h \cdot K \cdot I_y$

where $h$ = water table, $K$ = saturated hydraulic conductivity, $I_i$ =hydraulic gradient in i direction(i = x, y), $\lambda$ = effective porosity / valid porosity, $q_i$ = the flow of water in i direction(i = x, y). From the above equations, we created Landslide Prediction Simulation System as below.

3. Landslide Prediction Simulation System

In Landslide Prediction Simulation System(LPSS), there are Map Simulator and File Simulator. In Map Simulator, a chosen simulation can be performed from the maps, which have been already set in the system by using Map Editor as in Figure 3. In Map Editor, we can load map and select the area from where we collected the data. Upon the map selection, the selected studied site will be shown and we can input the collected parameters. Figure 4 shows the input of parameters and rainfall volume in certain duration into LPSS. The result of the simulation for the selected map and the changes of F value time by time can also be analyzed by LPSS as in Figure 5.
Besides simulation through Map Simulator, the system also provides File Simulator, which we can simulate in batch file form and save the input data into batch file, as shown in Figure 6.

![Figure 6. Input data in batch file](image)

We input sets of data with different value of certain parameters to study the behavior of parameters from the output graph. From the result, we can also study and analyze the relations between outputs and parameters. Figure 7 shows the result of File Simulator.

![Figure 7. Sample output of simulation by batch file](image)

When the factor of safety value $F$ is less than 1, the possibility of the slope failure is high and the system will automatically assume that there is a road cut off and indicate the alternative path on how to access to the failure slope site as shown in Figure 8.

![Figure 8. Rerouting the access path to the failure slope](image)

Furthermore, LPSS links to the warning system. The warning system will alert and guide the folks to safety area and shelter nearby as shown in Figure 9, after receiving warning message from LPSS.

![Figure 9. Warning system](image)

4. Further Enhancement and Discussion

In our further study on landslides, LPSS will be enhanced to simulate wide area and cut slope along roadside and highway. Remote sensing method to predict the possibilities of landslide in huge area is one of our future studies to determine the risk area to provide information to folks who stay near to risk area. In future, sending warning message to certain authorized person and decision makers via SMS(Short Message Service) and email for them to decision making on releasing emergency alert to the public is one of LPSS function in future.

5. Conclusions

We have created Landslide Prediction Simulation System (LPSS) to predict the stability of natural slope in rainfall duration as the first step. From LPSS, we can obtain various results by input various parameters and predict the stability of slope for certain volume of rain in certain period. However, LPSS only focuses on natural slope prediction in rural area, Malaysia.

In our next study, we may develop simulation system on cut slope in tropical residual soils. By integrating natural slope and cut slope simulation system, LPSS will become a useful decision making tool for authority to send out warning message to folks.

6. Acknowledgement

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Rockfall Hazard and Hisk for a High Promontory: Monemvasia Historical Site, Greece.

Paul Marinos (NTUA, Greece), George Tsiambaos (NTUA, Greece), Haralambos Saroglou (NTUA, Greece), Vassilios Marinos (NTUA, Greece)

Abstract. The paper presents the kinematics of rock instability of a high promontory, where Monemvasia historical site is situated, in the Peloponnesian peninsula. The instability phenomena pose a significant threat on the town situated immediately down slope. Rock fall episodes occurred in the past, whereas, due to the relaxation of the high cliff, significant undermining of the castle frontiers has been observed at the slope crest. The predominant types of kinematic instability are of planar or wedge failure and toppling of large blocks. In order to investigate the existing stability conditions and decide upon the protection measures, stability and rockfall analysis were carried out for numerous slope sections under different loading conditions. A rock-fall risk rating system is proposed, which is based on morphological, lithological and structural criteria and on vulnerability. The rating system is applied for individual sections along the slope and a risk map was produced, which depicted areas having different degree of risk against rockfall occurrences. Protection measures were designed based on the stability analysis as well as the hazard map. Moreover, an assessment of the residual risk which could remain after the stabilization would be made by taking into account additional special criteria.

Keywords. Limestone, rock-fall protection, risk map, historical site.

1. Introduction

The impact of rockfalls on archaeological sites and historical monuments in the Greek territory is significant, since most of the landscapes are mountainous and the sites are usually founded near or on steep rock slopes. Geotechnical problems related to slope instability and protection of historical sites in Greece have been addressed by several authors (Marinos and Koukis, 1988) and recently by Marinos et al. (2002) and Marinos and Rondoyanni (2005). The hazard of rock falls is higher in areas with intense seismic activity, where earthquakes are the principal triggering factor (Marinos and Tsiambaos, 2002). The archaeological site of Monemvasia in South Peloponnese, consists of a historic city situated at the foot of a 100 m high limestone rock cliff and an ancient and medieval city and castle at the slope crest (Fig. 1). The site is a typical example with high impact of rockfalls. The lower city is inhabited and attracts many visitors under a high risk. Rock falls existed long before the development of the city in the ancient time, as evidenced by the foundation of several ancient structures on large fallen blocks of rock as well as the abundance of rock fragments on the slope foot.

2. Engineering geological conditions

The geological formations encountered in the area consist of Jurassic bedded, dolomitic limestones and Cretaceous unstratified massif limestones. The rock slope overhanging the historical city consists of the Cretaceous limestone. Two major fault zones, with E-W and NE-SW strike, intersect the formation respectively, forming the horst of the promontory.

The limestone rock mass is moderately fractured, intersected by numerous major vertical fractures, which have a parallel strike to the fault structures forming the rock face. The spacing of the discontinuities is relatively large (more than 1 m); hence the size of the rock blocks is large to very large. In places, mainly due to stress relief the rock mass is very blocky and the size of the rock blocks is smaller, especially on the upper part and the crest of the slope, where the wall of the upper ancient city is founded. The limestone is karstified in places and karstic voids of large dimensions are formed, undermining the rock slope.

Fig. 1 View of Monemvasia historical site.

3. Rockfall analysis – slope stabilization

The stability conditions of the slope are mainly controlled by the following factors: a) due to spacing large blocks are prone to fall, b) lack of persistency of the discontinuity planes, which results in instabilities only in specific parts of the slope, c) lack of weak zones, which would result in large shear failures, d) the rock face is in a state of relaxation due to the high inclination of the slope.

The rock slope stability analyses were based on the prevailing mode of instability of each potential rock failure. The principal failure type is rockfall due to a sort of toppling, but some planar or wedge failures also exist (Fig.2).

The rock blocks were delineated and their geometry and mass was determined. Due to the inaccessible nature of the slope, the assessment of the above characteristics was based on the geodetic mapping of the rock cliff. These
characteristics were grouped and specific sections (A to H) were formed for separate analysis (Fig. 5).

The surveying and mapping of the high rock cliff was based on a new geodetic methodology (existing geodetic surveying method and its combination by the use of modern reflector-less total stations), which resulted in a three-dimensional Digital Terrestrial Model (DTM) of the ground surface (Lambrou E and Pantazis G., 2006).

The height of the source of potentially to fall blocks is minimum 70 metres above the base of the cliff, while in some places it reaches 100 metres. The size of the unstable blocks, which are more probable to fall, is only between 1.5 m$^3$ to about 4 m$^3$, except the area between Sections A and B (Fig.4 and 5) as well as Section E where the size can go up to 30 m$^3$. In the area shown in Figure 4 the potentially unstable blocks have very large dimensions.

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The analysis was performed using software RocFall of Rocscience Inc (1998). The coefficients of normal and tangential restitution for the limestone were assumed $R_n=0.315$ and $R_t=0.712$ as suggested by Robotham et al, (1995). The initial velocity of the falls was taken equal to 0.48 m/sec due to seismic triggering. An example of the analysis in Section D is shown in Fig.3.

It is evident that the installation of a barrier in locations with adequate catchment area can protect the path and the structures at the foot slope. The maximum impact energy on the barrier in this location (for rock weight of 5 tn) is 360 kJ for a confidence level of 99%.

It should be noted that the width of the zone, just behind the barrier, is very decisive for the impact energy since this portion of the slope provides considerable damping and therefore loss in energy. In some locations there is no space behind the barrier resulting in enhanced impact energy.

Since the weight of the rocks varies, a parametric analysis was carried out for a range of rock block weights (between 0.5 and 10 tn) and for a range of seeder heights (between 70 and 100 m). The total kinetic energy, which is produced by the falling rock blocks, as calculated at the different sections of the slope, does not exceed 750 kJ. For larger blocks the barriers are not adequate.

The necessary support measures can be divided in two categories: a) those which apply an external force on the rock face e.g. tensioned rock anchors, patterned rock bolts, and b) those which offer protection once the rockfall will occur, mainly rock fall barriers.

Other support measures, such as grouting of rock joints with associated drainage, construction of buttresses in overhanging areas and removal of unstable blocks are necessary, but can be inapplicable or very difficult to construct in high rock cliffs.

The application of pattern bolting in the locations where large individual blocks exist (or spot bolting for smaller ones) at the slope crest is adequate. The installation of a steel protection net or sprayed concrete is not acceptable due to the archaeological restrictions of the area.

As mentioned earlier, the scale of some potential failures is such, that no stabilization measures can minimize or withdraw the risk of a potential rock fall after their application. Even high-energy rock fall barriers would prove insufficient in the case that the rock block shown in Fig. 3 detached from the cliff. A possible support solution, in this case, would be to install tensioned wire-rope cables around the rock block to resist its movement and to grout any open discontinuities with appropriate drainage holes. Grouting of joints could prove inapplicable in a permeable limestone rockmass.
4. Rockfall risk assessment

In order to assess rockfall risk, a number of rating systems have been developed. Pritchard et al. (2005) developed a rating methodology, which is applied to predict rockfall risk along railways.

A similar system is the Rockfall Hazard Rating System (RHRS) (Pierson et al., 1990), which is most widely accepted. These systems give a reasonable assessment of the relative hazards due to rockfalls from cut slopes adjacent to highways and railways.

In the present study, a rockfall risk rating system for natural rock slopes is proposed. It defines 19 rating parameters, grouped in 3 major categories on hazard and consequences, which have a different weight factor in the assessment of the total risk. The parameters are presented in Table 1.

The rating system was applied at selected locations along the rock cliff, since the parameter rating differs for each slope segment.

The parameters that vary from one location to another were: a) the size and number of rock blocks, b) the spacing and persistence of discontinuities, c) the seeder height, d) the width of the available catchment zone and e) the existence of structures or human activity at the underlying area. The slope height and angle of Monemvasia slopes don't vary significantly.

The result of the application is a risk zonation of the cliff against rockfall occurrence, presented on the risk map shown in Fig. 5. The map depicts the areas having a high and very high risk due to either increased number of existing unstable blocks or restricted area for their cathcment or combination of both. The risk categories are presented in Table 2.

The slope foot area between sections C to E presents very high risk due to the numerous unstable blocks on the cliff and proximity of structures as well as human activity (stairs to upper city). The area between sections E to Z has medium to high risk, due to the wide cathcment zone at the base, which offers ideal conditions for installation of barriers. The area between sections A to B has very restricted catchment zone and the installation of barriers is rather not possible. However, the impact on the derelict structures in this area is low hence the risk is medium to high.

5. Conclusions

The rock slope stability of the high cliff overhanging the historical site of Monemvasia promontory was studied, based on kinematic analysis of unstable blocks and calculation of their rockfall trajectories. It is evident that in the case of blocks having weights higher than 10 t, even the installation of high capacity rockfall barriers cannot remove the hazard due to impact of falling rocks on structures, either because the impact energy is extremely high or the catchment zone is not sufficient for optimized protection. The application of active support measures would be adequate if the height of the slope was not prohibitive.

Table 1 Parameters of rockfall rating system for natural rock slopes to define risk.

<table>
<thead>
<tr>
<th>Cat. No.</th>
<th>Parameters of risk</th>
<th>Relevance</th>
<th>Weight (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Slope angle, weathering, Discontinuity condition, rock block size, permeability of rock mass and condition of drainage</td>
<td>Hazard</td>
<td>55</td>
</tr>
<tr>
<td>2</td>
<td>Number of potential blocks, number of past rockfalls, rainfall, seismic activity</td>
<td>Additional to hazard</td>
<td>15</td>
</tr>
<tr>
<td>3</td>
<td>Slope height, seeder height, availability and geometry of catchment zone at foot of slope, slope accessibility, potential result of impact and value of structures, pathways</td>
<td>Consequences and impact of hazard</td>
<td>30</td>
</tr>
</tbody>
</table>
Fig. 5 Risk zonation of rock slope based on rockfall rating system.

Table 2 Rockfall risk of natural rock slopes based on the proposed rating system.

<table>
<thead>
<tr>
<th>Cat eg.</th>
<th>Rating</th>
<th>Risk</th>
<th>Protection measures</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>&lt; 20</td>
<td>Very Low</td>
<td>Not necessary</td>
</tr>
<tr>
<td>II</td>
<td>20-40</td>
<td>Low</td>
<td>Occasionally necessary</td>
</tr>
<tr>
<td>III</td>
<td>40-60</td>
<td>Medium</td>
<td>In limited extent</td>
</tr>
<tr>
<td>IV</td>
<td>60-80</td>
<td>High</td>
<td>Combination of active and passive measures is necessary</td>
</tr>
<tr>
<td>V</td>
<td>80-100</td>
<td>Very High</td>
<td>Critical state of stability, Combination of active and passive measures is absolutely necessary</td>
</tr>
</tbody>
</table>

To calculate the potential risk of the rockfalls, a hazard rating system for natural rock slopes is proposed and the locations with maximum residual risk are defined.

Acknowledgments
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References
A New Soil Nail Technology for Protecting Heritage Sites Against Landslide

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Abstract In Japan, the cultural heritage sites often locate close to mountainous area. Most of them are threatened directly or indirectly by natural disasters concerning slope failure, landslide or debris flow. The situation becomes more terrible under influences of frequent occurrence of typhoons and strong earthquakes. A number of shrines, temples, and old castle were damaged, ruined or even vanished. Unfortunately, a common method of slope stabilization usually clears all or a part of the trees, an important factor of a cultural heritage that sometime is considered as sacred symbols and are protected by high spirituality. Thus the traditional methods such as concrete slope, retaining wall…are themselves often harm natural landscape and thus indirectly damage cultural heritage sites. It is necessary to develop a new technology of slope stabilization. The new technology must be considered based on two factors: 1) protection of trees, 2) application in heavy rain and strong earthquake zones. In that context, a new soil nail technology was proposed that can stabilize natural slope while not cutting off the trees and more importantly, it let the tree roots of mother nature play their role in protecting against landslide disaster. This paper shows how Non-frame successfully protecting the heritage sites against slope failure or landslide under the difficult conditions of typhoon and earthquake.

Keywords. Heritage site protection, landslide, natural landscape, soil nail

1. Reinforcement mechanism of natural root and Non-frame

A tree root system distributed from unstable soil layer to stable soil layer or bedrock reinforces the slope by fixing unstable soil layer into stable one. Root reinforcement increases due to the displacement of unstable soil layer (see Figs. 1a and 1b). Non-frame is developed as a simulation of this natural root system (Figs. 2 and 3). The fixed plate is model of stump, soil nail is model of vertical main root and the wire connection is model of horizontal root fibers.

Factor of safety of slope is calculated by the equation 1. Fs is the safety factor of the slope. Rc is the reinforcement of Non-frame in combining with tree root. Rc is calculated from shear force and axial force (Rc = shear force + axial force · tan Φ; Φ is internal friction angle of soil).

\[ F_s = \frac{\text{Resisting force} + \text{Rc}}{\text{Driving force}} \]  \hspace{1cm} (1)

Fig. 2 Distribution of Non-frame on slope

Fig. 3 Structure of Non-frame
2. Slope failure in heritage sites and application of Non-frame:

![Photo 1 Shrine buried by slope failure](image1)

Photo 1 Shrine buried by slope failure (Nagaoka, Niigata Pref.)

There are about 10 typhoons/year and 21% of the world’s strong earthquakes occur in Japan those caused a lot of landslide disasters. In other side, almost heritage sites of Japan locate closely to the mountainous areas where the landslide disaster frequency occurs. There are 13 world heritage sites in Japan, with 8 of them located in forested mountains and other 3 located closely to mountainous areas. The natural slopes of a landscape or cultural heritage site are often covered by very weak and soft topsoil which tree roots can penetrate into. The slope failure or landslides are easily triggered in this vulnerable topsoil. The problem becomes more terrible when typhoon and earthquake occur together. As an example, in Oct. 2004, when the Niigata Prefecture was quaked by Chuetsu earthquake (M6.8) right after typhoon number 0423, it caused 341 slope failures. More than 70% of these slope failures had the depth of slip surface less than 3.0m, the depth of weak topsoil as mentioned above (Fig. 4). Due to the influences of the slope failure, some of shrines, a kind of cultural heritage were destroyed. Photo 1 shows a beautiful shrine that was buried by a shallow landslide. Slope failure also damaged the natural landscape around heritage site. Photo 2 is an illustration of that kind of damage when slope behind a shrine collapsed. In Jul. 2007, other earthquake named Chuetsu Oki earthquake (M6.8) occurred in the same Niigata Prefecture with the epicenter was very close to that of Chuetsu earthquake (2004). It also caused a lot of slope failures and landslides as Chuetsu earthquake did.

2.1 Application of Non-frame in natural landscape sites:

There is Non-frame field in Kuziranami beach that is near the epicenter of both earthquakes Chuetsu and Chuetsu Oki. Photo 3 shows the field with stable green slope on right side that was reinforced by Non-frame and the failure slope on the left side that had no reinforcement.

Photo 4 (a,b and c) focus into part A, the most unstable part of Kuziranami slope in Non-frame reinforcement area. These photos were made in the investigations right after the earthquakes occurred. The slope was stable even though it was affected by two earthquakes.

Photos 5a, b show natural landscape in Izuinatorigulf, a cultural-landscape of Izu peninsula, a famous tourist point. Non-frame stabilized slope and thus protected the national route in upper part as well as protected the prefecture route and railway at foot of slope. Non-frame is completely different to method of cemented slope (Fig. 5a). Izu is one of places that have most frequency typhoon and earthquake in Japan.

