Hazard assessment of a landslide reactivated in a residential area in Japan

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Abstract: As urban sprawl compels more and more people to develop residences on formerly uninhabited mountain and river slopes, the question of how to estimate the landslide hazard of such areas arises. A good deal of research about this problem has been conducted in Japan, where this problem is of significant importance due to the great number of landslides that occur each year in the country. This paper will present the hazard assessment of a reactivated landslide that occurred in a residential area near the City of Kyoto. Comprehensive field and laboratory investigations were conducted to estimate vital parameters such as the geological setting of the area, the properties of soils, the rate of landslide movement, the location of the sliding surface, and the ground water level. In addition, a slope stability analysis was performed to estimate the factor of safety for the slope. Based on the results obtained remedial measures were designed.

Résumé: Pendant que la position abandonnée urbaine contraint de plus en plus personnes pour développer des résidences dans les pentes autrefois inhabitées de montagne et de fleuve, la question de la façon estimer le risque d'éboulement de tels secteurs se pose. Beaucoup de la recherche au sujet de ce problème a été conduit au Japon, où ce problème est d'importance significative due à le grand nombre d'éboulements qui se produisent tous les ans dans le pays. Ce papier se concentrera sur l'évaluation de risque des éboulements réactivés, qui sont des phénomènes communs au Japon. Une étude de cas d'un éboulement réactivé, qui s'est produit dans un secteur résidentiel près de la ville de Kyoto, sera employée comme exemple. Des investigations complètes de champ et de laboratoire ont été conduites pour estimer des paramètres essentiels tels que l'arrangement géologique du secteur, aussi bien que les propriétés des sols, du taux de mouvement d'éboulement, de l'endroit de la surface de glissement et du niveau d'eaux souterraines. En outre, une analyse de stabilité de pente a été exécutée pour estimer un facteur de sûreté de la pente. Basé sur les résultats obtenus, un ensemble de mesures réparatrices a été conçu.

Keywords: Landslides, clay, shear tests, stability, mechanical properties.

INTRODUCTION

Reactivated landslides are a common phenomena in Japan. Most of these landslides are residual slides which occur on gentle slopes and move slowly mainly due to an increase in groundwater level (Sassa 1985). A series of studies conducted in recent years revealed the strong influence of clay content and clay mineralogy on the mechanism of reactivated landslides. Shuzui (2001) and Gratchev, Sassa & Fukuoka (2005a) pointed out that the sliding surface (or slip plane) was usually formed in clayey soils and existed in the residual state due to previous movements of the landslide. However, further research (Gibo et. al 2002; Gratchev, Sassa & Fukuoka 2005b) suggested that some strength recovery could occur during a stable period of time before reactivation, and the shear strength of the sliding surface might be higher than the residual value. Therefore, the shear strength of the sliding surface should be studied carefully, especially for a hazard assessment.

Figure 1. Location of the studied landslide

The hazard assessment of reactivated landslides must be conducted on the basis of field and laboratory investigations. The field investigation will provide vital information such as the location of the sliding surface and groundwater level. The laboratory examinations of soil collected from the sliding surface of a landslide will determine the soil's geotechnical properties, which will be of great importance for the estimation of the slope's stability. An example of the hazard assessment of a reactivated landslide is described in this paper. The studied landslide was reactivated in 2004 in the town of Otsu in the vicinity of the City of Kyoto, Japan (Figure 1). The removal of material at the toe of the slope and the high level of ground water were put forward as the causes of the reactivation. To design the most appropriate remedial measures, a comprehensive investigation of the area was carried out by the local government in cooperation with the Research Centre on Landslides, Kyoto University, Japan.

FIELD INVESTIGATION

A field investigation, including a series of boreholes, was carried out to study the geology of the landslide site. The borehole cores were collected and examined visually. Then, borehole inclinometers were installed to determine the location of the sliding surface and to monitor the direction and rate of movement of the landslide. The monitoring record revealed that:

1) There were two landslide blocks: Block 1 and Block 2 (Figure 2).
2) The dip angle of the sliding surface was gentle and varied from 10 to 11 degrees.
3) The maximum depth of the sliding surface was estimated to be about 10 m.
4) The sliding velocity of Block 1 was higher (≈0.5 mm/month) than that of Block 2 (≈0.06 mm/month). Based on the these results, it was assumed that the reactivation of Block 1 caused the formation and movement of Block 2.

