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SEISMIC STABILITY OF COHESIVE SOIL SLOPES

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Abstract

The paper presents a re-evaluation of the Fourth Avenue landslide in Anchorage during the 1964 Alaskan earthquake. The re-evaluation reveals that the slide was caused by a large undrained strength loss and development of an undrained residual strength condition in the Bootlegger Cove clay. This mechanism differs from the liquefaction mechanism proposed by Shannon and Wilson (1964), Seed and Wilson (1967), and Seed (1968). A constant volume ring shear apparatus was developed to measure the undrained peak and residual shear strength of cohesive soils. The results of constant volume ring shear tests on Bootlegger Cove clay are presented and compared to back-calculated shear strength values. The comparison shows that slide blocks, which moved less than one-half foot mobilized the undrained peak shear strength. Slide blocks that moved more than one-half foot mobilized the undrained residual strength. The results of this study suggest that the peak undrained shear strength should be used to evaluate the seismic stability of cohesive soil slopes. If sliding will be triggered due to earthquake shaking, an undrained residual strength will be mobilized and should be used to estimate the permanent lateral displacements.

Keywords

soft clays, seismic slope stability, undrained shear strength, ring shear tests

Introduction

A significant number of landslides involving cohesive soils have occurred during earthquakes. Some of the earthquakes in which these slides have occurred are the New Madrid earthquake of 1811, the Chilean earthquake of 1960, the Alaskan earthquake of 1964, and most recently the Saguenay earthquake of 1988. The most notable landslides are the Fourth Avenue, L-Street, Government Hill, First Avenue, Romig Hill, and Turnagain Heights in Anchorage that were caused by the 1964 Alaska earthquake (Seed, 1968) and the Sainte-Thècle and Saint-Adelphe landslides that took place during the

1988 Saguenay earthquake (Lefebvre et al., 1992). These landslides have lead to an interest in the undrained peak and residual shear strength of cohesive soils. It is anticipated that these slides were caused by an undrained failure and post-peak strength loss in the cohesive soil involved in the slides.

Following the 1964 Alaska earthquake, many of the landslides that occurred in Anchorage were intensively studied. In particular, the Fourth Avenue landslide, prompted considerable investigation because of the large destruction resulting from the slide. The main failure mechanism proposed in the resultant literature was liquefaction of sand lenses (Shannon and Wilson, 1964; Seed and Wilson, 1967; Seed, 1967,1968; Wilson, 1967). A re-evaluation of the Fourth Avenue slide by Woodward Clyde Consultants (1982), revealed that the slide was not caused by liquefaction but by a large undrained strength loss in the Bootlegger Cove clay. However, an apparatus that could measure the entire undrained post-peak strength loss and the collapse of the soil structure was not available during the 1982 study. Stark and Contreras (1994) developed a constant volume torsional ring shear apparatus that allows the measurement of the magnitude and rate of undrained post-peak strength loss, and the undrained residual strength. The amount of lateral movement in the Fourth Avenue slide was either several feet or less than a few inches. It is postulated that in areas where the lateral movements were greater than one-half foot, the shear strength of the Bootlegger Cove clay was reduced to the undrained residual value. This paper presents a re-evaluation of the Fourth Avenue landslide based on tests results from the new constant volume ring shear apparatus.

Fourth Avenue Landslide

The Fourth Avenue slide occurred during the great 1964 Alaska earthquake. This earthquake had an epicenter approximately 130 km east of Anchorage. The earthquake magnitude was estimated as 8.5 (Woodward-Clyde Consultants, 1982) and the surface ground motion levels at Anchorage were estimated to be 0.15 to 0.2 g (Newmark, 1965; Housner and Jennings, 1964; and Shannon and Wilson, 1964). However, no accelerograms of the earthquake shaking were obtained. The duration of the ground motions in Anchorage was reported to range from 4 to 7 minutes, with strong, potentially damaging shaking lasting approximately 2 to 3 minutes (Steinbrugge, 1970; Housner and Jennings, 1964).

The slide mechanism was primarily horizontal translatory which is characterized by lateral spreading and graben development. Two grabens were created east of C street. The survey of ground movements suggested that by the end of the earthquake, another graben had begun to form between Fourth and Fifth Avenues at D and E streets (Wilson, 1967). The greatest damage to structures developed within and adjacent to the grabens, and along the pressure ridge that developed at the toe of the slide between First and Second Avenues. In contrast, very little damage was suffered by buildings and streets that were located on the sliding mass.