Photos 6a and 6b show the Kumano shrine, which was protected successfully by Non-frame even though the shrine was affected by West Off Fukuoka earthquake (March 2005) and rainfall. The slope in the shrine area (Photo 6a) failed. The photos were taken just 2 days after the rainfall and earthquake occurred.

![Fig. 4 Depth of slope failure by heavy rain and Chuetsu earthquake](image2)

Fig. 4 Depth of slope failure by heavy rain and Chuetsu earthquake

![Photo 3 Slope was reinforced by Non-frame in the field near Kuziranami beach](image3)

Photo 3 Slope was reinforced by Non-frame in the field near Kuziranami beach
2.2 Application of Non-frame in cultural heritage site

Matsuyama Castle, a famous cultural heritage built in 1602 located at the top of the mountain (see Photos 7a and 7b). In 2001, the unstable part at lower slope was reinforced to protect the castle at the top, and the buildings at the foot. Photo 7a (CG) shows how is the mountain look like if a concrete frame reinforced the slope. And Photo 7b shows the original landscape of the heritage site is preserved successfully by Non-frame method.

Discussions

The influences of landslide on cultural and landscape heritage sites can be classified by two types of indirect damage and direct damage in the Fig. 5. Hereby, pour design of landslide countermeasure is also considered as a kind of indirect damage. One difficult problem for the designers is how to stabilize slopes while preserving natural landscape of a heritage site. In this issue, the origin of landscape concerning the trees must be considered carefully. The structures such as concrete frame usually cleared off a part or all tree and thus was not always applicable but Non-frame method, a model of tree root was suitable to stabilize slope while protect the trees on slope. Further it successfully worked under hard conditions of heavy rain and strong earthquake.
The First World Landslide Forum, 2008, Tokyo

Fig. 5 Influences of slope failure on the damage of natural landscape and cultural heritage sites

References
Cabinet office of Japan (from 1994 to 1998) Data from Japan meteological agency and USGS
Tendency of Growth of Landslides Amount in the Boundary of XXI Century and System of Geoindicators for the Stage-by-stage Landslide Hazard Warning System in Republic Uzbekistan, Central Asia

Rustam Niyazov (Institute Hydroengeo, Uzbekistan)

Abstract. Mountain territories of Republic of Uzbekistan are susceptible to hazardous geological processes (landslides, rockfalls, landslides - streams, suffosion failures). Landslide service in Uzbekistan exists 50 years, for the given period of time since 1958 up to 2008 was fixed 8362 cases of displacement of various types of landslides. Thus, at the end of XX century (1990-2000), in the beginning of XXI century (2001-2008) the amount of landslides for one decade has increased in 2-4 times. Tendencies of growth of amount of landslides in boundary of XXI century is connected with increase of frequency of change of low and rich in water years and increase of number of days with intensive precipitation are considered. Features of activization of secondary new landslides in borders of ancient residual seismic gravitational deformations are emphasized. The structure of monitoring of spatial and local supervision of quickly proceeding landslides in loess soils is analyzed. Types and the kinds of geoindicators recommended for stage-by-stage system of the warning landslide hazard are offered.

Keywords. Geoindicators, monitoring, early warning

1. The mechanism of influence of climate change on growth of amount of landslides on boundary of XXI century

Supervision over hazardous geological processes during 50 years have allowed to reveal some tendencies of growth of amount of landslides last years.

The first - since 1990 in a climate of a mountain part of republic is observed the big contrast - the number of years with the large and small amount of precipitation has increased. The analysis of amount of an atmospheric precipitation for the period from 1940 up to 2008 at conditional border more than 1000 as the characteristic of the damp period and less - dry, it is possible to note than 600 mm / year, that for last 20 years of 7 times were damp and 4 times dry periods. While the previous 20 years (1964-1986) 1 damp year and 3 dry, and for the period of 1943-1966 - 3 damp and one dry(fig.1,2). In connection with that the periods of change from low to rich in water years in 1990-2007 have become frequent, the amount landslide displacement for last 18 years has increased have reached 57 % from total (8362) fixed for last 50 years.

The second - the sum of an atmospheric precipitation of previous autumn-winter period since November till February (XI-II) began to exceed 550 mm (fig.3). In March, April rains with 30-40 mm, within several hours, intensity 8-12 mm / hour more often drop out. Cases when the size of rains for 2-3 day was equal to 90-110 mm have increased. As a whole, the amount of liquid deposits as storm rains raises. Critical situations of mass display of various types of landslide displacement began to arise. In 2005 from March, 12 till March, 19 it is fixed 191 case of their display, these are 40 % from all fixed in this year.

Fig. 1. Atmospheric precipitation for the period 1940 – 2005 meteostation Charvak

Fig. 2. Changes of damp and dry years for 20 years periods time from 1943 up to 2005.
The analysis of the given various types of displacement has shown, that gravitational movement of soils is equal to landslides, landslide cracks - 70 %, collapses, rockfalls - 20 %, karst-suffosion, settlement deformations of buildings - 10 %. On the area of distribution, most mass occur in Tashkent region - from 50 up to 67 %, then in Kashkadarya - from 14 up to 20 %, Surkhan-Darya - 8-13 % and in 4-7 % Samarkand and Dzhizak and in all Fergana valley. Fine displacement in volume up to 1 thousand m$^3$ in various years is equal to 60-80 %, large - 5-10 %. On time it is the most active (80 %) they are occurs in March, April. Natural-technogenous landslides most actively occur lengthways auto-and railways, in zones of mining enterprises. The most representative numbers of frequency of display of large landslides for the period from 1962 up to 2007 are observed in Tashkent region in Bostanlyk and Angren stations of monitoring.

In Bostanlyk zone the basic amount of landslides has taken place in water reach years 1969, 1994, 2005 described by high and very high risk when annually occured from 90 up to 212 cases of displacement of soils with volume from several tens thousand m$^3$ up to several million m$^3$. The greatest amount of years from 10 up to 50 cases of displacement and less than 10 cases (fig. 4). The increase or reduction of frequency of landslide occurrences, basically, is connected with climatic situation, and changes of this tendency in time is not observed. In Akhangaran zone for the given period 418 large landslides (fig. 5) from which 285, or 67 %, have taken place for the period from 1991 up to 2007 are fixed. This tendency is connected, first of all, to increase of technogenous landslides caused by the mining industry, operation of linear and water-exploration constructions.

The largest works for last 50 years on extraction are carried out in Angren coal mine, there 15 landslides with volume from 400 thousand m$^3$ up to 800 million m$^3$ were formed. Largest of them - Bagaran 1950 (volume 400 thousand m$^3$), Turkski 1954 (10 million m$^3$), Zagansanski 1958 (20 million m$^3$), Verhneturkski 1954 (20 million m$^3$), Atchinski 1972 (800 million m$^3$), Central 1985 (58 million m$^3$), Old substation 1993 (3,5 million m$^3$), Naugarzanski 1995 (22 million m$^3$). The general size of displacement on Zagasan-Atchincki landslide was equal to 35 m, Turk - Naugarzan - 40-80 m, Verhneturkski - 30-33 m and Central - 17-60 m. The main feature of landslides is that, despite of the long period of development, the large distance of moving, they year after year continue to move and become less predicted.

2. Tendencies of growth of amount of secondary landslides in borders of ancient residual seismic gravitational deformations. Change of climatic conditions in a mountain zone has caused activization (80-90 %) of modern landslides as the secondary displacement shown in borders of ancient landslide circuses, therefore in a place of display they concern to inherited slope processes. Residual forms of the mentioned ancient landslides are characterized by stability of scales and forms. These are landslides of complex structure with steady relief inside a circus and presence of stagnant lakes. In a relief of a slope they are allocated on reorganization of system of
water-currents, traces of sudden changes as ledges, sharp turns.
In ancient landslide cirques uniform sustained constant water bearing horizon it was not kept, and there are local sites of temporal water bearing horizon where the area of feed and unloading of ground water are located by line. Ancient landslide cirques change the form of a slope, expand the area of reservoir and concentrate in due course a superficial drain of rains. In result processes of ravine erosion, undermining of the basis of a slope amplify. Sites of an output of underground waters is changed, created corking and the underground stream increases the hydraulic gradient. On canyons where the area of a reservoir changes, the charge of water in the spring period within 2-3 hours is increased from 10-12 l/sec up to 180-200 l/sec, and in some cases up to 5 m³/ sec, that constantly increases undermining the basis of a slope and development of repeated landslide displacement. Secondary landslides began to be formed in 30-35 years in zone of landslide cirques of 1969. The problem of inheritance places of display of modern landslides with ancient first of all is connected to character of new watering conditions already before the broken slopes. Second, erosive processes have amplified. Very much frequently places of undermining of the basis of a slope coincide with a place of basis of ravines then development of landslides is sharply accelerated.

Classical example of formation of landslides at intensive undermining the basis of slope is the site - Pustinlik 2004. The landslide of block type with volume 1.0 million m³ was formed March, 19, 2004 on the left board of small river Pustinliksay. Displacement of loessial soils with thickness 15-25 m has taken place on sandy-argillaceous soils of Neogen age. Length of landslide 208 m, width 230 m, height of slope - 80 m, steepness 20-25°, it is convex - concave form of a slope. The cause of formation of landslide - cutting the bases of slope in height 12-14m on an extent 30m and moving river channel aside slope on distance of 20-25 m (fig. 6). Movement of landslide began nearly at 15 o’clock and continued 1,5-2.0 hour (from words of the local resident). Development of landslide occurred from below upwards, general size of moving in the central part has reached 35-45 m. Originally in 1993 the crack with extent up to 200 m and amplitude of 0,3-0,5 m was formed. Two days prior to displacement in March, 17, 2004 has dropped out 32 mm / day. The charge of water in river these days has made 200 l/sec, when usual is 40-50 l/sec. After displacement on landslide in the top and middle zones 5 springs appeared with the charge from 0,3 l/sec up to 0,8 l/sec each. In place of crinkle of channel where there was greatest undermining the basis of slope the ravine developed. In result the landslide temporarily has blocked river channel at distance of 40 m, height 3-5 m, has destroyed 200 m of motorway.

3. Geoindicators for a stage-by-stage early warning system for landslide hazards
In Uzbekistan the early warning system of hazardous geological processes (HGP) is part of the state Policy. The State system of early warning and actions in emergency situations is created. Coordination of State System is assigned to Ministry of Emergency Situations. Parliament accepts the law on protection of the population and territory from emergency situations of natural and technogenous character. Only for years of independence by State for
mountain settlements living in landslide hazard zones were built new settlements where were moved 1,5 thousands of families.

<table>
<thead>
<tr>
<th>Types</th>
<th>Kinds of geoindicators</th>
</tr>
</thead>
<tbody>
<tr>
<td>Estimation</td>
<td></td>
</tr>
<tr>
<td>State</td>
<td>The sum of deposits of the winter - spring period (IX, XII, I, II) 500-600 mm height of a snow cover</td>
</tr>
<tr>
<td>Impact</td>
<td>Intensity of snow melting Temperature of air and ground. High-altitude borders of a snow cover. Occurrence of new temporal springs</td>
</tr>
<tr>
<td>Earthquakes</td>
<td>The sum of an atmospheric precipitation for: 10 days 2-3 days - 70-110 mm 1 day - 35-40 mm</td>
</tr>
<tr>
<td>Technogenous</td>
<td>Intensity of precipitation 14-16 mm / hour. Occurrence of muddy water in springs Increase of the charge of springs in 3-5 times Rise of ground water level on 2-3 m Increase of pore pressure up to 2,4 MPa</td>
</tr>
<tr>
<td>Dynamic impacts</td>
<td>Formation of new cracks, change of the form, amplitude and lengths of existing cracks, increase of speed of horizontal movement &gt; 50 mm / day</td>
</tr>
<tr>
<td>Response</td>
<td>Direction of mud stream, range, area of distribution Thickness of blockage, probable volume of water in blocked lake prospective character of washout of blockage. Height of soils blow on an opposite board</td>
</tr>
</tbody>
</table>

Due to constant supervision over increase of charges in springs and occurrence of muddy water, under tendencies of raising ground water level, expansion and lengthening of cracks, increase of speed of movement, monitoring team of State Committee of Geology inhabitants of 77 facilities from 12 settlements have been in due time warned which were filled up or destroyed by landslides. There were also tragic cases connected to delayed revealing places of formation of new landslide sites, inexact definition of zones of distribution of landslides, change of direction and range of distribution of landslides - streams. In this regard in Uzbekistan and other Central Asian republics the system of monitoring of hazardous geological processes consists from spatial and local supervision. Spatial local complex supervision on basic stationary sites are directed on revealing of new landslide hazardous zones, and system of monitoring consists of supervision, estimation, control for the warning. The overall objective - on the basis of threshold levels of estimated and controllable geoindicators to develop a stage-by-stage early warning system. The geoindicator - is considered as previous part in the engineering - geological information - the index, harbinger, barometer, allowing to feel the tendency, which else it is impossible to reveal. For the warning we are conducting three types of indicators - conditions, influences and reactions which are subdivided in system of monitoring of processes on estimated and controllable. For the warning it is recommended to use seven kinds of geoindicators - an estimation of the sum of deposits of previous period, the daily control of intensity of snow melting and rains, change of the charge and a level of ground waters behind tendencies of development of cracks, undermining of the basis of slope and increase of speed of displacement of rocks.

Stage-by-stage system of the warning consists of 3 stages. The first stage - characterizes an estimation of a climatic condition of the autumn - winter period and the control of intensity of snow melting in the beginning of the spring period. The second - the control of rains and displacement of landslides in the spring period. The third - the control of an atmospheric precipitation and change of the charge of ground waters at the end of spring and the beginning of summer.

On the basis of long-term supervision threshold levels are offered to geoindicators for the warning about landslide hazard. Many of these parameters have confirmation on the large number of cases, others as dynamic fluctuations of measurement of pore pressure are characterized by single measurements and demand the further specification. Set and sequence of these parameters is used by service of monitoring for the early warning of local authorities and inhabitants of mountain settlements.

References
Case Study of Estimation of Financial Loss by Landslide Disaster

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Abstract. Intensive rainfall and recent land development are major causes of landslides. If roads, railways, or other infrastructure are damaged by a landslide, traffic is blocked, impacting regional social and economic life. Occurrence of landslides not only induces direct damage to property and roads but also causes emotional distress, economic losses and harmful rumor for the area. This indirect damage tends to last long in proportion to time required to stabilize the slope, although the actual conditions of those damages have not been investigated well until today.

In this study, the authors introduce the case study from Miyagi Prefecture in Japan, where a regional road was closed due to the damage caused by a landslide in February 2007. This road was frequented by local residents and for transportation. Detour took extra 86 minutes at maximum to travel, and the time differed depending on traffic origin and destination. Road was closed for 44 days in total causing inconvenience for local residents and damaging to tourist industry because of decrease in the number of visitors.

The authors estimated total monetary loss caused by the road closure, concerning extra traveling time and cost occurred by detouring, traffic volume and the number of tourists, to be found approximately 500 million yen was lost for the 44 days. Since the disaster occurred in a busy sightseeing season, the amount was also expected to be higher than other time of a year. As above, road closure by the occurrence of a landslide was considered to have caused significant financial loss in the area. To reduce the loss appropriate emergency measure is to be applied as soon as possible with understanding of landslide behavior.

Keywords. Landslide, Social impact, damage, Road closure

1. Background

A lot of landslide disasters occur every year in Japan, 216 landslide disasters occurred in 2006 (Sabo Department of the Ministry of Land, Infrastructure and Transport, 2008). Also intensive rainfall and recent land development are major causes of landslides. If roads, railways, or other infrastructure are damaged by a landslide, traffic is blocked, impacting regional social and economic life. In particular, indirect damage tends to last long in proportion to time required to stabilize the slope.

To mitigate direct and indirect damages by landslide disaster, it needs effective measures against the risk that was estimated from the probability of sediment-related disasters and the estimated damage by the disasters. However, the actual conditions of those damages have not been investigated well, and the method for evaluation of those damages has not been defined well until today.
3. The calculation of monetary loss by landslide

There is no structural damage except for covering the road by soil collapse. On the other hand, road closure has lasted for 44 days by landslide disaster. Large-sized vehicle travelable detour wasn’t near the landslide site.

Route 108 is frequently used by local residents in Onikoube district (population is approximately 1,500 persons) and for transportation from Akita prefecture to the area on the Pacific side (Fujisawa et al. 2007). This area is sightseeing place including hot springs, ski resorts and hotels. Therefore, the daily lives of local residents and local economy were significantly affected indirect damage by landslide disaster.

3.1. Methods

The authors estimated monetary loss including direct and indirect damage by landslide disaster.

(1) Direct damage (Road damage)

Road damage was estimated as restoration cost with following expression (Sabo Department of the Ministry of Construction, 2000).

\[
\text{Road damage} = \text{Length of road damage (m)} \times \text{Restoration cost per meter}
\]

(2) Indirect damage

If a number of indirect damage were evaluated individually, there is a possibility that these monetary losses were double counted. To avoid double counting these losses, the authors estimated total monetary loss caused by the road closure, considering extra traveling time and cost occurred by detouring (Kohashi et al. 2004).

One-day indirect damage = One-day traffic volume in normal time \( \times \) Extra traveling cost
Extra traveling cost consists of loss of time value and traveling cost.

3.2. Results

(1) Direct damage (Road damage)

Length of road damage is 35m based on width of the landslide, if the landslide collapsed. Restoration cost of national highway is 103 thousand yen per meter.

\[
\text{Road damage} = 35m \times 103,000 \text{ yen} = 3,605,000 \text{ yen}
\]

However if the road was caused damage by the landslide, emergency response will be needed to stabilize the slope for restoring road. So it is important to consider not only restoration cost but also cost of emergency response for evaluating direct damage by landslide disaster.

(2) Indirect damage

The authors estimated total monetary loss caused by the road closure for the 44 days. Large-sized vehicle went through prefectural road as a detour. But actually, there was more monetary loss for the following reason that the prefectural road is not a good road condition like narrow and road surface freezing at night time. If there is more detailed data such as traffic volume change after occurring landslide disaster, evaluation of monetary loss can be calculated to fit the actual condition.