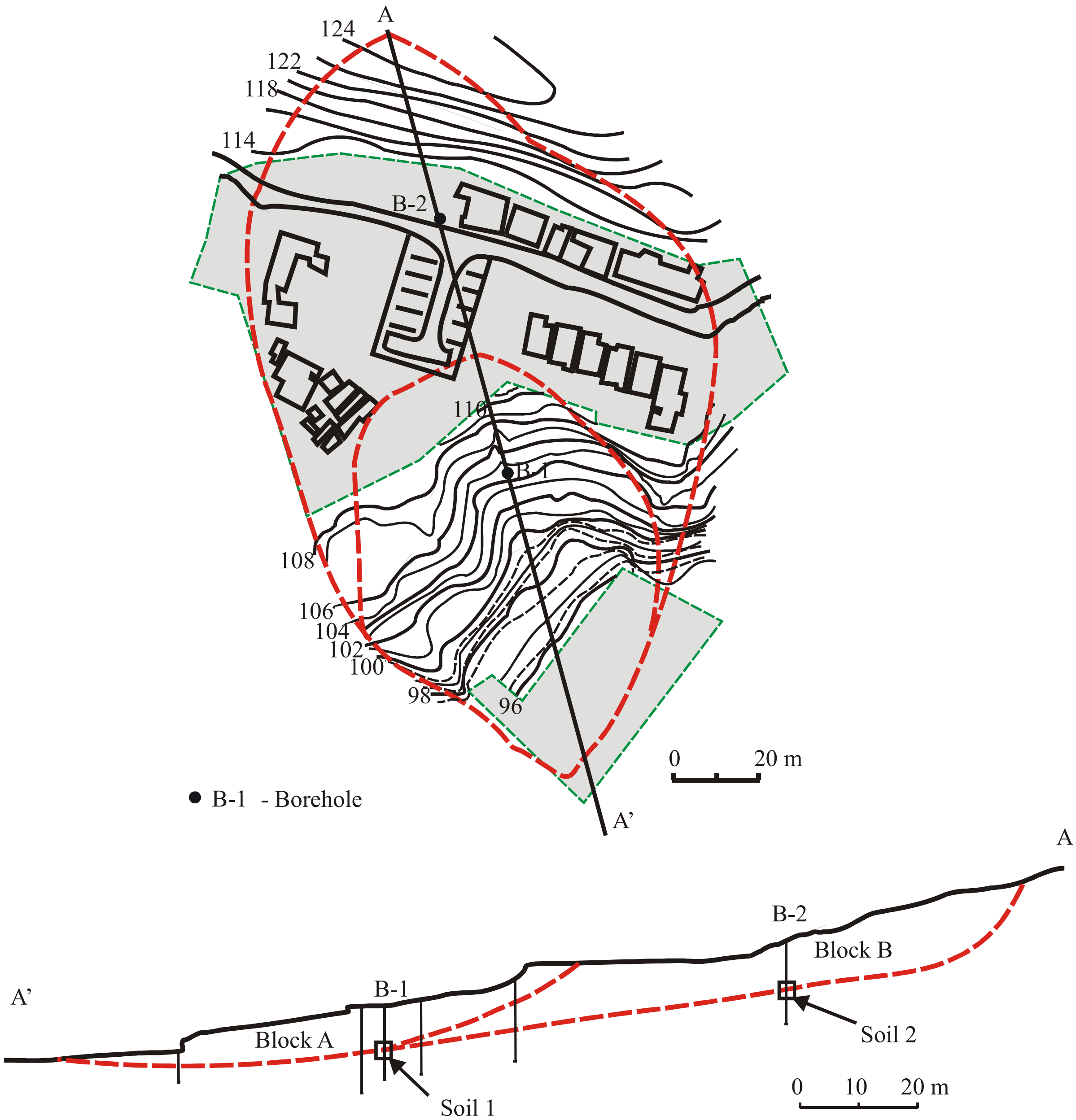

Figure 2. Schematic plan and the cross-section (through A-A') of the studied landslide

To study the geotechnical properties of the soils from the shear zone, two core samples were collected from B-1 (Soil 1) and B-2 (Soil 2). An initial examination revealed the presence of a sliding surface in both samples, and that Soil 1 was softer than Soil 2.