Based on analyses of the slide conducted shortly after the earthquake, the zone of shearing was found to be at a depth of approximately 60 to 70 ft below the ground surface. This depth is at or near the interface between a fine sandy layer and the underlying soft Bootlegger Cove clay. At that time it was not clear whether the slide occurred through the fine sandy layer because of liquefaction or through the soft Bootlegger Cove clay layer because of undrained failure (Shannon and Wilson, 1964). This uncertainty regarding the failure mechanism was in part due to the relatively limited data, particularly on the sands, available at that time.

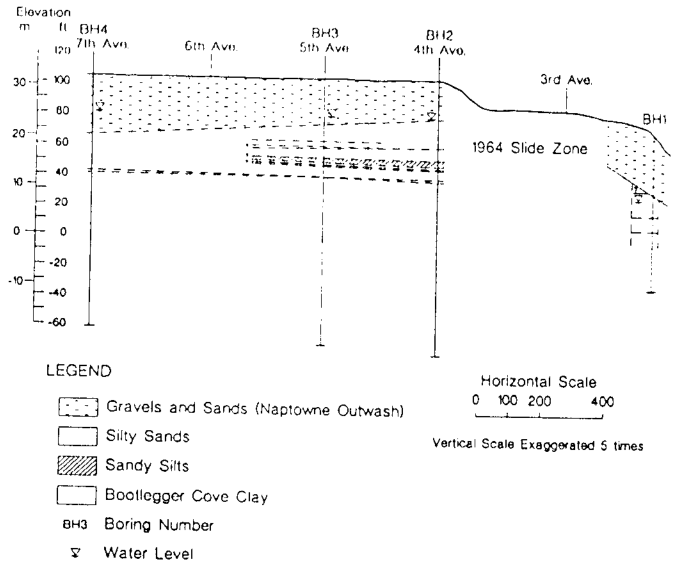

Fig. 1. Subsurface Conditions Along D Street (After Idriss, 1985)

The subsurface conditions in the slide area are illustrated in Fig. 1, which is a cross section along D Street. From the subsurface investigations conducted after the earthquake (Shannon and Wilson, 1964) and the 1982 Woodward Clyde Consultants investigation, the following subsurface conditions were obtained:

1.) The upper most zone consists of very dense gravels and sands (Naptowne Outwash). The thickness of the Naptowne Outwash generally ranges from 25 ft to 40 ft. The SPT blow counts in the outwash are typically greater then 60 blows/ft.

2.) An interbedded zone underlies the Naptowne Outwash and consists of alternating layers of clay, silty sands, and sandy silts to a maximum depth of 62 ft below the ground surface (elevation 40 ft). This interbedded zone corresponds to the upper portion of the Bootlegger Cove Formation. The silty sands and sandy silts in this zone are not continuous and are dense to very dense. The SPT N_1 blow counts range from 21 to 70 blows/ft. The clays in this zone are stiff to very stiff probably due to desiccation (Updike, 1988). Overconsolidation ratios (OCR) of about 3 to 4 were measured for the clays in the interbedded zone (Woodward Clyde Consultants, 1982).

3.) Below the interbedded zone, a soft clay is found to the maximum depths explored (about 150 ft below the ground surface). This soft clay belongs to the intermediate zone of the Bootlegger Cove Formation. This soft clay displays an OCR of 1.2 to 1.5. The intermediate zone of the Bootlegger Cove Formation has been identified with zones of sensitive clays with values of sensitivity ranging from 3 to 11.

As noted earlier, the evaluation of the Fourth Avenue slide after the earthquake indicated that the zone of shearing was located near the lower boundary of the interbedded zone. The liquefaction potential of the silty sand and sandy silt layers in the interbedded zone was evaluated (Woodward-Clyde Consultants, 1982) using the empirical procedure based on field observations (Seed and Idriss, 1981). The modified blow counts for these layers (Fig. 2) correspond to a minimum factor of safety against liquefaction of approximately 1.6. As a result, it was concluded that the sandy silt or silty sand layers in the interbedded zone did not liquefy, and thus was not the cause of the slide. Therefore, the cause of the slide was thought to be an undrained failure of the soft Bootlegger Cove clay.