![Fig. 3 Detour route and four trips](image)

Table 1: Change of traveling time and distance due to detour

<table>
<thead>
<tr>
<th>Trip</th>
<th>Traffic volume (vehicle/ day)</th>
<th>Extra traveling time (minute)</th>
<th>Extra traveling distance (km)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Weekday</td>
<td>Holiday</td>
<td></td>
</tr>
<tr>
<td>I</td>
<td>1,434</td>
<td>1,714</td>
<td>12</td>
</tr>
<tr>
<td>II</td>
<td>491</td>
<td>115</td>
<td>42</td>
</tr>
<tr>
<td>III</td>
<td>800</td>
<td>800</td>
<td>86</td>
</tr>
<tr>
<td>IV</td>
<td>1,129</td>
<td>1,999</td>
<td>34</td>
</tr>
</tbody>
</table>

Table 2: Detour loss per day

<table>
<thead>
<tr>
<th>Trip</th>
<th>Loss of time value (yen/ day)</th>
<th>Loss of traveling cost (yen/ day)</th>
<th>Total loss (yen/ day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>1,388,227</td>
<td>115,038</td>
<td>1,503,265</td>
</tr>
<tr>
<td>II</td>
<td>1,549,701</td>
<td>308,425</td>
<td>1,858,126</td>
</tr>
<tr>
<td>III</td>
<td>4,496,693</td>
<td>378,400</td>
<td>4,875,093</td>
</tr>
<tr>
<td>IV</td>
<td>3,618,603</td>
<td>397,851</td>
<td>4,016,454</td>
</tr>
<tr>
<td>Total</td>
<td>11,053,224</td>
<td>1,199,714</td>
<td>12,252,938</td>
</tr>
</tbody>
</table>

Conclusions

In spite of little direct damage by a landslide, the road closure by the landslide was considered to have caused significant economical loss in the area. Total monetary loss caused by the road closure for the 44 days was approximately 500 million yen. So, the reality of various damage caused by landslide will get to be known with collection similar case studies.

To mitigate the social impacts of landslide, appropriate emergency measure is to be applied as soon as possible with the good understanding of landslide behavior, and the experience of dealing with risks should be fed back to preliminary risk control.
Acknowledgments
Miyagi Prefecture is very much thanked for providing their some data and information.

References
An Efficiency Test for Ground Behavior Sensing of Optic Fiber Sensor by a Scale Model Test

Kwon O-Il · Baek Yong (Korea Institute of Construction Technology, Korea)

Abstract. The purpose of this paper is to observe the mode of sensor of sensing the ground breakdown which is occurred by compulsory rainfall simulation conducted through scale model test in order to evaluate the efficiency of the optic fiber which is currently under development and to utilize the result of the efficiency test as reference data for the development of final measuring sensor system with high reliability. In conducting the efficiency test, rainfall simulation was realized to induce the breakdown of the sediment slope of model soil bin first, the test on the sensing capability of optic fiber sensor was conducted by measuring the process of the occurrence of displacement using measuring sensor, and then problems of the optic fiber sensor was analyzed.

To conduct the scale model test, optic fiber sensor package for sensing short circuit was produced with the reduction in size to fit for soil bin. And mini-OTDR was used as an equipment to measure the reduction of light source by the occurrence of displacement of optic fiber. To conduct comparative analysis on the mode of the occurrence of slope behavior with the measuring data for displacement, equipments such as gap water-pressure meter and data logger were installed as the equipments to measure gap water-pressure in the model soil bin.

From the time when approximately 10 minutes were lapsed after generating rainfall simulation with the intensity of 200mm/hr, it was observed by the naked eye that displacement was occurred from the surface. The result of analysis of the data obtained through gap water-pressure meter revealed that rapid change in the measured value was made from the time when 12 minutes were passed and breakdown could be forecasted. Based on the data from the optic fiber sensor, it is judged that the efficiency for sensing displacement is superior in general as behaviors were measured from the time of 6 minutes after the commencement of the test. It is expected that optic fiber sensor for the sensing of short circuit will be useful for the observation of slope behavior.

In order to forecast the breakdown of slope in advance through the sensing of behavior of the slope, data logger system which enables to acquire necessary data on real time basis should be developed and a study will have to be performed on the preparation of the evaluation method to analyze the acquired data on real time basis.

Keywords. Slope, ground behavior, model test, rainfall simulation, optic fiber sensor

1. Introduction

Over 3 decades in the past, the economic growth of our country has emphasized quantitative growth focusing on the introduction of technology from the advanced foreign countries and production, and, due to the increase in the deterioration of major social infrastructures of the country which were established together with the economic growth, socio-economic losses continue to incur. In relation to rapid deterioration of major social infrastructures, it is necessary to establish overall safety management network for major facilities which enables to forecast the possible damages in advance and to serve and manage by compiling site data, establishing database, extracting information and conducting accurate analysis and evaluation to return the result of research to society through the implementation of research and development which reflected the social requirements of securing safety of major social infrastructures of the country.

In this regard, the trend of latest researches in the pertinent field is to minimize the damages caused by natural disaster by detecting the signs of the disaster in advance through the examination of slope breakdown mechanism under domestic conditions and simultaneously to provide crucial data for the selection of effective measures.

Technologies for measuring the displacement of cutting slope are being attempted in many aspects. Existing technology uses extensometer which uses wire or ground displacement meter which uses clinometer in many cases but there are problems to some extent to apply the existing technology to cutting slope by section considering the characteristics of current roads of our country as the existing technology is simple measuring system to apply to individual cutting slopes separately. Accordingly it is necessary to develop the technology with which the section with falling rocks or a certain section with poor ground characteristics can be effectively managed. Optic fiber has advantages of low material cost and superior durability as well as the constant measuring precision even in the case of long distance installation. Using such characteristics of optic fiber, the development of short circuit detection sensor system is now in progress.

The purpose of this paper is to observe the mode of sensor of sensing the ground breakdown which is occurred by compulsory rainfall simulation conducted through scale model test in order to evaluate the efficiency of the optic fiber which is currently under development and to utilize the result of the efficiency test as reference data for the development of final measuring sensor system with high reliability.

2. Optic Fiber Short Circuit Detection Sensor

Optic fiber is optical fiber in which light passing through the glass in the central part is totally reflected using the glass with high refractive index called Core in the central part while using glass with low refractive index called Cladding.

Optic fiber has the advantage that it has very little energy loss and thus has low loss rate of data in transmission and reception and also almost no influence by electronic wave or other external factors. As optic fiber has, however, weakness
in sudden deflection and is very weak for big flexural strength, it has shortfalls that the intensity of signal transmitted through optic fiber gets low depending on the degree of deflection if the optic fiber is not well protected with jacket and in some cases optic fiber may be cut by very big flexural strength. Optic fiber sensor for the detection of short circuit is a system which utilized the shortfalls of optic fiber and is a monitoring sensor system which can identify the situation of a site by measuring the data value from the sensor package installed in the ground structure such as cutting slope through measuring the level of attenuation of signal depending on the degree of deflection or cutting of the optic fiber installed on the site.

When stable and fixed data is transmitted to the system from the sensor without showing any problem in the optic fiber installed on cutting slope, the date is measured as the data for which voltage level transmitted to optic signal receiver is stabilized. In case, however, there is any abnormal phenomena occurred in cable such as the deflection of optic fiber due to slip in the slope or the optic fiber being cut due to the breakdown of slope, voltage level received by the optic signal receiver will be lowered or signal is not received and thus the situation of the site where optic fiber is installed can be easily monitored.

Whole configuration of the optic fiber sensor for detecting short circuit is comprised of Pigtailed Laser Diode light source, Light Source part with controller for driving the light source, Module for the reception of optic signal received by respective channels, Signal Processing Module for processing the received data, Display System and Measuring Software. This paper deals with the issues in the stage of evaluating the efficiency of ground behavior monitoring of the whole system which is now in the process of development.

In order to conduct the test, optic fiber for sensing is fixed using the sensor package which was manufactured on the cutting slope. Optic fiber is made to maintain the Loop shape with a certain diameter in the package, and the cable maintains the tension for next package thanks to the tension of spring. When there is slip or breakdown at a certain part of slope, the tension applied to the optic fiber is changed and thus the optic fiber is deflected to have more acute angle which generates light. The principle of the sensor is to detect ground behavior by converting the loss of transmission of the light generated at that time (Fig. 1).

3. Scale Model Test
3.1 Configuration of Scale Model Test Equipment Scale Model Test

The model test equipment used for this research is comprised of soil bin produced to analyze the occurrence of landslide and the behavioral characteristics of sliding material, rainfall simulation equipment and measuring equipment which includes a package for fixing optic fiber produced in reduced size to fit for the model test, which were all produced by Byung-Gon Chae et al (2006).

Soil bin has the dimensions for inner soil filling part of total length of 2.3m, height of 0.5m and width of 0.3m and its slope can be adjusted up to maximum 40°. Rainfall simulation equipment is comprised of sprinkling device part, rainfall control device part and water tank part. Sprinkling device part is composed of two levels with the nozzles of 2.0mm and 1.5mm diameter respectively in order to imitate diversified natural rainfall types. The intensity of rainfall can be adjusted within the range of 100 ~ 1,000mm/hr so that the influence of intensive rainfall can be sufficiently considered (Won-Young Kim et al, 2005).

For measuring device, the optic fiber sensor package for detecting short circuit was manufactured in reduced size for model test purpose (Fig. 2). And mini-OTDR was used as an equipment to measure the reduction of light source based on the occurrence of displacement of optic fiber (Fig. 3). For the analysis of the mode of slope behavior occurrence in comparison with measured data for displacement, gap water-pressure meter for measuring the gap water-pressure within the model soil bin and other data logger were installed.
3.2 Test Method and Condition

The slope of soil bin was adjusted to 30° and weathered granite soil with initial water content of 19% was filled with the thickness of approximately 20cm and appropriately pounded. In order to facilitate lateral observation of the occurrence of breakdown process, colored soil was intruded with 10cm interval. And 3 gap water-pressure meters were installed at the upper, middle and lower parts of soil bin with 60cm interval respectively so that gap water-pressure can be measured through data logger with the interval of 10 seconds. As the main purpose of the test is to evaluate the efficiency of the measuring sensor, the intensity of rainfall was set at 200mm to make sure that breakdown is occurred within a short period of time.

3 optic fiber sensor packages for model test were installed by intruding at the upper, middle and lower parts of soil bin respectively. Then optic fiber was fixed at the steel frame which is at the upper part of soil bin, and each sensor package was also fixed using high-strength adhesive so that displacements occurred in between the installed sensors. When fixing cable to sensor, initial value was set after allowing tensile strength to some extent.

After setting initial values for optic fiber sensor and gap water-pressure meter, photographing cameras were installed in front side and lateral side of the test equipment and then test was commenced by operating rainfall simulation equipment.

4. Analysis of Test Result

Starting from the time after the lapse of approximately 10 minutes from operating rainfall simulation with the rainfall intensity of 200mm/hr, the occurrence of displacement at the surface part was observed by the naked eye. With the interval of 3 minutes, the phenomena of displacement in between sensors were measured using the changed value of intensity of radiation based on the deflection of the optic fiber. The measured values can be indicated in the form of the curve for the change depending on the lapse of time, which is shown in the Fig. 5 below.

Fig. 5 Graph for the Change in the Measured Values acquired by Optic Fiber Sensor

Starting from the upper part of soil bin, sensors are given the sequence of No. 1, 2 and 3, and the changes in quantity in between sensor No. 1, 2 and 3 were indicated in graph form. Passing 6 minutes after the test was commenced, the occurrence of displacement was started to be observed in between the sensors No. 1 and 2. After the lapse of 9 minutes, there was displacement occurred also in between the sensors No. 2 and 3 as well and the displacement was rapidly increased after 12 minutes. After around 15 minutes model slope was completely broken down. Namely, displacement was started to occur from the upper part of the slope due to intensive rainfall, the displacement was transferred down to the lower part, and then the breakdown was processed showing the breakdown of whole slope at last (Fig. 6).

Fig. 6 Whole Lateral View of Soil Bin after the completion of Breakdown Occurrence Test

Fig. 7 below is a graph which shows the changes in gap water-pressure measured simultaneously with measuring optic fiber sensor while carrying out the model test. Looking at the changes in gap water-pressure, measured value showed the mode of rapid increase from approximately 12 minutes after the commencement of the test. The fact that changes in the values were occurred first at the lower part than other parts showed the same result as the process of breakdown of
slope which was analyzed by measuring.

![Graph for the Changes in Gap Water-Pressure](image)

**Fig. 7** Graph for the Changes in Gap Water-Pressure

## 5. Conclusion

The purpose of this research is to evaluate the efficiency of a sensor which is currently being processed to develop the ground behavior sensing for short circuit using optic fiber. Rainfall simulation was compulsorily materialized to induce the breakdown of sediment slope of model soil bin and then the process of displacement occurrence was measures using measuring sensor to test the sensing efficiency of the sensor and to analyze problems.

The result of test showed the occurrence of final breakdown after 15 minutes from the commencement of the test. Based on the result of observation by naked eye for the surface and the lateral part of soil bin, the occurrence of displacement was confirmed after 10 minutes from the commencement of the test. And the result of analysis of the data acquired through the gap water-pressure meter showed that there was rapid change in the measured value after 12 minutes from the test and breakdown could be detected. Based on the above results of the test using optic fiber sensor, ground behavior was observed from 6 minutes after the commencement of the test, which is judged to have superior efficiency for the detection of displacement.

Through this test, it is expected that the optic fiber sensor for detecting short circuit, which is now under development, will be useful for the observation of the behavior of slope. In order to be able to forecast breakdown in advance through the detecting the behavior of slope, however, it will be necessary to develop data logger which acquires data on real time basis and to further pursue researches to prepare evaluation method for the analysis of the acquired data on real time basis.

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Terrafirma Landslide Services for Europe Based on Space-borne InSAR Data

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Abstract. Terrafirma is one of a number of services being run by the European Space Agency under the Service Element Program as part of the Global Monitoring for Environment and Security initiative (GMES) of the European Union. The project started in 2003 and will end in 2008 when it is planned that services will be adopted by the European Commission as part of their GMES strategy. Terrafirma is providing a Pan-European ground motion information service in each of the 25 member states of the EU to detect and monitor ground movements in relation to building stability, subsidence and ground heave, landslides, seismic activity and engineered excavations. The technology at the base of such a large-scale undertaking uses the data collected by European radar satellites, namely ERS1, ERS2 and ENVISAT processed through SAR interferometry (InSAR). By using state-of-the-art InSAR processing techniques, such as the Persistent Scatterers Interferometry (PSI) approach, thanks to the available archive of repeat satellite data, measurements of ground displacements with a millimetre scale accuracy can uniquely be provided back in time for the last 15 years. The project is aimed at informing specialists, planners and the community at large about the new approach to the assessments of risks from ground movements across Europe and beyond. Terrafirma intends to achieve it through practical examples of how ESA satellites can create ground motion measurements that, when coupled with expert knowledge, geosciences and engineering information, provide insights into these problems at a detail level, sometimes technically not reachable through the use of conventional techniques. The services provided by Terrafirma are to be delivered to end-users, represented mainly by public or private organizations dealing with ground movements connected both to natural hazards and human activities, primarily by National Geological Surveys who, integrating pre-existing and possibly in situ data with InSAR results can offer them enhanced products providing causal and modelled information services.

The landslide services consists of two different products: Landslide Inventory (LSI) helps to update pre-existing inventory maps produced with conventional geomorphologic tools by integrating PSI ground displacement information with cartographic, optical and ancillary data used to identify possible diagnostic morphologies and terrain features related to landslide and to extend spatially the point wise PS information. Landslide Monitoring (LSM) usually related to a specific slope, in which PSI measurements are combined in a GIS environment with cartographic optical and ancillary data to obtain an accurate analysis of spatial distribution of the ground displacements.

The first two-year Stage 1 of the project (which ended in 2005) was focused on the consolidation of both service providers and users. In November 2005 Terrafirma entered Stage 2, aimed at rolling-out the service across all 25 Member States of the EC. During this stage, processing equally covering all of the E.U25 Member States will be conducted along with the release of seven landslide products within Greece, Italy and Switzerland.

The present paper is focused on the description of the results obtained in Stage 2 for landslide mapping at regional scale in Graubünden Canton (Switzerland) and landslide monitoring at local scale for the Gorgoglione landslide (Southern Italy).

Keywords. Interferometry, Persistent Scatterers techniques, landslide inventory, landslide monitoring.

1. Landslide Inventory

The analysis, concerning the Canton Graubünden (Switzerland), for supporting landslide investigation has been carried out through a combined approach based on the use of multi-interferometric analysis, and photo-interpretation. The technique employed is the Point Target Analysis (IPTA) developed by Gamma Remote sensing based on the extraction of natural benchmarks from the SAR scene, typically parts of buildings, metallic structure and rock outcrops, which are not affected by temporal and geometrical decorrelation, and on the analysis of a large dataset of SAR images (at least 25 scenes). This approach permits to obtain for every Persistent Scatterer the medium deformation rate with an accuracy of 0.1–1 mm/year (Werner et al., 2003).

For this purpose thematic layers, including landslide inventory, aerial photos, digital elevation model and topographic maps, were managed within a GIS environment.

Canton Graubünden is located in the Swiss Alps between the Gotthard Crystalline Massive and Austrian-Italian border in the Penninic and Austroalpine nappes. This densely populated region is a landslide prone area. The Buendnerschiefer and the Flysch Formations of Eastalps are fine-grained rocks, which are affected by slope instability processes. The largest rock slide of the Holocene is located in Flims and concerns a total volume of 9 km$^3$ (Noveraz et al., 1998). There are also many installations for touristic purposes in the unstable slopes, sometimes they are located in the permafrost areas. Between St. Moritz, Chur and Disentis many other landslides are still active and there are high annually costs for the mitigation and the countermeasures. The damage concern inhabited areas, industrial zones and many roads in the lateral valleys. For the Canton a landslide inventory is not available, but several of the large landslides are known and the geological documents and field surveys have allowed an integration of InSAR technique for hazard assessment. The analysed area...
has an extension of about 3800 km$^2$. Radar datasets used are SAR images acquired by ERS1 and ERS2 satellite (spanning the temporal interval from 1992 to 2002) acquired both in ascending and descending geometry and SAR images from ENVISAT satellite (spanning the temporal interval from 2002 to 2006) acquired both in ascending and descending geometry. Processed through the IPTA analyses, allowed us to investigate and confirm most of the large known landslide and to identify several new landslides, producing a final inventory of 112 landslides covering an extension of 270 km$^2$ that represent the 7% of the whole investigated area.