LABORATORY INVESTIGATION

Laboratory examination

To determine the geotechnical properties of the soils, a series of laboratory tests including particle size distribution and X-ray analyses, as well as the Atterberg limit analysis, were carried out. The particle size distribution curves for both soils are shown in Figure 3, while the soil's plasticity index and the mineral composition are summarized in Table 1.

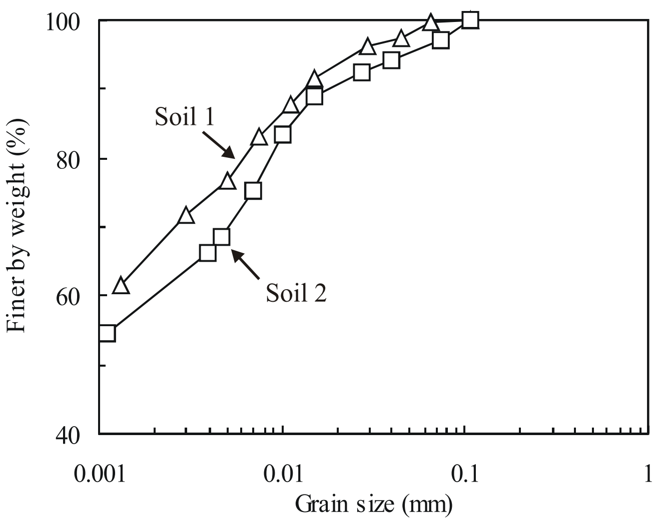

Figure 3. Grain size distribution curves of Soil 1 and Soil 2

Altough the mineral composition was the same (montmorillonite, mica, kaolinite, quartz and plagioclase), in Soil 1, the clay fraction (67%) was greater than that of Soil 2 (59%), which might account for the higher plasticity of Soil 1 (PI=57.2).

Table 1. The geotechnical properties of Soil 1 and Soil 2

Sample	Soil 1	Soil 2
Liquid Limit, %	96.3	76.1
Plasticity Index, %	57.2	49.5
Clay fraction, %	67	59
Minerals	Montmorillonite, Mica, Kaolinite, Quartz, Plagioclase	

Ring-shear tests

Ring-shear apparatus

The ring-shear apparatus used in this work was a DPRI-4, one of a series of intelligent ring-shear machines developed at the Disaster Prevention Research Institute, Kyoto University. The design of this ring-shear apparatus and the method of sample preparation have been described by Vankov & Sassa (1998). One of the advantages of this apparatus is its big shear box, measuring 210 mm and 290 mm in the inner and outer diameters, respectively. Two personal computers are used for controlling the test and for recording the data respectively. A test can be carried out using either shear torque control, shear speed control or shear displacement control. Pore pressure, applied stress and displacement are measured by the transducers and recorded automatically.

Test procedure

The specimens were prepared from slurry, and set into the shear box. The degree of saturation was determined by measuring the B_D value, which was defined as the ratio between the increments of generated pore pressure (Δu) and normal stress ($\Delta\sigma$) ($B_D=\Delta u/\Delta\sigma$) (Sassa 1988). The ratio for each test was ensured to be more than 0.95, a value that approximates to a fully saturation condition. The specimens were normally consolidated under confining stresses of about 50, 100 and 200 kPa. To prevent generation of pore water pressure during the testing, the lowest speed of 0.1 mm/min was used.

Results

Figure 4 shows the results obtained for Soil 1. As can be seen, the value of the residual friction angle was relatively low at approximated 11.0°. The peak shear strength of this soil was about 19.6° and the cohesion was about 4.1 kPa. The residual friction angle and the peak shear strength of Soil 2 (Figure 5) were found to be higher, at 17.4° and 28.1°, respectively. The cohesion of Soil 2 was 6.6 kPa. For all the specimens formed from Soil 1 and Soil 2, the sliding surface was observed to be of a similar nature: polished with smooth striations in the direction of shear.