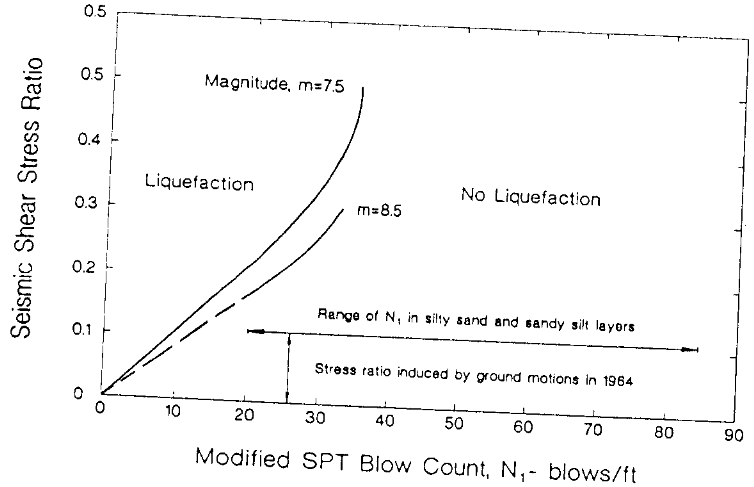

Fig. 2 Liquefaction Potential in the Sandy Strata in the Interbedded Zone During the 1964 Alaska Earthquake (After Idriss, 1985)

Shear Strength of the Soft Bootlegger Cove Clay

A summary of undrained peak shear strength data available for the Bootlegger Cove clay from the investigations conducted after the earthquake (Shannon and Wilson, 1964) and the 1982 Woodward-Clyde Consultants study is shown in Fig. 3. Figure 3 includes relationships for triaxial compression (TC), triaxial extension (TE), direct simple shear (DSS), and laboratory vane shear (LV) modes of shear. In general, the relative positions of the TC, DSS, TE, and LV data points agree well with the average lines proposed by Jamiolkowski et al. (1985), Chandler (1987), and Mesri (1989).

As indicated earlier the slide mechanism was primarily horizontal translatory. As a result, the direct simple shear apparatus allows the closest laboratory simulation of the stresses and deformations imposed by the earthquake shaking and translational sliding on soil elements in the field. The undrained shear strength ratio from the DSS test, $S_{u(DSS)}/\sigma'_{vc}$ in Fig. 3 varies from 0.18 to 0.24 with an average of 0.21. Changes to this strength can result due to cyclic shear strains induced by the earthquake loading and/or due to large displacements during slide movements (Idriss, 1985).

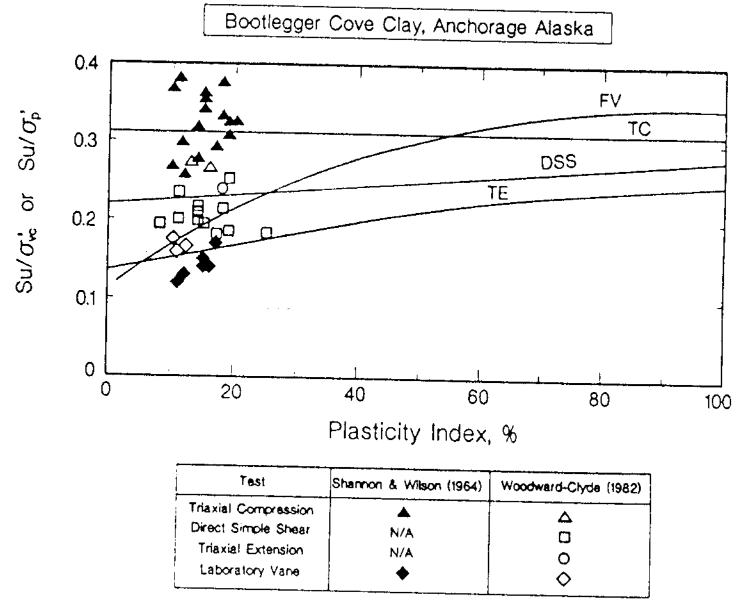

Fig. 3. Summary of Undrained Peak Strength Ratios of Bootlegger Cove Clay

The potential for strength changes due to cyclic loading on Bootlegger Cove clay was investigated by Woodward-Clyde Consultants (1982) using cyclic direct simple shear tests followed by post-cyclic static tests. It was found that reduction of the strength due to cyclic loading is rather small and generally does not exceed more than 25 to 30 percent of the original static strength. Lefebvre and LeBoeuf (1989), concluded that even extrasensitive clays are not believed to suffer significant loss of shear strength during the cyclic loading representing a moderate earthquake.