Landslide classification is based on the medium velocity computed for every landslides from ERS/Envisat IPTA data (Fig. 1).

Fig. 1 Landslide Inventory of the Graubuenden Canton carried out by means of integration of InSAR technique and conventional geomorphologic tools.

2. Landslide Monitoring

The local case study is related to an earth landslide, in a silico-clastic turbidite formation (Boiano, 1997) in Gorgoglione locality, a small village located in Southern Italy (Basilicata region), affected by an ancient landslide, re-activated during the Irpinia earthquake (1980), as testified by its classification as area at moderate - to - very high risk (class R4) reported in the P.A.I. (Hydrogeological Asset Plan). Following an acceleration of the ground movements observed between late 2003 and the summer 2004, which induced the evacuation and the demolition of several buildings, field surveys, carried out by experts from GNDCI (National Group for Geo-hydrological Disaster Prevention of the Italian funded by the National Civil Protection Department), highlighted the presence of a general slope instability in the portion of the village located below the main square. After the 2003-2004 acceleration in situ instrumentation has been installed by the Gorgoglione municipality, but the causes of the slope movements are still under investigation.

The Gorgoglione landslide is described as a compound landslide: an earth slide with rotational and translational components of the movement. A PSI analysis, through the Permanent Scatterers (PS) technique (Ferretti et al., 2001) developed at the Politecnico di Milano (POLIMI) in advanced mode (APSA), on historical ERS1/2 data and current ENVISAT data was performed. The PS spatial distribution and velocities highlighted the presence of movements in the southern portion of the village, mapped as area of high risk by the P.A.I., whereas the rest of the village is in a stable condition (Fig. 2). The analysis has allowed us to redraw correctly the landslide boundaries, in particular to define better the landslide crown. Apart from the spatial distribution of movements, the APSA analysis provides information about the temporal evolution of displacement rates from a backscattering structure on the ground. For every acquisition used in the interferometric processing it is possible to determine displacements values relative to a reference date. The whole observation period, spanning of 16 years, shows an overall movement of ca. 16 cm, resulting in a different displacement rate for the two
time intervals, ca. 10 mm/y for historical dataset (1992-2002) and ca. 20 mm/y for current dataset (2002-2008). This difference is in agreement with the landslide acceleration occurred in the period from late 2003 and summer 2004. The PS analysis performed along the cross section, based on the velocity rate interpolation for every dataset, shows a good correspondence with the geomorphologic aspects; in particular, the movement starts in correspondence to the disturbed flysch, with an initial increase of velocity that become constant along the slope. It is also evident, as highlighted before, that in correspondence of Piazza Zanardelli Envisat data show an increase of velocity with respect to ERS data.

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Fig. 2 Distribution of PS ERS and Envisat projected on aerial-photo (Voloitalia 2006) and overlaid on the P.A.I. hazard map.

The upper part of the slope is characterized by the presence of a sharp boundary between PS with null velocity and PS with high velocity values. This suggests an advancing phenomenon without any retrogressive activity, this information is consistent with the geological cross section and with the slope instability behaviour since its reactivation in 1980.
Fig. 3 Geomorphologic cross section along Gorgoglione landslide (from: Lo Bosco et al., 2005) and PS velocity rate interpolation along the cross section

Conclusions
The results confirm the capabilities of multi-interferometric InSAR data, integrated and coupled with conventional techniques, to support landslides investigation at regional scale.

Considering the high costs related to landslide damages and the difficulties in the assessment of the state of activity, especially over urban areas, the use of an InSAR approach can positively impact on the current hazard mitigation activities along national and local authorities. As a next step the monitoring of the actual movements using InSAR techniques could be implemented in the risk management procedures.

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References
The Relationship Between Landslides and the Development of Stream Network

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Abstract. Landslides are considered as some of the important contributory factors in the process of shaping the landscape. Scientists have identified various kinds of landslides, and out of these the "flow" takes a special place among other kinds of landslides due to the destruction it causes to life and property. Therefore it has been recognised as the most destructive landslide.

Landslides are being considered as most severe among natural disasters in Sri Lanka. They frequently occur in hilly areas in Sri Lanka during rainy seasons. Southwest monsoon brings incessant heavy rains to southwest and western parts of the hilly areas and northeast monsoon brings heavy rains to eastern parts of the hill country. In addition to these two monsoonal rains, inter monsoon also bring rains to the slopes in Sri Lanka.

In this paper on landslides, attention is focused only on "flow" which deems to bring the most disastrous consequences. In this paper the term "landslide" also has been used in some places, for "flow".

Photographs presented wherein in support of facts put forward, are taken from areas in Nuwaraeliya and Ratnapura districts wherein frequent landslides occur. Aerial photographs depict scenes of landslides which occurred in the Nuwaraeliya District during March to April in 1986. Close up shots which have been taken during field surveys conducted from March 2005 to June 2006, will show views of landslides that occurred in May 2003, in the Ratnapura District.

All the landslides subjected to this study indicate the fact that the mass on a slope will travel downwards to a river, carrying flow of mud and debris while depositing the mud and debris it carries in the surrounding area on its way downwards. This proves the fact that there exists a relationship between the slope where the landslide begins and the river where the process ends. In this study special attention has been paid to analyse the role of ground water, spring, head and foot of the landslide, the route, cracks, and mud. The objective of this study is to explain the relationship between landslides and the development of stream network.

Keywords: head, foot, ground water, stream, spring, cracks.

Introduction: In billion years of history of geology, creation of geomorphic features on the surface of the earth is a natural phenomenon. Though this is a natural process it has become a formidable task to protect the equilibrium of the earth and to carry it towards an unforeseen future. One phase of this process can be described as the changing of existing geomorphic features and subsequent creation of new features in those places. Some processes of such formations are the removal of mass from steep slopes elsewhere and emergence of new springs in such places where once those mass stood.

According to the literature on landslides, Sharp (1938) was the first to give a definition on landslides. However, both the definition and classification are subject to controversial. This may be due to the difference of opinion held by scientists who where not satisfied with the previous definitions and classifications given. However, in almost all the classification "flow" has been included.

Occurrence of a landslide
From a geomorphic point of view, the landform subject to landslide is a slope. A landslide can occur at any point of the slope. On most of the slides observed, the slide head was in between the upper part of the slope and its middle. Out of numerous landslides observed, the occurrence of a landslide at a lower point of the slope was evident only in a landslide which occurred at Elapatha in the Ratnapura District where a loss of 57 lives was recorded. Not a single landslide that occurred on the summit has been observed so far.

There should be a contributory factor in a landslide capable of removing the mass out of the slopes. There are two factors contributing to removal of mass out of the slopes. These two contributory factors are water and vibration. The main contributory factor in almost all the slides observed in this study happened to be water. The heavy rainfall experienced in the hilly areas of Sri Lanka during two monsoons has become the main reason for landslides in that area.

Destination of the flow
The presence of excessive water in the surface and within a mass of a steep slope will create the necessary conditions required for mud flow. If the surface of the mass becomes muddy while the area beneath is dry there would be no sufficient force to flow. Is there any deciding factor on the foot of the landslide? It very often happens that a flow travels a
long distance and ends up in a stream already existing (Fig. 1). Sometimes the flow could move a certain distance and get bifurcated and the movement will end when it meets the stream, in the same way, in two places. In all the observed slides there was a continuous flow right up to the point of the stream. The specialty of this feature is that the journey of the flow ends at a point it meets an existing stream. It is clear from this that a slide beginning from a slope has a definite destination—that is a stream or a reservoir.

Deciding factors of a flow

The deciding factor of a landslide is the force it creates to cause the flow. Water and gravity are two imperative factors in this process. However, it has been observed, that though there was sufficient gravity, the flow has not traveled a long distance. In some cases adjacent to a landslide, there occurred movement of mass without a flow. On a close examination it has been revealed that gravity is not sufficient enough to cause a flow. It has been observed, in such cases the mass had water only on its surface, and not within. For the flow to be effective the mass should be muddy, and for this there should be accumulation of water on the surface of the mass and within. Therefore if the mass does not get water from within, by itself, gravity will not cause a flow. It appears that out of two identified factors, the most important contributory factor to cause a flow is water.

How far will a flow travel?

Almost all the landslides observed in this study have traveled to a stream nearest to the slope where the landslide occurred. However, the distance between the head and the foot of these landslides was different from each other. Generally the distance traveled has been observed to be in the range of ¾ - 1 ½ K.M. When the distance traveled by a landslide is considered it has been recorded that in 1881 a landslide traveled a distance of 1.5K.m., in Elm in Switzerland killing 115 people (Heim, 1932). A landslide which occurred in West Virginia of the United States in 1972 is reported to have traveled 24km., and 125 people had perished in this landslide (Davis, 1973). In all observed landslides it is found that the mud and debris has traveled far away from the rock base. Therefore it is difficult to give an exact distance a flow would travel. The flow travels until it meets an already existing stream or a reservoir.

Characteristics of the landslide head

Once a mass in a slope moves, new geomorphic features have been observed in the place where the mass stood. Therefore it is worthy to examine whether there is any relationship between these new features and the flow.

The head will be formed after the flow in the place where once the slope stood (Fig. 2). The size of the head may vary from place to place. The physical features of this formation will be similar to that of capital "L." and the dimension of this formation will indicate the magnitude of the mass removed from the slope. Therefore the recurrence of a landslide at this point, it is necessary that head so formed is refilled. However, it is difficult to say how long will it take to form a new mass at this point.

A large boulder can be seen as the headscarp (Fig. 3). This feature could be seen almost in all the places where the landslide occurred. This boulder will be helpful to stabilize the remaining upper part of the slope after a landslide had occurred.

It is worthy to examine whether there is any relationship between the water springs present at the head of the landslide and the flow. Some water springs could be seen around the place where the landslide occurred. However, a direct relationship of such water springs could not be established even after the landslide. These water springs were seen as isolated springs.
Fig. 2. View up to North. Northeast to Northwest is the part of the boundary of the head. The spring with full of ground water and marshy around the spring. (Photo H. L. Perera).

Fig. 3 The headscarp of the landslide at Elapatha. (Photo: H. L. Perera).

It is observed that the function of the water springs in the head was quite different from that of the isolated springs found at the head. The spring at the head was characterised by its muddy or marshy and slippery nature (Fig. 2). This was caused by ground water coming out through these springs and making the surrounding area marshy and muddy. Further, this ground water would form into a waterway and flow downwards. This waterway would get augmented gradually and ends up at its destination - the stream.

The relationship between flow and the stream

Since the flow will carry and deposit mud and debris in and around its path downward, it is difficult to reach the head of the landslide over this layer of mud soon after the flow. Therefore photographs included in this paper were taken two years after the occurrence of the flow. By that time the stream that was formed after the flow was well established and in some places the presence of fish was evident. This new stream carries mud and debris to the stream at the foot of the flow thus adding a new stream to the stream already existing from the head.

It is worthy to examine whether there is any relationship between the cracks appearing on the slope before a landslide, and the water springs at the head. The crack which is hardly discernible at the outset will become larger with the passage of time. This crack generates deep in the slope and it comes up to the surface and in the event of landslide, the mass with this crack moves. This will explain the fact that there is a relationship between the crack so formed before the landslide and the water springs at the head of the landslide. In addition to the water springs that appear on the slope, it is clear, that there will be emerged springs inside the slope causing cracks in the slope. The stability of the mass will get first disrupted due to this emergence of innermost springs.

The water springs are stabilized since they emanate from ground water, and the water way started at this area, would be augmented gradually, by other water springs that it meets on its way downward.

The path of the flow on the slope will be grooved by nature. Cross sections of these channels indicate that they take different forms. In some places it takes a "V" shaped cut, in another place it takes a "U" shaped cut. In some cases, it takes no definite form.

In the path of the landslide, it takes only a mere fraction of time to form this furrow. In the landslide that occurred at Elm in Switzerland in 1881, it took only 50 seconds to move a mass of 10Mm³ (Heim, 1932). A similar landslide occurred in Alberta in 1903. It took only 100 seconds to move a mass of 30 million cubic meters in magnitude (Dave 2008). When considering the time taken for the flow to happen the size of the mass moved, and the characteristics newly created in the process, it is clear that the mass would get itself ready before the flow properly occurs. The landslide only causes the movement of mass of mud and debris right up to the destination.

From the gradual increase of the water way from the head to the foot, it appears that along the path of the flow, there exists springs. These springs between head and the foot emerge up simultaneously. During the rainy season the ground water will get increased and the path opened up will be not enough to carry this water downwards. At this point the water logging occurs and then, from then onwards it will make the water flows turbulent and become muddy ultimately. This process will erode the area around the springs and the path as well and ultimately the flow becomes muddy. This process will be evident along the path from the head to the foot of the flow. In some places a big noise is heard close to the mass before the occurrence of the landslide, due to this turbulence within the mass. Mud moves outwards towards the surface of the earth before the landslide is activated by this inner formation. This formation around the boulders will dislocate the boulder from where it stood. This situation will cause the complete break down of the stability of the mass and by this
time heavy rains make the surface of the mass saturated with water and the balance part of the process is played by gravity which takes the landslide downwards. In most places of landslides, boulders move outwards and downwards initially. The formation of mud at a higher place of the slope cause a landslide instantaneously, while the same process at a lower part of the slope will cause a landslide belatedly. Gravity is the cause for this difference. It is clear that there is a relationship between the landslide and the development of the stream network.

Conclusions

1. Landslide removes the mass which was hitherto blocking the emergence of a stream to the surface. Therefore, it appears that landslide will function as a facilitator by contributing to remove a mass on a slope and allowing to develop the stream network. So, it is clear that landslide is a natural process.

2. A landslide may contain mud, debris, and soil in different proportions, especially of mud and of earth. When the causing factor for moving a mass from a slope is water, the flow will contain more mud. When the causing factor for moving a mass from a slope is vibration, the flow will contain more soil.

3. There are three important factors that cause a landslide. They are water, gravity, and vibration. Water makes the mass in to mud and drag the load. Gravity takes the mass downwards over the slope. Vibration shakes the mass which is already unstable.

4. There are various types of landslides all having a common definition. Since characteristics are different from each other they have to be defined accordingly.

5. If the landslide terminates in a naturally formed reservoir, the reservoir would get further stabilized. However, if the landslide ends in a man-made reservoir it needs to be studied further and see whether the man-made reservoir tries to expose to the nature.

6. This study reveals that there is a relationship between landslides and the development the stream network. The problem that corps up is whether the already existing stream stabilizes the flow of the river by taking more water from the ground water or whether the slope stabilizes by issuing more ground water to already existing stream. This issue needs to be studied further.

Acknowledgements

I thank Professor W. P. Wickramagamage, Department of Geography, University of Peradeniya, to encouragement and support given. The unstinted assistance rendered by Mr. Sanath Kumara, Mr. Premaratne, and Mr. Jayantha, Assistant Land Use Planning officers of the Land Use Planning Office in the Ratnapura District by conducting field surveys to collect information required for completion of this study, is very much appreciated. Finally, many thanks to my husband Mahinda, who always stood by me giving encouragement throughout this difficult period.

Reference


Landslide Hazard Mapping Around Phidim Bazaar, Eastern Nepal

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Abstract. The evaluation of various components, based on the topographic map, depicts landslide prone areas adjacent to the district capital of Panchthar District in Eastern Nepal. The hazard map is prepared by analysis of different types of maps and other field information. Landslide hazard mapping is based on soil characteristics and geo-morphological analysis. Thematic maps portraying soil depth, soil type, morpho-structure, geology, slope, hydro-geology, land use and landslide inventory were prepared, and all these maps were superimposed giving them rating values for each component. After giving proper ratings, all the values were added, and on the basis of the total value the hazard was categorized as: low, medium, high and very high. The main causes of landslides are thick soil cover, high gradient tributaries and streams, poor hydro-geological condition and heavy precipitation during monsoon. The landslides are primarily due to natural phenomena (geo-morphological) rather than man made infrastructure, but the desertification process is another cause of landslides in the study area. To decrease the hazard level, the poor drainage and irrigation system in the paddy field should be improved. Plantation should be done in the barren land. Proper breast wall and gabion walls are necessary to control gully erosion in the colluvial soils along the road cut. The proper drainage channel and the distribution and the management of the monsoon rainwater are to be managed.

The resulting composite hazard map is aimed for use by planners and developers as a multipurpose map.

Keywords: Landslide, field investigation, hazard map, Phidim, Eastern Nepal

1. Introduction
Landslides are the most common natural hazards in Nepalese mountains, which have fragile nature. The selected site is most vulnerable for landslides and also fastest urbanizing area of the eastern region of Nepal. Therefore the probable occurrence of landslides should be identified to reduce damage in the region. In the region, landslides occur due to heavy rainfall, steep slope, rugged topography, vegetation and deforestation, weak geological condition or fragile geological structures. However, human activities also aggravate hazards due to insufficient attention to infrastructure development, traditional methods of cultivation and overexploitation of natural resources. Landslide hazard assessment becomes vital in developing infrastructure and environmental protection. According to Varnes (1984), landslide hazard in a given area can be assessed in terms of probability of occurrence of a potentially damaging landslide event within a specified period.

2. Landslide hazard mapping
For landslide hazard mapping it is essential to collect data of all causative factors that initiate slope instability. This study includes hazard components such as soil type and depth, natural slope, rock type and strength, hydro-geological condition, vegetation, hydrodynamic condition, seismotectonic component, land-use pattern, etc. For this, basic maps such as slope map, rock and soil map (Fig. 3, Poudyal 2000), soil depth map, landslide distribution map, land use map, morpho-structural map, geological map (Fig. 2, Poudyal 2000), and drainage and spring distribution map and hydro-geological map were prepared. The rating values of different components are modified from the Mountain Risk Engineering Hand Book (Deoja et al. 1991) for rock and soil slope zone.