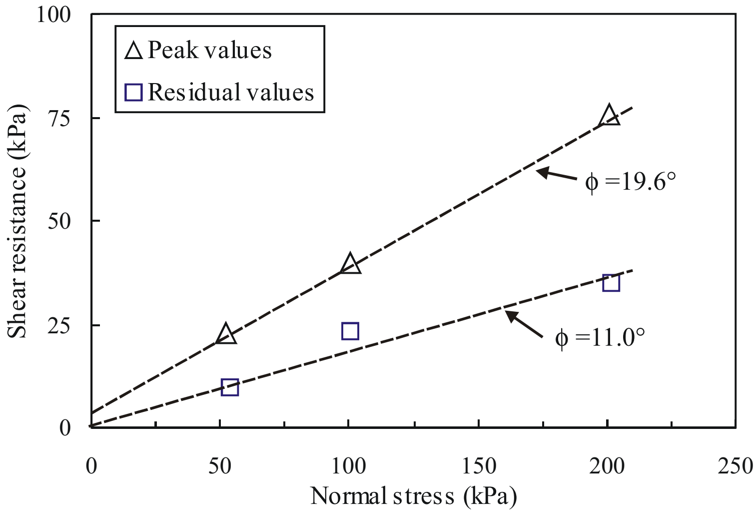

Figure 4. Results from drained speed-controlled ring-shear tests on Soil 1

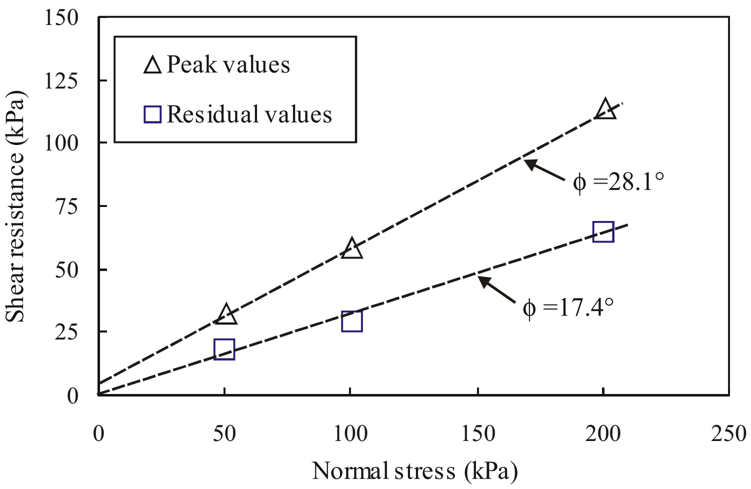

Figure 5. Results from drained speed-controlled ring-shear tests on Soil 2

SLOPE STABILITY ANALYSIS

A number of slope stability calculations were carried out for the studied landslide based on the limit equilibrium theory. The geometry of the slope, the location of the sliding surface, and the groundwater level were obtained from the boreholes. The calculations were performed according to the Morgenstern & Price (1965) method based on the hypothesis that the function describing the relation between horizontal and vertical forces is constant. First, the safety factor of 0.94 was calculated for Block 1 using the assumption that the residual friction angle was constant throughout the entire sliding surface and equal to 11.0° (the residual shear strength value of Soil 1 obtained from the ring-shear tests). Then, the safety factor was computed for the whole landslide, including Block 1 and Block 2, which was equal to 1.13. This value seemed to be overly high, suggesting a degree of slope safety that could be misleading. The apparent overestimation of the safety factor and the analysis of the landslide mechanism prompted the authors to use the progressive failure method (PFM) to estimate the slope stability. The principles of PFM, based on the limit equilibrium concept, were described by Law & Lumb (1976) and Tiande, Chongwu & Shengzhi (1999); thus, only a brief introduction to the method is given. The landslide body was divided into slices and the moments of forces acting on each slice were taken. The local failure was determined by comparing the shear stress due to the effect of gravity and the pore pressure which acted upon the slice, to the maximum available shear strength. If the former was smaller than the latter, the slice was assumed to be 'safe', if the former was higher, the slice was assumed to 'fail' and a local failure would begin to propagate. The inter-slice forces were calculated and the local failure propagation was estimated for the whole landslide. The final safety factor was defined as the ratio of the overall available strength to the actual shear stress acting on the slope.