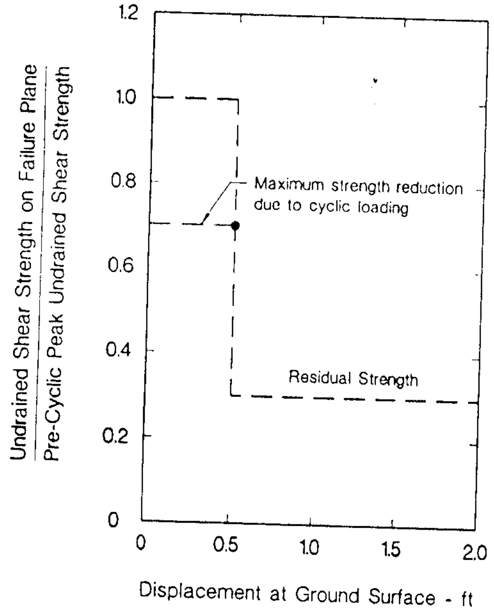

Fig. 4. Variation of Undrained Shear Strength on Potential Failure Plane as a Function of Displacement at the Ground Surface (After Idriss, 1985)

Reduction in shear strength can also occur in cohesive soils due to large deformations that result in clay particles rearranging into a preferred orientation, such that only the frictional resistance of parallel soil particles is available. This minimum strength is termed the residual strength. If sufficient deformation takes place during an earthquake, a residual strength may be achieved under undrained conditions. However, at the time of the 1982 Woodward-Clyde Consultants investigation a laboratory apparatus and test procedure for measuring the undrained residual strength of clays was not available. As a result, during the 1982 investigation the results of miniature laboratory vane shear tests, cone penetration tests, and engineering judgment were used to estimate the undrained residual strength.

Examination of the laboratory vane shear tests and the cyclic DSS tests lead to the relationship between undrained shear strength and ground surface displacement shown in Fig. 4.

It can be seen from Fig. 4 that for displacements less than one-half foot the undrained shear strength is reduced a maximum of 30 percent of the static peak strength due to the effects of cyclic loading. Idriss (1985) concluded that in the Fourth Avenue landslide the slide blocks that moved less than one-half foot underwent a shear strength reduction up to 30 percent. If sufficient deformations take place during an earthquake (i.e., more than one-half foot) the shear strength is further reduced to the residual value. Idriss (1985) concluded that this strength reduction corresponds to 70 percent of the peak value.

Constant Volume Ring Shear Apparatus

To investigate the peak and residual undrained shear strengths, the original Bromhead (1979) ring shear apparatus was modified by Stark and Contreras (1994) to conduct constant volume tests. The modifications include a mechanism for decreasing the normal stress during shear, such that the volume change is negligible during shear, and fabricating a new specimen container to allow undisturbed specimens to be trimmed directly into the container.

The ring shear specimen is annular with an inside diameter of 70 mm and an outside diameter of 100 mm. Drainage is provided from the bottom by a 6.5 mm thick annular bronze porous stone that is secured by four screws. The specimen is confined radially by the specimen container, which is 10 mm deep. Drainage is provided from the top by an annular bronze porous stone, which is attached to the loading platen. The porous stone is secured to the loading platen using four screws. It has a thickness of 5.7 mm and aids the transfer of shear stress to the top boundary of the soil specimen. The porous stone is serrated to develop a strong interlock with top boundary of the soil specimen. Two intersecting groups of serration lines are grooved in the porous stone. Spacing between lines in each group is 2 mm. The indentation angle is 45 degrees with a depth of 0.7 mm. These serrations prevent slippage at the loading platen/soil interface during shear. The annular bronze porous stone also has an inner and outer groove machined at 1 mm from the bottom edge of the ring to accommodate two O-rings. These O-rings close the gap between the top annular ring and the walls of the specimen container to minimize the amount of soil extrusion that takes place during shear.

The unload mechanism developed to maintain a constant specimen volume during shear, and thus conduct an undrained test, is illustrated in detail in Fig. 5. It consists of a steel bracket that is connected to the end of the loading arm. The loading arm applies the dead weight on the hanger to the top of the specimen. Attached to the top of the bracket, is a sensitive load cell. Connected to the top of the load cell, is a threaded steel rod 9 mm in diameter. This threaded steel rod goes through a 14 mm diameter hole in an aluminum beam that is connected to the frame of the original apparatus by two screws. A nut is threaded to the top of the steel rod. An electronic readout device is connected to the load cell, such that the load carried by the load cell can be determined. The nut is threaded and adjusted during shear to reduce the amount of normal load transferred to the loading platen. The nut is adjusted such that a negligible amount of vertical displacement of the soil specimen occurs during shear.