Fig. 1 Overlapping method of hazard mapping

By synthesizing or overlapping (Fig. 1) the above maps and other information with their rating values, the final landslide hazard maps were prepared, and the study area was divided into low, medium, high and very high hazard zones.

3. Conclusions
From the study of hazard mapping it is concluded (Fig. 4, Poudyal 2000) that Phidim bazaar area is safer then surrounding areas. The rocks exposed to the north of this bazaar are quartzite of the Lesser Himalaya, which dips towards south and supports bazaar area. Jorsal bazaar is found in medium hazard level with slope angle of 25° -35°. The south of Phewadin village is high hazard zone due to high slope angle (35°-45°) and occurrence of landslides.

The soil type and the land use pattern are also other parameters that control the hazard level in the study area. Due to the presence of coarse grained and highly porous nature of the colluvial soil and barren land, Bharapa, Hokse, Dandangaon, Siruwani villages are found in the high-hazard zone. The soil has high liquid limit during the monsoon season and exacerbate towards failure. To decrease the hazard level the poor drainage and irrigation system in the paddy field should be properly managed. Plantation should be done in the barren land. Proper breast wall and gabion walls are necessary to control gully erosion in the colluvial soils along the road cut.
Fig. 2 Geological map of Phidim area

Fig. 3 Rock and soil map
The study of the hazard level in the rock slopes shows that the slopes east of Siruwani along the small tributary, west of Malbase along the tributary, north of Jorsal bazaar along the road cut, and north of motorable bridge over Hewa khola are highly hazardous and need immediate action. The major factor contributing to high landslide hazard in the area include rock type (Phyllite with intense weathering and highly deformed), the presence of natural slopes towards south having low rock mass strength and forming wedge failures between joints and foliation, and sparsely distribution of vegetation, and road cut.

References


UNESCO, Paris

Abstract. Heavy rainfall from the storm of May 21-23, 2006 triggered landslides on steep slopes in the province of Uttaradit, northern of Thailand. In addition to landslides, heavy rainfall caused flooding and debris flow floods that killed hundreds of people and trapped thousands of other people. In this study, The SINMAP(Stability Index Mapping) model was applied to calculate landslide susceptibility of Namta-Namlee sub-catchment, Thapla district, Uttaradit province. As extensions to ArcView® 3.x, SINMAP is based on the infinite-slope stability model. Slope stability is calculated using topographic parameters such as slope and topographic wetness which derived from DEM (Digital Elevation Model). Soil strength parameters and hydrological parameters are considered more variable and can be adjusted to better match existing conditions. An inventory of actual landslide point locations, derived from interpretation of aerial photography, satellite image SPOT-2 and field survey are used to verify model results. The major output of the SINMAP model is the Stability Index (SI) grid theme. The SI was converted into a susceptibility map and makes the result more helpful for decision makers. The results of this study indicate about 70% reliability in predicting slope instability in the selected study area.

Keywords. Landslides susceptibility, SINMAP, Slope stability

1. Introduction

During date 21th -23th of May 2006, it was heavy rainfall in the northern provinces of Thailand, which is 330 millimeter of maximum rainfall at Laplae district, Uttaradit province. Following by flash flood and landslides, it was affected on the provinces of Uttaradit, Sukhothai and Phrae. 88 peoples had been killed, while 29 peoples missing and 816,249 Rai (130,600 ha) of agricultural area was damaged (DDPM*). This disaster made initial loss of 144 million baht valued. Nevertheless, Thailand had been faced similar hazard in the other parts. Consequently, lessons must be learned from the past and present in order to find out the appropriate measure for prevention and mitigation.

Hence, to understand of this systematic phenomenon we need to study on involving factors in order to evaluate the hazard. To conduct a landslides hazard map, traditional methods have been based on extensive fieldwork. This is slow and expensive, with the increasing of available high resolution spatial data sets, GIS and huge capability of computers, it is possible to process landslides susceptibility map and minimize fieldwork. In this study, the SINMAP (Stability Index Mapping) model was applied to calculate landslide susceptibility of Namta-Namlee sub-catchment, Thapla district, Uttaradit province.

2. Study area

The study area Namta-Namlee sub-catchment is in Thapla district, Uttaradit province. It is in the northern part of Thailand, which cover an area of 12,492.96 ha. It located in the valley of the Nan River.

3. Methodology

Two main processes were used in this study: Image processing and GIS. First, to carry out sub-catchment of the study area, a hydrology model was applied by using DEM 5 m. resolution as a primary data. Then landslide susceptibility of the study area was conducted with a SINMAP model. This model is base on the infinite-slope stability model. Slope stability is calculated using topographic parameters such as slope and topographic wetness which derived from DEM 5 m. resolution as well. Soil strength parameters and hydrological parameters were considered more variable and were adjusted to better match existing conditions. The major output of the SINMAP model is the Stability Index (SI) grid theme and it provides slope area, plot charts and statistical summary for each calibration region in the study area.

After that, image processing was applied to obtain an actual landslide point locations from SPOT-2 imagery. Supervised classification using maximum likelihood technique achieved land use/land cover map, at the same time an aerial photos 1:4000 which took after landslide occur were exploited to identify landslide spot. Finally, the actual landslide
point locations and field survey were assistance to verify model results.

4. Result

In this study, it was found that 63.06 percentage of the study area have a height value from 300-500 m. above mean sea level. Meanwhile, slope gradient are average from 25-30 degree, it is 21.62 percentage of the study area. Furthermore, Land use/ Land cover was classified in to 6 categories consist of paddy field, orchard, urban area, forest plantation, deciduous forest and disturbed deciduous forest. The majority of land use/ land cover of Namta-Namlee sub-catchment is disturbed deciduous forest with 70.12 percentage of the study area.

Additionally, the out put of the SINMAP model; the SI values were ranged from 0 to 1.5 then it was reclassified in three susceptible zones (low susceptible, medium susceptible, and high susceptible) and converted into landslide susceptibility map. In this study SI was compared with actual landslide and it shows that about 70% of land slide susceptibility map is reliability in predicting slope instability in the selected study area. Moreover, it was found that both land use and slope degree are the most factors, which are affected on landslide concurrent.

5. Conclusion Remark

Landslide susceptibility map, which it is derived from a SINMAP model, it is reliable in predicting on landslides. However, it should be considers as a slope stability model which mainly use data derive from DEM. For further study other parameters such as climatic data should be taking into account.

Acknowledgement

In this study, we wish to express our gratitude to Mr. Phaitoon Khadeetham for his valuable advices and guidance. Moreover, we are truly thanks to Mr. Surin Wajjaroen, senior Soil Surveyor, Miss Prapa Taranet, Agronomist and other colleagues from Office of Land Use Administration and Development (OLAD), Land development Department (LDD) for their hard working and helpful in every aspect.

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http://www.em.gov.bc.ca/Mining/Geolsurv/Surficial/landslide/default.htm
Instability of Cut Slopes Comprising Deep Weathered Argillaceous Limestone in New Fengjie County on Three Gorges Reservoir in Central China

Sheng-wen Qi (Chinese Academy of Sciences, China) · Zhong Qi Yue (University of HongKong, China) · Zhong-hua Chang (Chinese Academy of Sciences, China)

Abstract. One of the many counties that were totally relocated owing to the impounding of Three Gorges Reservoir (TGR) is Fengjie County. The new Fengjie County is relocated at the area of Sannashan as the town centre and eight sub-areas. During the selection and construction of the new county site, many deep foundation pits and high cut slopes were formed.

A majority of the cut slopes, excavated through the deep weathered group of argillaceous limestone of Badong formation of the reservoir bank, have not been strengthened yet. The large area excavation has sped up the weathering process, and then induced the instabilities. The instabilities are posing an actual threat to the traffics and the buildings.

The paper analyzes the rock behavior of the excavated slopes in the deep weathered bank, and classifies the instabilities into surface flake, erosion, wedge sliding, circular slide, debris flow, rock fall, rock slumping and toppling, and then discusses the relationship between the instabilities and the structures of the slopes.

Keywords. Slope, instability, weathering, excavation, weathered argillaceous limestone.

1. Introduction
The Three Gorges Reservoir is the biggest one being constructed in the world and will be completed by 2009. It has concrete gravitational dam, with height of 185m and length of 2309 m, is being constructed in Sandouping, 40km upstream from Yichang city (see Fig. 1)(Sheng et al, 2002). The reservoir has a normal pool level (NPL) of 175m and will inundate 2 cities, 11 counties and 116 towns, among them 9 counties and 55 towns are totally submerged, and the totally inundated land area is about 632km². About 847,500 people had to be resettled, and the number of planned dynamic emigrants is about 1,208,800. In order to relocate the submerged counties and arrange for such a large number emigrants, new suitable sites had to be searched.

One of the many counties that were totally relocated owing to the impounding of Three Gorges Reservoir (TGR) is Fengjie County. As shown in Fig. 1, the old Fengjie County had been located just on and above the Yangtze River at the Three Gorges in Central China for thousand years. The new Fengjie County is relocated at the area of Sannashan as the town centre and eight sub-areas, the hillside slopes on the Yangtze River. During the selection and construction of the new county site, many deep foundation pits and high cut slopes were formed.

A majority of the cut slopes, excavated through the deep weathered group of argillaceous limestone of Badong formation of the reservoir bank, have not been strengthened yet. The large area excavation has sped up the weathering process, and then induced the instabilities. The instabilities are posing an actual threat to the traffics and the buildings.

The paper analyzes the rock behavior of the excavated slopes in the deep weathered bank, and classifies the instabilities into surface flake, erosion, wedge sliding, circular slide, debris flow, rock fall, rock slumping and toppling, and then discusses the relationship between the instabilities and the structures of the slopes.

Fig. 1 The location of the studied site (modified after Qi et al, 2008)

2. Site geological setting
The site is located at the about 200 km upstream of the Three Gorges dam (Fig. 1). The site topography is an erosion type and has low to middle high mountains and steep valleys. It has developed five elevated platforms with six terraces.

The site climate belongs to the subtropical humid and south to east monsoon zone. The annual average temperature is 17 °C. The annual average precipitation is about 1107 mm. The utmost precipitation is about 1410 mm. The annual average humidity is about 69% (Weathering Station of the Fengjie County, WSFC, 1981).

The main bedrock outcropped in Fengjie is the Badong Formation of the Tertiary system (T2b), which consists of a series of strata of impurity carbonate rocks including argillaceous limestone and marl, and detrital rock (i.e. argillaceous sandstone), with littoral-shallow facies and lagoonal facies (see Fig. 2). The Badong formation is more than 700 m thick and can be divided into four members namely T2b1, T2b2, T2b3 and T2b4 (Team 107 of Geological Bureau of Sichuan province, 1980). And the most of the new town has been or being developed in the third member of T2b3 as indicated in Fig. 2.
3. Instability of cut slopes comprising deep weathered argillaceous

During the construction and operation of the new county, lots of cut slopes were formed and not strengthened, especially the cut slopes along the Feng-Wu highway. As noted above, the most of slopes were formed in the third member of T2b3. However, it has been gradually found that the argillaceous limestone of the third member of T2b3 was deeply weathered, and the most depth of the frontier of the highly weathered can even over 150 m (Qi et al, 2008). Meanwhile, it has been proofed that the argillaceous limestone was susceptible to weathering especially karst (Zhang, 2004; Lei, 2005; Tang, 2005), and the highly weathered argillaceous limestone can be quickly completed weathered after excavation (Qi et al, 2008). Weathering induced the group of argillaceous limestone dramatically worsen in strength and elastic modulus owing to losing of calcite of the rock. Study shows that the reductions in uniaxial compressive strength and elastic modulus of argillaceous limestone caused by weathering from fresh to highly weathered reach 45.6% and 61.1%, respectively (Zhang, 2004). Therefore lots of slopes not strengthened timely failed.

![Fig. 2 Badong formation in the studied site of the Fengjie County (after Qi et al, 2008)](image)

After carefully investigation, we classified the instability of cut slopes into 8 types as follows: surface flake, erosion, wedge sliding, circular slide, debris flow, rock fall, rock slumping and toppling. Table 1 lists the characteristics and typical photo of each failure type.

Study indicates that the instability type is closely related with structure of the cut slopes. Surface flake mainly occurred on the surface of slope with thick purple-red mudstone of T2b2 (see Photo No.1 in Table 1), erosion mainly occurred in the slope with completely weathered argillaceous limestone or residual soil under surface runflow, and gullies often formed on the slope surface (see Photo No. 2 in Table 1). Wedge sliding mainly occurred in the middle to thick layered slope with highly to minor weathered argillaceous limestone (see Photo No. 3 in Table 1). Circular sliding often occurred in the highly weathered or closely fractured rock mass, and rock fills (see Photo No. 4 in Table 1). Debris flow often occurred in the slope with highly weathered or closely fractured rock and rock fills under rainstorm (see Photo No. 5 in Table 1). Rock fall often occurred in the steep slope with highly weathered fractured argillaceous limestone (see Photo No. 6 in Table 1). Rock slumping and toppling often occurred in the horizontal layered argillaceous limestone (see Photo No. 7 and Photo No.8 in Table 1).

Conclusions

This paper introduces the instability of cut slopes in the relocated site of new Fengjie County, and classifies the instabilities into surface flake, erosion, wedge sliding, circular slide, debris flow, rock fall, rock slumping and toppling. Meanwhile, the relationship between the instabilities and the structures of the slopes is discussed. The findings of this study will be valuable to the coming remediation measures.

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<tr>
<th>Failure mode</th>
<th>Characteristics</th>
<th>Typical photo</th>
</tr>
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<tbody>
<tr>
<td>Surface flake</td>
<td>The weathered rock surface falls as thin sheet, like snow flake.</td>
<td>Photo No. 01</td>
</tr>
<tr>
<td>Erosion</td>
<td>This type often has gullies formed on the slope surface by surface water, and caused the soil and water loss problem.</td>
<td>Photo No. 02</td>
</tr>
<tr>
<td>Wedge sliding</td>
<td>The block enveloped by two intersected planes, and slides parallel to the intersection line of the two nonparallel planes without rotation.</td>
<td>Photo No. 03 (after Wu et al, 2005)</td>
</tr>
<tr>
<td>Circular slide</td>
<td>The rock mass slides along a surface that approaches a circular shape.</td>
<td>Photo No. 04</td>
</tr>
<tr>
<td>Debris flow</td>
<td>The slope, comprising completely or highly weathered argillaceous limestone, fails and turns into mud and rock torrent under rainstorm.</td>
<td></td>
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<tr>
<td>-------------</td>
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<td></td>
</tr>
<tr>
<td>Photo No. 05</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rock fall</td>
<td>The isolated rock blocks lose base support and fall.</td>
<td></td>
</tr>
<tr>
<td>Photo No. 06</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rock slumping</td>
<td>Single or multiple blocks rotates backward, and moves into edge/face contact to form one or more detached beams.</td>
<td></td>
</tr>
<tr>
<td>Photo No. 07</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Toppling</td>
<td>Single or multiple blocks rotates forward.</td>
<td></td>
</tr>
<tr>
<td>Photo No. 08 (after Qi et al, 2008)</td>
<td></td>
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</table>

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Two-dimensional Motion Analysis of Landslides Using Particle Image Velocimetry Method

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Abstract.
It is important to clarify the motion mechanism of landslides for reasonable prevention. Recently, video cameras have been set up at some sites where landslide collapse may occur. Video images of the collapse process captured at two landslide sites in Japan are obtained. The Particle Image Velocimetry (PIV) method, which is one of the typical techniques for velocity vector detection, is applied to the video images in order to understand the landslide collapse. As a result, the surface movement of the landslides is analyzed in detail, and it is suggested the possibility to estimate the inner movement of landslide blocks.

Keywords. Landslide, Motion analysis, Particle image velocimetry

1. Introduction
It is important to clarify the occurrence and motion mechanism of landslides for reasonable solution planning. Observation equipments such as extensometers can be set up when a landslide activity is expected, and the minute change of a landslide process until the beginning of collapse can often be captured. The detailed measurement data is used, for instance, to observe the danger in emergency responses, and forecast collapse time etc. However, the displacement cannot be continuously measured with the equipments such as extensometers when finally the collapse occurs accompanying by a great deformation. Thus, currently, it is difficult to estimate neither the scale and speed nor the movement form of the movement mass in the final activity of the landslide, and consequently, to presume the attainment range of the movement mass accurately.

Recently, surveillance cameras such as video cameras are set up at some sites where landslides may occur for risk avoidance. It is considered that if the whole process of a landslide movement from beginning to the end can be made clear, the understanding of the detailed motion mechanism of a landslide and the planning of effective landslide countermeasures will become possible. In this study, a two-dimensional motion analysis of landslides using image sequences derived from video cameras is attempted.

2. Case study
Two examples of landslide collapse captured by video cameras in Japan are analyzed in this study.

2.1 Higashi-Yokoyama landslide
The first analytical example is the landslide occurred on May 12 and 13, 2006 in Higashi-Yokoyama district, Ibigawa town, Gifu prefecture, Japan. Video pictures from the period 12:00 on May 12 to 10:00 on May 13 were taken by a surveillance video camera operated by Etsumi Mountain Area Sabo Office, Chubu Regional Development Bureau, Ministry of Land, Infrastructure and Transport. Among intermittent landslide collapses, the one occurred at 20:02 on May 12 with the volume of debris about 1,000m³ (1st one), and the one occurred at 22:40 on May 12 with the volume of debris about 10,000m³ (2nd one), also, the one occurred at 7:59 on May 13 with the volume of debris about 40,000m³ (3rd one) are extracted for analysis.

2.2 Ootou landslide
The second analytical example is the landslide occurred at the slope of the national road of Ootou village, Nara prefecture, Japan in 2004. After the crack of the wall on the national road was confirmed in the end of January, 2004, the landslide movement was activated by the rainfall of several typhoons that came after June, and the large collapse was caused at 0:15 AM, August 10. The moment of the collapse was captured by a surveillance video camera equipped on the satellite communication car of Kinki Regional Development Bureau, Ministry of Land, Infrastructure and Transport in the local site.