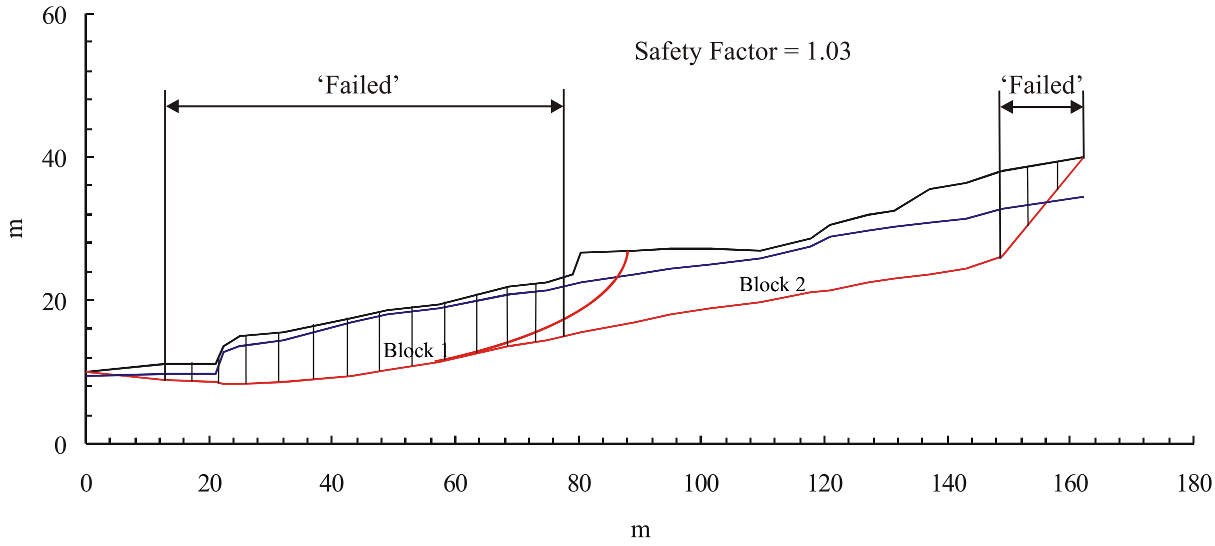

Figure 6. Results from slope stability analysis based on the progressive failure method

The results are shown in Figure 6. As can be seen, Block 1 is unstable and the local failure occurred along the sliding surface (marked as "Failed" in Figure 6), which meant that the soil resistance of Block 1 had reached its final residual value. The failure of Block 1 probably triggered the failure of Block 2. However, the soil resistance of Block 2 did not arrive at the residual state (except the top part, which is marked as 'failed' in Figure 6). The final safety factor was computed to be 1.03, a value that seemed to be more reasonable than that obtained from the conventional Morgenstern & Price (1965) method. In addition, the analysis facilitated a visual understanding of the mechanism of the studied landslide and explained why the sliding velocity of Block 1 was higher than that of Block 2.

SUMMARY

Field and laboratory examinations were carried out to perform a hazard assessment of a reactivated landslide in a residential area near the City of Kyoto, Japan. Two sliding blocks (Block 1 and Block 2) were monitored by means of borehole inclinometers and the highest rate of movement was estimated to be about 0.5 mm/month for Block 1. The analysis of results from field observations and laboratory measurements of samples from the sliding zone of Block 1 showed that the sliding surface was formed in a layer of soft plastic clay with a relatively low value of residual friction angle of 11.0°. The residual friction angle of soil from the sliding surface of Block 2 was determined to be 17.4°. The results of an analysis using the progressive failure method based on the limit equilibrium concept suggested that the reactivation of Block 1, which was initially caused by cutting away material at the toe of the slope and by high groundwater level, was the trigger for the movement of Block 2. Based on the results of this investigation, remedial measures were designed.

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