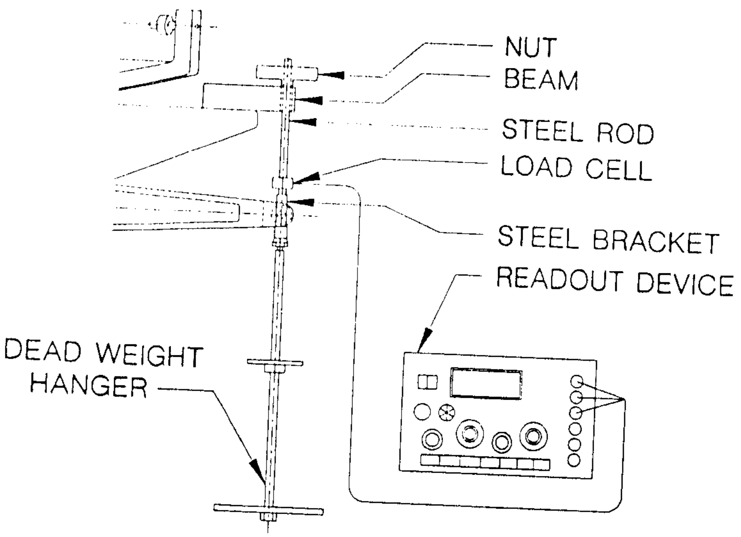

Fig. 5. Unload Mechanism for Ring Shear Apparatus

Constant Volume Ring Shear Test Procedure

In the constant volume ring shear apparatus, the specimen is sheared by rotating the specimen container past the stationary loading platen at a constant rate of 0.018 mm/minute. The procedure described by Gibson and Henkel (1954) is used to determine the shear displacement rate. This displacement rate is selected such that the pore-water pressures in the specimen are zero throughout the test. The normal stress is reduced during shear to maintain a constant specimen height or volume. It is assumed that the decrease in applied vertical stress during shear is equivalent to the increase in shear-induced pore-water pressure that would occur in an undrained test with constant vertical stress. The validity of the pore-water pressure assumption for constant volume tests was verified by Dyvik et al. (1987) for the direct simple shear apparatus and by Berre (1981) for the triaxial apparatus.

During a constant volume test the height of the specimen is monitored using the dial gauge that measures vertical displacement of the loading platen. The nut connected to the steel rod in the unload mechanism (Fig. 5) is used in combination with the vertical dial gauge to adjust the normal stress, such that the specimen thickness remains constant during shearing. Adjustments are made by rotating the nut an amount required to maintain zero vertical displacement of the soil specimen. When the nut is rotated, a portion of the dead weight is transmitted to the rod and the load cell indicates the magnitude of this load. Details of the constant volume ring shear apparatus can be found in Stark and Contreras (1994).

Constant Volume Ring Shear Test Results

Undisturbed samples of Bootlegger Cove clay were obtained using a 100 mm diameter sampler at the Fourth Avenue Slide. This Bootlegger Cove clay has the following index properties: (1) natural water content of 34 percent; (2) liquid limit of 36 percent; (3) plasticity index of 15 percent; and (4) clay-size fraction of 49 percent.

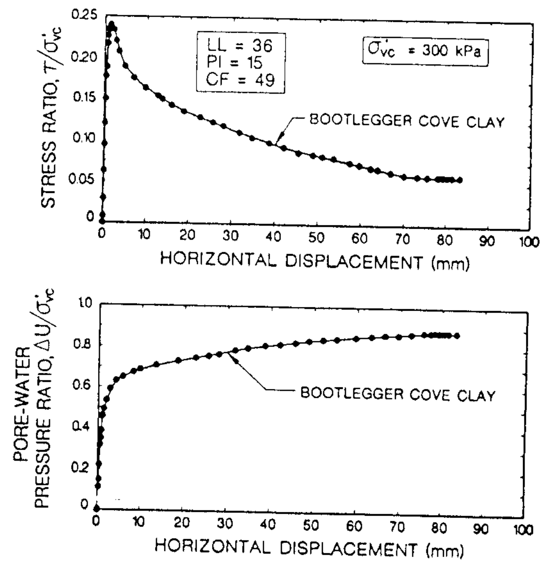

Fig. 6. Constant Volume Ring Shear Test on Bootlegger Cove Clay

Figure 6 shows a typical set of results of a constant volume test on a specimen of Bootlegger Cove clay. Figure 6 shows the stress ratio-horizontal displacement relationship. The stress ratio is defined as the shear stress, τ, divided by the consolidation stress, σ'_{vc}. It can be seen that the undrained peak stress ratio is reached after approximately 1.5 mm of horizontal displacement and the undrained residual stress ratio is reached at a displacement of about 80 mm. The undrained peak stress ratio, $S_{u(RS)}/\sigma'_{vc}$, is approximately 0.23. These $S_{u(RS)}/\sigma'_{vc}$ ratio is within the range of values of $S_{u(DSS)}/\sigma'_{vc}$ obtained by Woodward Clyde Consultants (1982), and represents a typical ratio for soft clays of this plasticity (Fig. 3).