Fig. 1 Location of two landslides
3. Methodology

3.1 Principle of the Particle Image Velocimetry method

The Particle Image Velocimetry (PIV) method, which is one of the typical techniques for velocity vector detection is used in this study. The correspondence points between neighboring frames are decided by intensity correlation, and the movements of the brightness distribution patterns in the images are pursued to perform the multi-point measurement of velocity vectors at same time. Thus, the detailed information about the actual movement directions and velocities of each part of a landslide can be acquired. Figure 2 shows the principle of PIV.

4. Analysis results

4.1 Higashi-Yokoyama landslide

The image sequences of the three landslide collapses are analyzed using the PIV method, and the velocity vector and velocity distribution are understood. Figure 4 shows the analytical results of the third landslide collapse. The third collapse was the largest one among a series of collapses of the Higashi-Yokoyama landslide. The entire slope collapsed, and the final landform shape was formed. Moreover, unlike the first and the second collapse, the third collapse occurred on the morning of May 13, the collapse process can be interpreted well from the acquired images.

① Before the collapse took place, rockfalls began at the lower-left of the slope that a former collapse occurred. The rockfalls extended to the entire slope right before the collapse took place.
② The collapse initiated from the vicinity of scarp at the middle-left of the slope, which was formed at the stage the 2nd collapse (1.83 seconds from the beginning of the collapse). At the same time, the head of landslide began to move very slowly.
③ The entire slope began to move at a low speed around 2.7m/s. The middle-left part moved at a little fast speed about 5-6m/s (7.33 seconds from the beginning of the collapse).
④ Afterwards, the head and middle part still moved dully, the movement could be observed that height differences occurred gradually inside the movable body and several blocks were generated (7.83 seconds and 8.83 seconds from the beginning of the collapse).
⑤ Also, the moving masses were crushed, and the debris dropped from the head moved at a speed faster than the surrounding (9.40 seconds).
⑥ At the vicinity of the central part of sand movement range, a relatively fast velocity field about 4-6m/s or more appeared. This part is the masses dropped from the head of the collapse.
⑦ Then, the entire landslide body collapsed and the moving range of the masses extended. The falling masses accelerated further and came to cover the lower part of the slope. Compared with the middle and lower part, the upper part of the slope still moved at a low speed.
⑧ Afterwards, the movement of the slope head accelerated a little, and the moving range of the masses shifted downward.
⑨ The collapse came to the end after 28.33 seconds from the beginning of the collapse. Although small collapses and falling rocks still lasted, the collapse that greatly changed landform shape did not occur, and the final landform shape formed then.

According to the analytical results, the surface movement situation of the landslide has been clarified, and it is suggested the possibility to estimate the inner movement of a landslide blocks.
4.2 Ootou landslide

375 scenes of the video image sequence of the Ootou, Nara Prefecture landslide recorded from the beginning of slope collapse in about 12.5 seconds are analyzed. The image measurement area of 333×339 pixels is set according to the visually interpreted collapse range. Within the measurement area, 15 by 15 traverse lines are set in vertical direction and horizontal direction, respectively, and all the intersection points are used for the measurement of velocity distribution. Also, the measured speed is converted to a real velocity by scale setting. The template size of 16 by 16 pixels and the search range of 32 by 32 pixels are set in the measurement.

Although the collapse is in a short period of time, also, the video pictures are taken from the side of the collapse slope and the actual landslide slope can not be seen, however, the progress of the slope collapse can be presumed by measuring the movement of the trees from the landslide images. Moreover, since the influence of noise, the mis-measurement of velocity vectors in the black upper half part of the image can be seen from the beginning several scenes, the velocity measurement is performed almost well generally.

The following points can be made out from the result of image measurement.

① The poles and trees in the measurement area began to move very slowly downward to the left. The measured velocity is around 1.2m/s after 0.23 seconds from the beginning of the collapse.
② A relatively low-speed movement lasted for a while. The measured velocity is around 1.6m/s after 1.23 seconds from the beginning of the collapse.
③ The low-speed movement continued for a while. After 4.17 seconds from the beginning of collapse, the tendency that the falling speed accelerated a little can be noticed. The measured velocity is around 3.3m/s after 4.17 seconds from the beginning of the collapse.
④ The falling speed accelerated, the measured velocity is around 4.9m/s after 5.7 seconds from the beginning of the collapse.
⑤ The accelerated tendency continued, and the measured velocity is around 6m/s after 7.1 seconds, and 6.3m/s after 8.43 seconds from the beginning of the collapse.
⑥ Afterwards, the deceleration tendency began to appear, the measured velocity is around 4.9-5.3m/s after 10.6 seconds from the beginning of the collapse.
⑦ The deceleration tendency continued, the measured velocity is around 3.3-4.9m/s after 11.43 seconds, and 3.3-4.2m/s after 12.37 seconds from the beginning of the collapse.
⑧ After 12.5 seconds from the beginning of the collapse, the movement of the trees in the measurement area is considered not to be falling but tilting down, the measurement is put to end.
Figure 6 shows the speed change at the center of the image. A constant speed movement of about 1.2-1.6m/s is shown at the beginning. Then, a uniformly accelerated motion is presented three seconds later, and the speed rises rapidly. The speed decreases after it reaches 6.3m/s and faces to the end. The process, which the landslide collapse initiates from a creep movement, and accelerates due to gravity, then, comes to the end after deceleration, is understood from the speed change chart.

5. Conclusions
This research is a part of the so called project "Joint research on understanding landslide behavior" executed by the authors. In this study, the two-dimensional video image sequence is adopted to a landslide slope collapse analysis. The feasibility of the PIV analysis for landslide behavior understanding is examined, and the conditions of image acquiring and analysis are investigated.

For future study, sequential stereo image pairs will be applied to landslide collapse analysis, and three dimensional landslide behaviors will be investigated.

The authors would like to thank the Etsumi Mountain Area Sabo Office, Chubu Regional Development Bureau, and the Kinki Regional Development Bureau, Ministry of Land, Infrastructure and Transport for providing the video images.

References


The Groundwater Tracing Investigation with High Density Oxygen as a Tracer

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・HANAOKA Masaaki (NEXCO,JAPAN)

Abstract. The purpose of this study is to examine the effectiveness of high-density oxygen gas for tracing groundwater flow.

To grasp groundwater flow accurately is important for efficient landslide groundwater drainage works. Generally, it has been employed tracer such as salt, chemicals, dyes, but these tracers are considered to make bad influences to the groundwater occasionally. Therefore, we developed a new method for groundwater tracing, using high-density oxygen as a tracer instead of others. As the result of this new method we had examined in several landslide sites in Japan, and obtain effectiveness of this method.

Keywords: groundwater, landslide, tracer, oxygen.

1. Introduction.

In order to plan and carry out efficient landslide groundwater drainage works, it is important to grasp exactly the direction of groundwater flow in landslide area, and to design groundwater drainage works.

When we generally examine groundwater tracing, salt, chemicals, organic dyes, etc. have been used as a tracer for several decades in Japan. But these tracers may make bad influences to the groundwater occasionally. In addition, making reagent, the preparation in the outdoors and the analysis takes more time.

The method using high-density dissolved oxygen as a tracer is developed as a groundwater tracer so as to solve the demerits described as above. The most advantage of this method is that results are obtained immediately in the field.

We have examined the new method in four landslide sites in Japan, since 2004, and clarified the method is effective. In this paper, we describe the method and results of the investigation conducted in Okubo-jizo, central Niigata Prefecture, Japan.

2. Method and study area

2.1. Method

The idea of this method is simple: after raising an oxygen concentration of the groundwater in the landslide site using an oxygen supplier into a borehole on the upper part of landslide slope, the dissolved oxygen concentration in borehole at the lower part of the landslide site is detected. Oxygen gas is supplied to groundwater in borehole from an oxygen cylinder, on the upper part of landslide site, and it could raise oxygen concentration of groundwater in the borehole. The dissolved oxygen concentration of groundwater is measured periodically at boreholes on the lower part of landslide slope, using a dissolved oxygen meter (UC12-H). And the detection of oxygen tracer is judged and the three-dimensional direction of groundwater is investigated (Fig.1). The Snow Avalanche and Landslide Research Center, Public Works Research Institute, Japan devised the technique concerned in this paper.

Schematic figure of this study is shown in Fig.1. The procedure of this study is as follows.
1) Determination of an oxygen-supply borehole.
2) Measuring groundwater table and measuring background (B.G., for short) of dissolved oxygen concentration (DO, for short) every 25 cm.
3) Supply oxygen gas to borehole, and measure DO every 25 cm in each measuring borehole.
4) Making a graphic chart-relationship of DO variation and time, depth.

Oxygen supplying time should be 5 hours. Observation interval is set to be 1hr. - 2 hrs, and it should be measured continuously for a half day-6days.

As concerns detection-reference, empirically it decided to be increased more than 3-5mg/L compared to the BG value, and to be continuously measured twice or more in general.
2.2. Study area
The landslide site we studied is Okubo-jizo in central Niigata Prefecture (Fig.2). The Mountains in this study site is less than 500m.a.s.l. and most is 450-400m.a.s.l. Okubo-jizo landslide has of 450m in width, 600m in length, and about 30m in thickness. Landslide movement is mostly stable according to pipe strain gage observation by Yuzawa Sabo (2007).

Landslide material consists of Tertiary weathered mudstone, sandstone, and interbedded layers of them. And debris derived from main scarp covered surface of landslide. Bedrock of the landslide site consists of Tertiary mudstone, sandstone, and interbedded layers of them. Dip of the layers range 40 – 45 degrees, and are inclined to SSE-SE. An anticline axis trend of NNE-SSW is located east of the landslide. There are many joints characterizes landslide material and a basement rock.

Gradient of a landslide surface is 20 degrees in general; the toe of landslide has 40 - 45 degrees and reaches the Higashi river (Fig.2).

Land use of this area is mainly paddy field and Japanese carps feeding ponds.

3. Result
Field investigations carried out in January 25th, 2008 to February 30th, 2008. In consideration of Okubo-jizo area is relatively large, three boreholes were determined for supply oxygen, 16-4, OKZ-3, and OKZ-9. There are located on north side, center, and southern side of landslide. And the groups of boreholes for DO measuring were TR-1, TR-2, and TR-3 (Fig.2, Table 1).

Before oxygen supplying, DO of three above-mentioned boreholes were 0.0-4.8mg/L. Immediately after supplying oxygen, the observed value was 50 – 65mg/L in observed depth. High dissolved oxygen concentration which it is satisfied with the reference value was detected in OKZ-5 of TR-3, as a result of investigation (Table 2).

Fig.3 shows the depth and DO concentration relationship of borehole (OKZ-9) in which oxygen was supplied. According to this figure, DO concentration increases after oxygen supplying, and showing a mostly steady value, ca. 65mg/L, except near groundwater surface (February.2nd.). After that, DO concentration decreases with the time almost uniformly about all the depth within a borehole.

Fig. 4 shows the variation of DO concentration in oxygen, the observed value was ca. 50 – 65mg/L in observed depth.

![Table 1 Result of this investigation](image)

<table>
<thead>
<tr>
<th>Name of tracing group and depth of oxygen supply borehole (below G.L,m)</th>
<th>TR 1</th>
<th>TR 2</th>
<th>TR 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>OKZ-3 40</td>
<td>OKZ-7</td>
<td>OKZ-9 44</td>
<td></td>
</tr>
<tr>
<td>16-4 20</td>
<td>OKZ-7</td>
<td>OKZ-9 44</td>
<td></td>
</tr>
<tr>
<td>16-2</td>
<td>OKZ-5</td>
<td>OKZ-3</td>
<td></td>
</tr>
<tr>
<td>Name of DO observation borehole, depth of detection (below G.L,m)</td>
<td>OKZ-5</td>
<td>OKZ-8</td>
<td>OKZ-11</td>
</tr>
<tr>
<td>OKZ-7</td>
<td>OKZ-11</td>
<td>OKZ-11</td>
<td></td>
</tr>
<tr>
<td>OKZ-8</td>
<td>16-3</td>
<td>16-3</td>
<td></td>
</tr>
<tr>
<td>OKZ-11</td>
<td>OKZ-10</td>
<td>OKZ-10</td>
<td></td>
</tr>
<tr>
<td>16-3</td>
<td>OKZ-10</td>
<td>OKZ-10</td>
<td></td>
</tr>
<tr>
<td>OKZ-9</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

© detected
× non-detected
OKZ-5, the observation depth is 40 to 45 m. When Oxygen supplied into the borehole OKZ-9 on January 28th, DO concentration OKZ-5 was 0-3.0mg/L. Four days later, DO of OKZ-5 was increased greater than 5mg/L relatively, and reached 8-10mg/L. And high DO concentration has been observed continuously twice or more. We made the Fig.4 with eliminating values from observation, because values have some errors by operational accident of DO meter. Precipitation and river water percolation influences oxygen concentration in groundwater, therefore we carefully analyzed the data. Fig.5 is giving an example for non-detection of DO concentration of OKZ-6. The result showing that variation in each depth is small, and observation values are 0.2 ~ 0.7mg/L during observation.

**Fig.3** Relationships between depth and concentration for every observational day, the case of OKZ-9, TR-3.

**Fig.4** The variation of high DO concentration. Observation depth is between 40-45m of OKZ-5, TR-3

**Fig.5** Non-detected high DO concentration. Depth of observation between 35 to 40m of OKZ-6.

### 4. Discussion

#### 4.1. Direction of groundwater flow

Table 2 shows the result of detection in OKZ-9. Estimated direction of groundwater flow is from OKZ-9 to OKZ-5, showing as in Fig. 6. In addition, groundwater table contour line is also shown in Fig.6, based on the groundwater table observation on December 26th 2007.

**Table 2** Estimated groundwater flow direction

<table>
<thead>
<tr>
<th>borehole</th>
<th>depth</th>
<th>time duration</th>
<th>B.G.</th>
<th>borehole</th>
<th>depth</th>
<th>value</th>
<th>time/distance/velocity</th>
</tr>
</thead>
<tbody>
<tr>
<td>OKZ-9</td>
<td>40m</td>
<td>Dec 26th, 2007</td>
<td>48mg/L</td>
<td>OKZ-5</td>
<td>40-50m</td>
<td>10mg/L</td>
<td>109m 5/10 cm/sec</td>
</tr>
</tbody>
</table>

Direction of groundwater flow in cross section of “A” line (see Fig.6) shown in Fig. 7. Both figures indicated that:

1. Groundwater direction is located on sparse groundwater table contour lines around the central part of landslide. Direction of groundwater flow intersects perpendicularly to the groundwater table contour line in general. Therefore, direction of groundwater flow has brought the accordance with the groundwater table contour line (Fig. 6).

2. Direction of groundwater flow is located around the boundary of relatively low permeable mudstone and relatively higher permeable sandstone (Fig. 6 and 7). Direction of groundwater flow is near a slip surface generally (Fig. 7). According to geological investigation, bottom elevation of weathered mudstone on slip surface is lowest on B line. Therefore, groundwater flows relatively higher permeable sandstone, and concentrates to lower part of B line.

Groundwater flow velocity is approximately $10^{-2}$ cm/sec. Landslide material consisted of weathered mudstone has generally low permeability, therefore existence of a groundwater flowing path is estimated.
4.2. Joint plane and groundwater flow direction

The rose diagram showing in Fig. 6 is joint plane direction near main scarp under crown at Ookubo-jizo landslide. It indicates that the direction of joint plane of ESE is parallel to groundwater flowing direction. Therefore, groundwater flowing direction is regulated by joint system. In addition, direction of main ridge is also parallel to joint plane of WNW-ESE. It also indicates that shape of the landslide has regulated by joint system.

4.3. Groundwater flow as a factor for landslide

According to a field observation and geomorphological analysis, and direction of groundwater flow, topography of landslide around B line is estimated to be younger than the others A or C line. Concave counter lines of upper part and lobe form of toe are judged to have removed. According to the geological investigations, heavy weathered mudstone and a colluvial deposit are thick here.

Moreover, it is reported that the groundwater level is high. Weathering advance and a part of landslide is presumed to have reactivated, because groundwater has concentrated in the direction of B line.

5. Conclusions

According to investigation in several landslide sites, we achieved results as follows. A new method using high-density oxygen is a useful method for catching groundwater flow in landslide sites. The maximum traceable distance in 4 landslide sites in Japan is about 300 m.

Groundwater flowing direction is grasped in 3-dimensionally in landslide sites by this technique, while the usual tracer, such as salt, only grasped the flow direction in 2-dimensionally.
Mapping and Assessment of Pre- and post-landslide Using High-resolution 3D Satellite Remote Sensing

Ken Tsutsui (NTT DATA Corp., Japan) • Hideaki Nakagawa (Center of Urban Infrastructure, Environment and Resources, Japan) • Shuichi Rokugawa (The University of Tokyo, Japan) • Katsuaki Koike (Kumamoto University, Japan)

Abstract. This paper presents a new technique for three-dimensional mapping and quantitative assessment of pre-and post-landslide that uses high resolution satellite remote sensing. Digital elevation model (DEM) based on satellite remote sensing are expected to become the premier tool for 3-D topography observation due to their wide area coverage ability and periodicity. We studied a method for highly accurate DEM generation from high-resolution satellite imagery and applied DEMs to the mapping and assessment of landslides. The technique consists of two landslide applications. 1) One application is the surface change observation by using temporally spaced DEMs and provides the landslide location, depth, and volume over large areas needed for post-landslide assessment. 2) The other application is the pre-landslide hazard mapping by topographic analysis of DEM over large areas.

Three cases of post-landslide application in recent earthquake-induced and rainfall-induced landslides (2004 the Mid Niigata Prefecture Earthquake, 2005 Typhoon Nabi in Japan and 2004 Typhoon Mindulle in Taiwan) show the effectiveness of the proposed technique because the volume of eroded and accumulated materials can be estimated based on the detected elevation changes. A case study of a slope stability analysis for pre-landslide assessment in Miyazaki Prefecture shows a potential of the technique to support hazard mapping. The developed technique is quite effective in assessing landslides over large areas because the three-dimensional and quantitative indexes for landslide assessment can be determined by remote sensing.

Keywords. Digital elevation model, satellite remote sensing, landslide assessment, hazard mapping

1. Background

Landslides triggered by heavy rainfalls and earthquakes are one of the serious geological hazards in the mountainous areas in the Asian region. In recent years, the damages have been increased by the number of extremely powerful typhoons generated by the recent global abnormal weather conditions. Due to the spatial extension of steep topography in the areas, it seems effective to develop a hazard monitoring system based on remote sensing that can provide early mapping and assessment over large areas.

Satellite remote sensing is a powerful tool that provides wide area and periodic monitoring for many geohazard applications such as mapping distributed landslides to target investigation sites and to make damage restoration plans. Many landslide applications based on satellite remote sensing have been developed. However, there is a major limitation; quantitative damage assessment is lacking because the two-dimensional satellite imagery could not provide the quantitative topographic measurements needed. The Quantitative landslide measurements such as depth, movement, volume and slope indexes are essential in all phases of disaster management. The acquisition of three-dimensional (3-D) topography including surface elevation is necessary for these quantitative assessments.

This paper introduces our studies on a new technique for landslide mapping and assessment that uses high accurate DEM (Digital Elevation Model) constructed from high resolution satellite imagery. We studied a method to generate high accurate DEM and two landslide applications. The first application is the elevation change observation by using temporally spaced DEMs and provides the landslide location, depth, and volume over large areas needed for post-landslide assessment. The second one is the pre-landslide hazard mapping by topographic analysis of DEM over large areas.

2. Study areas

Three cases of recent large-scale landslides induced by earthquake or heavy rainfalls in Japan and Taiwan were studied to develop a practical use technique. One was the earthquake-induced landslide in a moderate mountainous area in Mid Niigata prefecture in Central Japan, which was triggered by the 2004 Mid Niigata Prefecture Earthquake (M6.8) on October 23. Another one was the rainfall-induced landslide in a steep mountainous area in the upper Dajia River basin in Central Taiwan, which was caused by the typhoon Mindulle in July 2 to 4, 2004. The other one was also the rainfall-induced landslide at Mt. Wanizuka in Miyazaki prefecture in Southern Japan, which was caused by the typhoon Nabi that struck on July 6, 2005. The areas description is listed in Table 1.

Fig.1 The location of the study areas.

- 109 -
Table 1 Description of the study areas

<table>
<thead>
<tr>
<th>Area name</th>
<th>Mid Niigata</th>
<th>Dajia River</th>
<th>Mt. Wanizuka</th>
</tr>
</thead>
<tbody>
<tr>
<td>Geohazard</td>
<td>Landslide</td>
<td>Landslide,</td>
<td>Landslide,</td>
</tr>
<tr>
<td></td>
<td></td>
<td>debris flow</td>
<td>debris flow</td>
</tr>
<tr>
<td>Cause</td>
<td>Mid Niigata Earthquake (M6.8)</td>
<td>Typhoon Mindulle (1200mm /5days)</td>
<td>Typhoon Nabi (1300mm /3days)</td>
</tr>
<tr>
<td>Date</td>
<td>October 23, 2004</td>
<td>July 2-4,</td>
<td>July 6,</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2004</td>
<td>2005</td>
</tr>
<tr>
<td>Ave. slope angle(deg.)</td>
<td>20</td>
<td>37</td>
<td>24</td>
</tr>
<tr>
<td>Geological age</td>
<td>Late Miocene to Pliocene</td>
<td>Eocen to Oligocene</td>
<td>Paleogene to early Miocene</td>
</tr>
<tr>
<td>Rock type</td>
<td>Mudstone, sandstone</td>
<td>Sandstone,</td>
<td>Sandstone,</td>
</tr>
<tr>
<td></td>
<td></td>
<td>slate, shale</td>
<td>shale</td>
</tr>
<tr>
<td>Total volume</td>
<td>70,000</td>
<td>30,000</td>
<td>6,700</td>
</tr>
</tbody>
</table>

Table 2 Description of the DEM data

<table>
<thead>
<tr>
<th>Area name</th>
<th>Mid Niigata</th>
<th>Dajia River</th>
<th>Mt. Wanizuka</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area size (km)</td>
<td>10 × 15</td>
<td>20 × 20</td>
<td>10 × 7</td>
</tr>
<tr>
<td>Grid size (m)</td>
<td>5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Elevation accuracy (m)</td>
<td>Approx. 5 (1σ)</td>
<td>(depending on slope angle)</td>
<td></td>
</tr>
</tbody>
</table>

Fig.2 Ground photos of the study areas.

3. Remote sensing data

Pre- and post-landslide DEMs were generated from SPOT5 satellite stereo pair images by a stereo image processing (Tsutsui et al. 2007). SPOT-5 satellite was launched in 2002 by the Centre National d’Etudes Spatiales (CNES) in France and its HRG sensor outputs 2.5- or 5-m-resolution panchromatic images with a swath of 60 km. Stereo pair images can be acquired by taking more than two HRG images on the same area from different orbits. The specifications of the generated DEMs are shown in Table 2. The elevation accuracy depends on the slope angle (Tsutsui et al. 2007). The RMSE of relative height accuracy, which means the height error in the elevation differences between two temporal DEMs, was 4-5m on slopes under 30°, whereas for slopes over 30°, the RMSE was 5-9m. The absolute accuracy was about 5m relative to airborne light detection and ranging (LIDAR) DEM at the Niigata area, which nearly equals that of the relative accuracy. Examples of the extracted DEMs are shown in Fig. 3.

Fig.3 The extracted DEMs at the Mt. Wanizuka area.
3. Quantitative landslide mapping for post-landslide assessment

The measurement of quantitative landslide indices such as depth, movement and volume are essential in all phases of post disaster management. We applied the satellite based wide coverage DEMs to surface change observation and landslide volume estimation.

Pre- and post-landslide DEMs were used for elevation change detection and volume estimation. The analysis flow was as follows. First, the elevation changes were calculated between two temporal DEMs. Second, the potential elevation error areas (defined as image matching errors in the DEM generation process) in the detected elevation change areas were removed. The low contrast areas on the post-landslide satellite imagery, which were thick forest areas or showed areas, were also removed to reduce the errors; the elevation changes on the bare surface areas after the landslides were extracted. Last, the elevation change map was generated and landslide volume was estimated based on the detected elevation changes.

The elevation change map at the Mt.Wanizuka area is shown in Fig. 4. Many landslides appear to be combinations of positive and negative elevation changes in the both three areas: the negative elevation changes were detected in the depletion zones, and the positive changes were detected in the accumulation zones. Based on the elevation changes maps, the volume of each landslide could be estimated in units of $100 \times 10^3 \text{ m}^3$ in the Niigata area and the Dajia River area (Tsutsui et al. 2007), and the total volume in river basin could be estimated in units of $1,000 \times 10^3 \text{ m}^3$ in the Mt. Wanizuka area. These results show that the proposed method is applicable to the quantitative assessments for large scale landslides.

4. Slope stability mapping for pre-landslide assessment

The measurement and mapping of quantitative landslide risk indices such as slope stability and debris deposit are essential for pre-landslide assessments. We applied the satellite based DEMs to estimate the slope stability. In this application, the slope stability was analysed three dimensionally at the intervals of 20m grid based on an infinite slope stability model (Takahashi 2004, The Japanese Geotechnical Society 2006), which provides safety factor (SF) defined by the ratio of shear strength to shear stress under the condition of infinite slope. SF depends mainly on slope angle, geological type, seepage and water surface. Geological types were collected from the ground survey and geological map. Seepage value and water surface value were estimated using Darcy’s law and Manning’s Equation, respectively.

The slope stability map derived from the pre-landslide DEM at the Mt. Wanizuka area is shown in Fig. 5. Many slope collapses appear to be low SF successfully; whereas there are some miss detection areas defined as high SF in slope collapses. Main reason for these miss detections is that the simple SF model is not applicable to the deep seated landslide. An improved stability model, which can be used for complex geological conditions, is necessary to assess them. Our future studies will address the model improvement and the development of an efficient hazard mapping method based on combination of remote sensing and ground geological survey.
Conclusions
A new technique for landslide mapping and assessment that uses high accurate DEM extracted from satellite imagery is introduced. The following findings were obtained from our case studies.

1. Pre- and post-landslide DEMs over large areas can be generated high-accurately using high resolution satellite remote sensing. Based on our results on DEMs extracted from 2.5m resolution satellite imagery, the elevation accuracy was approximately 5m (1σ).

2. Two-temporal DEMs were used for elevation change detection and volume estimation. Three case studies (2004 the Mid Niigata Prefecture Earthquake, 2005 Typhoon Nabi in Japan and 2004 Typhoon Mindulle in Taiwan) show the effectiveness of the proposed technique because the volume of eroded and accumulated materials can be estimated based on the detected elevation changes.

3. The slope stability was assessed three dimensionally using pre-landslide DEM. A case study for pre-landslide assessment in Miyazaki Prefecture shows a potential of the technique, whereas it is needed to improve the applicability to deep seated landslides.

The developed technique is quite effective in assessing landslides over large areas because the three-dimensional and quantitative indices can be determined by remote sensing. Our future studies will address the development of a practical hazard mapping by combination of remote sensing and ground survey.

Acknowledgments
This research is supported through a Research Grant from Foundation of River and Watershed Environment Management in 2007.

References


Analysis of Surface Deformation Induced by the Noto Hanto and the Chuetsu-oki Earthquakes in 2007 using Synthetic Aperture Radar Interferograms

Hiroshi UNE (College of Land, Infrastructure, Transport and Tourism, Japan), Hiroshi P. SATO, Mamoru KOARAI, Hiroshi YARAI and Mikio TOBITA (Geographical Survey Institute, Japan)

Abstract. The authors have carried out Synthetic Aperture Radar (SAR) interferometric analyses and succeeded to identify the characteristics of the land changes related to the Noto Hanto and the Niigataken Chuetsu-oki Earthquakes in 2007 such as the crustal deformation due to the coseismic slip on the faults and the local landslides and lateral flows triggered by the ground shaking.

Keywords. Advanced Land Observing Satellite, synthetic aperture radar, interferometry, crustal deformation, initial process of landslide

1. Introduction

Two large earthquakes hit the northwestern part of Japan in 2007, the Noto Hanto Earthquake on March 25 with a magnitude of 6.9, and the Niigataken Chuetsu-oki Earthquake on July 16 with a magnitude of 6.8. The occurrence of these earthquakes provided us the first opportunity to apply ALOS/PALSAR data for detection of earth surface deformation practically since the start of the operational phase of the Advanced Land Observing Satellite (ALOS) “Daichi.” PALSAR, the synthetic aperture radar (SAR) system loaded on ALOS, provides us more detailed information on the displacement of the ground surface ever than before. Consequently, it enabled us to detect the information on local landslides and soil deformation that had been removed as the noise in the process of clarifying wider-area coseismic crustal deformation. The Geographical Survey Institute (GSI) has carried out SAR interferometric analyses and succeeded to identify the characteristics of the land changes related to these earthquakes such as the crustal deformation due to the coseismic slip on the faults and the local landslides and lateral flows triggered by the ground shaking.

2. SAR Interferometry for Detection of Crustal Deformations

Since 1994, GSI has been conducting a study on applications of differential SAR Interferometry (InSAR) for the detection of crustal deformations associated with earthquakes and volcanic activities. Through correlation study using various pairs of SAR interferograms, we have confirmed that L-band SAR provides better coherence in vegetated area and smaller temporal decorrelation effects than C-band. ALOS/PALSAR is the only L-band SAR system in operation at this moment with the wavelength of 23.6cm, and has been generating a number of valuable results for the detection and analyses of crustal deformations.1)

Fig. 1 shows the general concept of the detection of crustal deformation using SAR interferometry. Changes of the distances between the ground surface and the satellite are represented as the interferometric fringes of the phase differences. A fringe cycle of phase differences corresponds the displacement relative to the satellite of a half of the wavelength, i.e. 11.8cm.

Fig. 1. Detection of crustal deformation using InSAR

3. Noto Hanto Earthquake (March 25, 2007)

On March 25, 2007, a disastrous earthquake with a magnitude of 6.9 struck Noto Peninsula, 320 km northwest to Tokyo, Japan (the Noto Hanto Earthquake in 2007). Its epicenter was decided off the northwestern coast of the peninsula. GSI has conducted various surveys and analyses to clarify the crustal deformation associated with the earthquake including the InSAR using the PALSAR data by ALOS.

We processed PALSAR data from ascending orbit acquired on February 23, 2007 and April 10, 2007 to generate an interferogram applying differential InSAR method. The result shows large movements of around 50 cm toward the satellite near the epicenter, with concentric...
fringes indicating elastic coseismic crustal deformation (Fig. 3(a)). The line of sight unit vector (EW, NS, UD) = (-0.728, -0.081, 0.681), with the east, north and upward vectors being positive. In Fig. 3, movements toward the satellite are taken to be negative.

Based on the detected crustal deformation, we estimated rectangular fault model, which trends from the northeast to the southwest and is 22 km long and 11 km wide. We also estimated a variable slip model on a fault patch that is 61 km long and 25 km wide, covering the above-mentioned rectangular fault (Fig. 4).

Besides, a number of small oval and horseshoe patterns of phase variation reflecting local deformation of land surface are found in the same interferogram. To isolate local deformation from wide-area crustal movement, we produced a residual image (Fig. 5) by subtracting the calculated elastic displacement based on the simulation of source fault model as shown in Fig. 3(b) from the observed one (Fig. 3(a)). We overlaid this image on our 1:25,000 topographic maps and landslide distribution maps (Fig. 6) and found that the distribution and the direction of movement of such deformation patterns are consistent with existing landslides. We carried out field surveys and clarified that these patterns reflect the landslides triggered by the ground shaking. The amount of the movement is between a few cm to a few tens cm. The largest one found in the image is at Furue region in Nanao City, with a width of 1.5km and a depth of 0.7km (Fig. 7). In order to detect the characteristics of such deformation, we processed another pair of PALSAR data acquired on December 12, 2006 and May 10, 2007 from descending orbit with the line of sight unit vector (0.618, -0.112, 0.778), and executed 2.5-D analysis from two different tracks interferograms using the method after Fujiwara et al. (2000)\textsuperscript{3} to resolve the displacement into...
Fig. 6. Residual image overlaid on topographic map data

Fig. 7. Deformation Pattern in Furue, Nanao City

Fig. 8. Results of 2.5D displacement analysis. (a) shows the vertical component (minus: subsidence, plus: uplift) and (b) shows the horizontal component (minus: westward, plus: eastward). (c) shows surface displacement vector along the line in (a) and (b).

Fig. 9. Interferograms for Chuetsu-oki Earthquake. (a) is descending result for the period between January 16 and July 19, 2007 and (b) shows ascending result for the period between June 14 and September 14.

4. Chuetsu-oki Earthquake

On July 16, 2007, another disastrous earthquake with a magnitude of 6.8 struck Chuetsu area along the coast of Japan Sea, 220 km northwest to Tokyo, Japan (the Niigataken Chuetsu-oki Earthquake in 2007). Its epicenter was at about 10 km off the coast. GSI again carried out various analyses to clarify the crustal deformation associated with the earthquake including the InSAR using the PALSAR data by ALOS.

We generated two interferograms by processing the PALSAR data from descending orbit acquired on January 16 and July 19 and from ascending orbit acquired on June 14 and September 14. The interferogram for the descending orbit with the line of sight unit vector (0.637, -0.113, 0.762) (Fig. 9(a)) clearly shows the coseismic deformation, a maximum of 25 centimeters of uplift near the epicenter and occurrence of subsidence in southern part of the affected area. In the interferogram for the ascending orbit with the unitary vector of line of sight (-0.620, -0.109, 0.777) (Fig. 9(b)), we identified not only the peak of deformation near the epicenter but also rather local strip-shaped deformation zone in the Nishiyama Hill, southeastern part of the area. The local deformation was supposed to reflect the aseismic growth of a folding structure triggered by the stress changes associated with the earthquake.4)
We also produced a residual image using the same method applied for the Noto Hanto Earthquake, and compared it with a landform classification and damaged map (Fig. 10). We found several characteristic patterns indicating subsidence and horizontal displacement of land, probably due to liquefaction and lateral flow of sandy soil triggered by the ground shaking (Fig. 11). Most of such patterns were located at the edge of the sand dunes and natural levees on low land. We consider that such liquefaction and lateral flow of sandy soil at the edge of sand dune were among the reasons for the concentration of heavy damages to buildings.

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References

Geomechanical Modeling Test on Deformation Fracture Mechanism of Mountain body Caused by Earthquake

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Abstract: The deformation fracture of mountain body caused by earthquake is a complicated process. In this paper, three typical geomechanical modes are summarized and concluded, i.e. the outer-inclining bedded landslide model, the high-steep block (layer) bedded slope collapse model and the weak base landslide model, which are representative for the deformation fracture and instability of mountain body in areas with high earthquake intensity. We have also designed a set of geomechanical modeling test equipment, and developed corresponding models and techniques under vibration so as to conduct the mechanism test of the three typical geomechanical models. Through the modeling reproduction test of the process of deformation fracture, its typical characteristics under vibration are shown. Formation conditions, relevant factors and evolvement principles of typical instability mechanism due to earthquake are presented to offer scientific and reliable test data and a basis for further research and analysis.

Key words: Areas with high earthquake intensity, Deformation fracture mechanism of mountain, typical geomechanical modes, Outer-inclining bedded landslide model, High-steep block (layer) bedded slope collapsing model, weak base landslide model, Geomechanical modeling test

1. Geomechanical Model Test Study on Typical Deformation Fracture
1.1 Modeling Test on Outer-inclining Bedded Landslide Mechanism

The outer-inclining layer slope model (Fig.1) is based on the relatively long consequent slope structure, and is stacked with sandstone block of 5cm cubed and with a rock formation dip angle of 10°-40°. The blocks are bonded by cohesive soil, with a gradient of 60°-70°. The base is a 10°-40° slope made of barite powder. The rectangular protractor of the model shall be 45cm long, 38cm high and 40cm wide with a scale of 1:1000.