After the undrained peak stress ratio is mobilized in the ring shear apparatus, shear displacement along the failure surface continues and the stress ratio starts to decrease with increasing displacement. Shear displacement along the failure surface causes orientation of soil particles parallel to the direction of shear. The movement and arrangement of particles along the failure surface is accompanied by a change in pore-water pressure. This continues until the undrained residual strength is mobilized. It can be seen from Fig. 6 that the undrained residual strength ratio is mobilized at a shear displacement of approximately 80 mm. The undrained residual stress ratio is measured to be approximately 0.06. It can be seen that this undrained residual stress ratio is significantly lower than the peak undrained stress ratio. It is postulated that this large post-peak strength loss, approximately 70 percent, led to the large lateral displacements observed during the earthquake. This undrained residual shear stress ratio is in agreement with the strength ratio reported by Idriss (1985) for blocks that move more than one-half foot.

Figure 6 also presents the pore-water pressure ratio-horizontal displacement relationship measured for Bootlegger Cove clay in the constant volume ring shear apparatus. The pore-water pressure is assumed to be equal to the decrease in normal stress. It can be seen that the pore-water pressure ratio increases and then becomes essentially constant at the residual condition.

At the undrained residual strength condition, it is possible to compute the drained residual friction angle, ϕ'_r, of a clay from the results of a constant volume ring shear test using the following equation:

$$\tan \phi'_r = \frac{\tau_r}{\sigma'_r} \qquad (1)$$

where τ_r is the residual shear stress and σ'_r is the effective stress at the residual condition. The value of ϕ'_r computed using Equation (1) is approximately 25 degrees. The measured ϕ'_r from a series of drained ring shear tests on the same Bootlegger Cove clay, following the procedure described by Stark and Eid (1994), is also 25 degrees. Therefore, this agreement reinforces the validity of the results and the constant volume test procedure.

Conclusions

Re-evaluation of the Fourth Avenue landslide that occurred in Anchorage, Alaska during the 1964 earthquake reveals that the slide was caused by a large undrained strength loss and development of a residual strength condition in the Bootlegger Cove clay. This mechanism differs from the liquefaction mechanism postulated immediately after the earthquake. A constant volume ring shear apparatus was developed to measure the undrained peak and residual shear strength of cohesive soils. The results of constant volume ring shear tests on Bootlegger Cove clay are presented and compared to the undrained shear strengths back-calculated for the slide blocks. The comparison shows that slide blocks, which moved less than one-half foot mobilized the undrained peak shear strength. Slide blocks that moved more than one-half foot mobilized the undrained residual strength. The results of this study suggest

that the peak undrained shear strength should be used to evaluate the seismic stability of cohesive soil slopes. If sliding will be triggered due to earthquake shaking, an undrained residual strength will be mobilized and should be used to estimate the permanent lateral displacements.

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A CORRELATION BETWEEN LANDSLIDES AND RAINFALL DATA ALONG A-LI SAN HIGHWAY OF TAIWAN

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ABSTRACT

A study was carried out to see the possibility of building a warning system for landslides along A-Li San Highway by the real-time rainfall data. A survey was done on the relationship between the rainfall data and landslide record. The stability of the rock slopes was evaluated by the SMR method . If the limiting thresholds of rainfall are expressed in the way of intensity-duration curves suggested by Caine, the rock slopes with higher SMR values would have a higher intensity-duration limiting threshold. It also suggests that a warning system with probability of occurrence is more probable.

KEYWORDS

Slope failure, rainfall data, warning system, SMR values, limiting threshold, occurrence rate.

INTRODUCTION

It is a common experience that landslides,especially natural slope failures, occur during the period of heavy rainfall. Indeed, rainfall is one of the major factors which can trigger the occur ence of landslides. This phenomenon may be explained by the following reasons:

1.Rainfall increases the pore pressure of the earth material and then reduce the effective normal stress acting on the potential sliding surfaces. Finally, it will reduce the resistance