1.2 Modeling Test on High-steep Bedded Slope Collapse Mechanism

High-steep bedded slope models(Fig.2) include high steep weathering and unloading zone model of horizontal terranes, and reverse ones. Bevels at different angels made of barite powder in the model bottom are used as the bases. Sandstone blocks of a 5 cm long are stacked to some height corresponding with the dip angles of different terranes, between which clay soil is used to cling. And on the top, there are thin rocks stacked together which are 5 cm in length and width, and 2.5 cm in height, and 5 cm in length and 2.5 cm in width and height. They stands for intense weathering and unloading zones incised by joint fissures, whose thickness in general ranges from 5 to 20 cm. The gradient of slopes are 60-80°. The rectangular size of the model are 35 cm in length, 45 to 75 cm in height and 40 cm in width. The model scale is 1:2000.

1.3 Modeling Test on Weak Base Landslide Mechanism

The prototype of weak base frame slopes or earthy slopes model (Fig.3) is designed according to the landslide geological structure of Ganhaizi in Diexi earthquake zone. The model is made from the mixture of talc
2. Conclusions

Through the modeling tests under vibration, the formation condition, the mechanism and process of the three typical geomechanical modes are demonstrated, and the results are as follows:

(1) The deformation fracture appears generally in inflection point of the vibration track, namely the inflection point at which direction of movement changes. When displacement reaches critical value instability comes into being. Deformation fracture is closely related to first motion direction.

(2) The vibration of elliptical track has the biggest influence on the sloping ground deformation fracture; Horizontal vibration track comes second; sloping vibration track comes third; Vertical vibration track basically has no obvious effect on deformation fracture. The general instability appears at the first inflection point after the first peak value of acceleration and direction change of a circulation.

(3) With respect to outer-inclining layer slope and high steep layered slope in weathering and unloading zones, when horizontal acceleration of vibration reaches 0.4g, the deformation fracture comes into being. When horizontal acceleration of vibration is close to 0.8g, wide-rang overall instability will appear. The critical horizontal acceleration of pore water pressure proliferating start is bigger than 0.2g, the critical acceleration causing slip is close to 0.4g. Bigger the amplitude is, larger the time of repeating vibrations is. Longer the duration is, more serious the destructiveness is.

(4) Deformation fracture of outer-inclining layered slope is obviously controlled by structural plane. As soon as the vibration starts, shear crack and tension crack come into being along weak plane of structural plane. And those cracks expand to deep part, causing slipping tension fracture displacement accumulation.

(5) Swaying in high steep outer-inclining and inside-inclining block (stratified) slope is more intense than that in horizontal block (layer) high steep slope. High steep horizontal block (stratified) slope is generally burst in the form of compression crack. After holing through is snipped instability comes into being in the form of collapse. Highly steep outer-inclining and inside-inclining block (stratified) slope is generally burst in the form of curve drawing crack. After holing through breaks away from matrix instability comes into being in the form of collapse.

(6) The swaying response peak value in high steep slope has obvious enlargement phenomenon (vertical enlargement) compared with the bottom of the model, the sway response peak value in characteristic point of the edge position of slope also has enlargement phenomenon (horizontal enlargement) compared with the interior. Intense degree of swaying is intensified along with the elevation difference and the increasing slope.

(7) The pivot position of swaying plate girder of high steep slope appears generally on the position of 1/3~1/2 of slope height apart from toe of slope, the height of breakage (collapse) appears generally on the position of among 1/3~1/2 of slope height apart from toe of slope.

References


Paleolandslides a Central Part of East European Plain (Russia)

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Abstract. The relief of a central part of East European plain represents a vast smoothed spaces. Upon the whole modern activity of landslides and a landslide hazard within the limits of East European plain are estimated as low. The study of geological condition at the large (volumes from several millions to several tens millions cubic meters) landslides development area within the limits of East European plain show that modern landslides are inherited as a rule. Upon the whole it is possible to identify periods of peak landslides activity in the Miocene, Eopleistocene and Holocene for the area considered. Formation and active development of large paleolandslides were likely to be caused by two major reasons. They were change of base level of erosion, which was accompanied by river valley over deepening and block slide formation in high (more that 300 m) river mouths on the one hand and significant climate transformation at glacial covers degradation. Simultaneously landslide cirques originated earlier were expanded. During subsequent time most of Miocene and Eopleistocene landslides were completely (within the limits of glacier cover) or partially (at large river valleys outside of glacier cover) overlapped by Quaternary sediments. This postglacial sediment cover does not allow to estimate landslide hazard caused by paleolandslides occurrence at East European plain.

Keywords. Paleolandslide, Central Part of East European Plain, Paleovalley

1. Introduction

European part of Russian Federation (fig.1) is characterized by high level of urbanization. There are many cities (Moscow, St.Peterburg, Nizhniy Novgorod, Saratov, Kazan, Rostov, Ulyanovsk, etc.) and a number of cascades of reservoirs in this region. This situation requires intent attention for landslide hazard assessment. The systematic study of landslides developed within the boundaries of European part of Russian Federation begun in second half of XIX century in connection with transport infrastructure development. Upon the whole modern landslide activity and landslides hazard level were assessed as not high. There are the specific river valley segments only within the limits of study area, which slope deformations are limits of the study area, which slope deformations are

2. Modern topographic and geological conditions

The area considered in the paper is located within the central part of the East-European plain. Relief of this area is characterized as leveled. There are wide leveled fields with absolute altitudes from 100-120 m to 200-300 m which smoothly downward to river valleys. The modern geomorphological conditions of the northern part of East-European plain were formed by Quaternary glacial processes and those of southern part of East-European plain outside of the glacial zone boundaries were caused by denudation processes.

The part of East-European plain under our consideration covers two large river basins – of the Volga river (the greater part) flowing into the Caspian Sea and of the Don river (south-western part) flowing into the Sea of Azov. As main stream valleys (Volga, Don) as primary large tributaries valleys (Oka, Kama, Tsna, Sviyaga, Sura) orient either sublatitudinally or submeridionally (Zerkal & Antipina, 2006). Common direction of river drainage is eastward (primary) and south-eastern within the northern “glacial” part of East-European plain and one is southern outside of glacial zone boundaries. Most of large river valleys are asymmetric.

The upper part of geological section of East-European plain is composed the terrigenous and carbonaceous formations from Devonian to Quaternary age. Pre-Quaternary deposits are the typical “platform” formations of transgressive sea basins. There are sand, clay, sandstone, marl, and limestone with different thickness laying subhorizontally or slightly inclined. In the eastern part of area under considerations gypsum and anhydrite are widespread. During Quaternary period in the northern part of East-European plain the glacial conditions prevailed. Donian (the valley type, Early Pleistocene (Mindel)) and Moscowian (the covering type, Middle Pleistocene (M. Riss)) glacial stages were most intensive. Moscowian glacial stage resulted in accumulation of moraine deposits up to 30 m thick and in formation of wide-spreading fluvioglacial deposits. These deposits overlie Pre-Quaternary sediments completely. In the southern part of East-European plain outside of the glacial zone boundaries there are blanket loams and loesses in Quaternary deposits composition. The loess thickness increases at south and may achieve 20 m and more.
3. The history of the relief formation

Upon the whole the general pattern of the East-European plain relief was formed in a few stages. The initial stage that predetermined the regional geomorphological conditions embraced the Late Carboniferous – Early Permian periods. During this period the continental conditions were predominated at the East-European plain (except for its eastern part where sea basin regressed in Late Permian). The surface of late Paleozoic paleo-relief in geological sections is cleanly identified by thick (up to 8 m) weathering crust which is confined to upper part of Carbonian section. A widespread river system was formed which had, generally, eastern flow direction (toward to paleo-Ural ocean). During the next stage (Mesozoic era) the relief that had been formed earlier was overlapped by upper Triassic, Jurassic and Cretaceous terrestrial deposits developed due to several transgressions of the southern sea which represented the shelf seas of the Tethys paleo-ocean. During Oligocene in the central part of East-European plain continental conditions revived. However, this time a newly forming river system had total southern flow direction. At the same time sea deposits laid up in river valleys. These sediments formed as a result of some ingressions of Paratetis sea basins. In the Late Neogene – Early Quaternary the basic features of the modern geomorphological structure were formed.

The comprehensive geological data characterizing Pliocene-Pleistocene geomorphological conditions of the central part of East-European plain have been collected and generalized (Goretsky, 1964, 1966; Paleogeographic Atlas, 1991). In the Middle Pliocene in the central part of East-European plain there were large river paleo-valleys from 90 up to 300 m deep. Some of paleo-valley segments represented narrow canyons like modern Grand Canyon (table 1). In the Late Pliocene (Akchagylian-Gelasian) a widespread Caspian sea transgression occurred. In the southern part of East-European plain terrestrial sea deposits filled the paleo-valleys and leveled Pliocene relief. In the western part of the transgressive shelf sea a coastal cliff formed. After sea regression the paleo-Volga valley developed along this cliff. The depth of the Early Quaternary river valleys was about 50 m – less than that of the Pliocene valleys.

The Pleistocene glaciations series took part in the modern relief formation too. The deposits of the covering Moscow Glaciation (Middle Pleistocene (M. Riss)) overlapped the Pliocene paleo-relief on the northern part of East-European plain completely. The fluvioglacial deposits accumulated toward the end of Middle Pleistocene finalized the relief leveling. Thus, the complete relief rearrangement took place in the central part of of East-European plain. The paleo-relief could be guessed from the modern structure of river valleys only.

3. The paleo-landslides of the central part of the East-European plain

The numerous drilling data obtained during XX century made possible to delineate several regions where landslides where actively formed in the past.

Pseudo-landslides in the Moscva River valley. The Moscva River belongs to Volga drainage basin. Within the limits of Moscow-city there are extended segment of the Moscva river valley (near the so called “Vorobyovy Gory”) where long-time landslide deformations of the valley right bank develop. They are:
- deep-seated landslides that affect entire slope of the valley;
- landslides that affect some part of valley slope only;
- near-surface landslides (shallow landslides).

Deep-seated landslides about 90 m thick with volume up to 20 millions m³ are most hazardous. According to the repeated bench-mark measurements performed by the Geomonitoring Service (Geocentre-Moscow), the average annual velocity of deformations within the limits of active block is 2-6 mm/year and significantly increases during periods of activation.

Landslides near Vorobyovy Gory were described first time by Fisher Valdgame in 1837 (Danshin, 1937). In 1908 A.V.Pavlov suggested that displacement zone of landslides extends below the modern Moscva-river level. In 1937 B.M.Danshin, based on the drilling data concluded that it coincides with Early Pleistocene paleo-channel level (Danshin, 1937). At present time the trial drilling determines that this level is 35 m lower than the modern Moscva River one.

It seems that the landslides formation a this river valley segment started in Early Pleistocene. The sliding surface came out to the paleo-channel level and landslides affect 4 the Cretaceous and Jurassic deposits of the valley side. In Middle Pleistocene till about 20 m thick left by Moscva Glaciation stage blanket the entire area. Subsequent Late Pleistocene – Holocene erosion of the Moscva River stream revealed these ancient landslides and they are destabilized by modern river erosion, which is indicated by repeated geodesic measurements.

Pseudo-landslides in the Kunya River valley. This valley belongs to Volga drainage basin too. The stability of the left valley side was studied during construction of Zagorsk pumped-storage plant (PSP). The relief of its area formed on Middle Pleistocene loamy moraine is, generally, gently sloping. However, in 1979 during of a Power Station foundation pit excavation a landslide of about 1 million m³ in volume started moving. The sliding rate registered by the geological monitoring survey was up to 2-13 mm/year (Yudkevich, 1994). Being located exactly at the construction site, this landslide was studied in detail and drilling revealed the existence of the paleo-Kunya River valley. It was found out that slope deformations inherited paleo-landslides (Samarin&Zerkal, 2004) composed of 3-4 distinct blocks (up to 100-150 thousands m³ in volume) of Cretaceous deposits. They were formed in the Late Pliocene due to the pale-Kunya valley 110 m deep incision. The erosion activity of the fluvioglacial streams developed during Early Pleistocene valley glaciers degradation was responsible for subsequent slope destabilization at the 9 km long valley segment. Total amount of landslide material was assessed to be not less than 20 millions m³.

In Middle Pleistocene in the Pale-Kunya River valley the relief planation caused by ice sheet of Moscow glaciation stage took place. It resulted in the valley slope stabilization. The modern landslide activity near the Zagorsk PSP was caused by construction works. We should point out that timely remedial measures resulted in slope stabilization and successful completion of the hydraulic scheme.

Pseudo-landslides in the Volga River valley. Those landslides in the Volga River valley undergo modern activation, which had been formed in early Pleistocene. They differ from landslides in the Moscva and Kunya valleys that had been originated in Pliocene. Pliocene landslides formed in the canyon-like paleo valley exist in the Vologa Valley too, but they are buried at present and could be found bellow the present day erosion level.
The paleo-landslides subjected to modern activation are located near the Ulynovsk city, south from the Samara city (Novodevichy region), and neat the Saratov city.

<table>
<thead>
<tr>
<th>Paleo-river, age</th>
<th>River valley</th>
<th>Width of paleo-valley</th>
<th>Paleo-cutting depth, height of valley bedrock walls</th>
<th>Paleo-landslides availability</th>
<th>References</th>
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<tr>
<td>Paleo-Kazanka (Kazanka – modern tributary of Volga), ( N_2 )</td>
<td>V-type with stream gradient 1-1.2 m/km (type – middle upland river)</td>
<td>-</td>
<td>90-130 m (up to 52 m from modern relief marks)</td>
<td>It wasn’t described</td>
<td>Kashtanov &amp; Neklidov, 1954</td>
</tr>
<tr>
<td>Kinel-river (paleo-river, located toward to west from modern Volga valley), ( N_2 ) Toward to north from Zhigulevskay upland</td>
<td>U-type &amp; V-type - on the intensive stream cutting sections</td>
<td>From 1.5 km up to 2.5-3.5 km</td>
<td>Height of valley bedrock sides achieved 200-250 m, with slope 13-19°. On the intensive cutting sections the steep slope of valley sides achieved 35°</td>
<td>In the ancient alluvium (N, kn) there are numerous landslide blocks composed Permian deposits with thickness up to 20-30 m which were uncovered by some boreholes.</td>
<td>Goretsky, 1966</td>
</tr>
<tr>
<td>Paleo-Volga-Kama (Pont-Volga river), ( N_2 ) Toward to south from Zhigulevskay upland</td>
<td>U-type &amp; V-type - on the intensive stream cutting sections</td>
<td>Up to 10 km</td>
<td>Up to 300 m</td>
<td>Paleo-landslides were not described.</td>
<td>Zhuteev, 1962</td>
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<td>Zhigulevsky paleo-canyon</td>
<td>Up to 2.5 km</td>
<td>Up to 300 m (the subface of Pliocene deposits – -270 m from modern relief marks)</td>
<td>Important note: (by Zerkal &amp; Samarin) In Pliocene for some times the upper part of paleo-valley represented series of lake basins.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Krasavsky paleo-canyon</td>
<td>1.5 km</td>
<td>Up to 300 m (the subface of Pliocene deposits – -360 m from modern relief marks).</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Paleo-Kunya river, ( N_2 )</td>
<td>V-type</td>
<td>-</td>
<td>Height of valley bedrock sides achieved 110 m, with steep slope 17-35°</td>
<td>The numerous buried blockslides composed of the Cretaceous deposits were revealed by boreholes.</td>
<td>Samarin &amp; Zerkal, 2004</td>
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<td>Paleo-Volga ( Q_1 ) Toward to north from Zhigulevskay upland, near Ulynovsk</td>
<td>U-type</td>
<td>Height of valley bedrock sides achieved 240 m.</td>
<td>The numerous buried blockslides composed of the Cretaceous deposits were revealed by boreholes. Their base is 46 m below the present day daylight surface</td>
<td>Tihvinsky, 2002</td>
<td></td>
</tr>
<tr>
<td>Paleo-Moscow ( Q_1 (?) )</td>
<td>U-type</td>
<td>Up to 35 m, height of valley bedrock sides achieved 85-90 m.</td>
<td>The numerous buried blockslides composed of the Cretaceous and Jurassic deposits were revealed by boreholes.</td>
<td>Danshin, 1937</td>
<td></td>
</tr>
</tbody>
</table>

Table 1. Paleo-valleys of Central Part of East European plain
During Pleistocene the Cretaceous deposits more than 30 m thick exposed in the Volga River valley bedrock sides were involved in landsliding (Recommendations, 1984; Tihvinsky, 2002). The modern reactivation of relict landslides on the river valley slopes composed of bedrock was caused by the dams and reservoirs construction.

Conclusion
Summarizing of the numerous data on buried paleo-landslides in the central part of East-European plain (Russia) allows to select two periods of their formation – Late Pliocene and Early Pleistocene. Paleo-landslides occurred on the slopes of deep paleo valleys. The significant relief reconstruction in the area considered took place in the Early (southern part of the East-European plain) and Middle (its northern part) Pleistocene. These planation periods resulted in slopes stabilization. At present there is an activation of the relic landslides caused by the modern river streams erosion and by construction works that are often carried out without appropriate study of the construction sites, of the relief evolution in particular.

References
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**Cover photo:** Landslides in the Beichuan county triggered by the 2008 Wenchuan earthquake, Sichuan, China (Courtesy by Yueping Yin, China Geological Survey)

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Global joint initiative by United Nations organizations, governmental organizations, non-governmental organizations, and individuals.

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- I symbolizes Cultural heritage at landslide risk.
- C symbolizes a moving landslide mass.
- L symbolizes Retaining wall to stabilize slopes.
- Slight inclination of C symbolizes motion of landslide and the Consortium.

- ICL was established in January 2002 and legally registered as a non-profit scientific organization (No.1300-05-005237) in the Government of Kyoto Prefecture, Japan in August 2002.
- A full-color international quarterly journal “Landslides” was founded by ICL in April 2004. The impact factor of this journal is 0.986 for 2007.
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