Evaluation of the rate of movement of a reactivated landslide

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ABSTRACT
Washington Park slide in Portland, Oregon, was first encountered in 1894 during the construction of the two water supply reservoirs. Thirty-three borings and 22 open-shafts revealed information about the subsoil conditions and the presence of a well-defined thin seam of blue clay along the surface of the bedrock. In this paper, the mechanism of the reactivated landslide is discussed, and the relation between the rainfall and observed movements is investigated.

RÉSUMÉ
Le premier cas de glissement à être enregistré dans le parc de Washington à Portland, dans l'Oregon, date de 1894 pendant la construction des deux réservoirs d'eau. 33 carottes et 22 puits ouverts ont ainsi pu donner des informations quant à l'état du sous-sol, révélant notamment la présence d'une fine couche d'argile bleue clairement définie le long de la surface du soubassement/du substrat rocheux. Dans cette étude, nous allons donc examiner les mécanismes de réactivation de glissements de terrain ainsi que la relation existant entre les précipitations enregistrées et les mouvements du sol.

1 INTRODUCTION

The landslide is located in the Central West Hills, west of downtown Portland in Oregon. The city of Portland constructed two water supply reservoirs in the 1890's by excavating in a small narrow valley. This paper presents the background information about this landslide, the stratigraphy and material properties, observed movement through the years and correlation of stability with rainfall. The 3-D view of the terrain is shown in Figure 1.

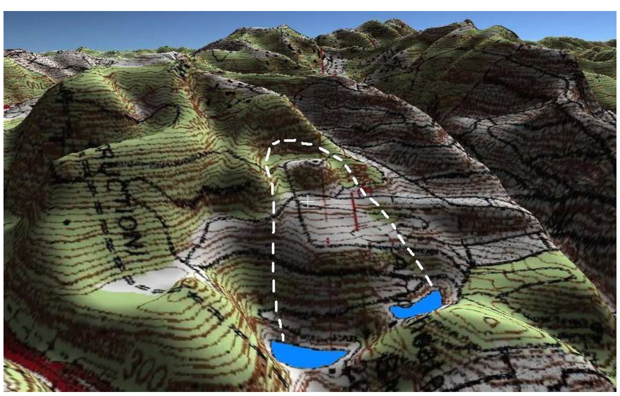

Figure 1. NASA World Wind 3-D view of the terrain in Washington Park Reservoir slide area looking northwest. White dash lines indicate the approximate boundary of the moving mass, the two reservoirs are shown toward the bottom of the figure with blue colour.

2 BACKGROUND INFORMATION

This is an extremely slow (< 16 mm/year) active landslide, where movements are concentrated within a narrow shear zone, above which the ground moves almost like a rigid body and movements are often controlled by rainfall-induced pore pressure fluctuations. These types of slides have been studied previously by various researchers such as Picarelli et al. 2004, Bonnard and Glastonbury 2005, Eshraghian et al. 2008, and Calvello et al. 2008. It is a good and historic example of a landslide moving at a long-term constant velocity.

On the West Hills of Portland, minor local instability was observed in the location of this landslide during the construction of a residential development upslope during 1891-1892. The reservoir construction for the City of Portland was started in October 1893 and completed in September 1894. In August 1894 engineers found a crack in the west wall of Reservoir 4 (the reservoir on the left in Figure 1) with movements of about 13 mm per day for several days. More indications of instability at the toe of the slope and bulging at the base of the reservoir in the concrete lining of Reservoir 3 were also observed. Continued movements were observed during the winter of 1894-95 at both reservoirs causing them to be out of service for about 10 years.

It should be noted here that there was a discussion in 1890's about whether the slide started moving due to the toe excavation during reservoir construction or due to heavy rainfall. Some people argued that the large main slide was in a state of rest until 1894, up to the time when the cracks first appeared in the west slope of Reservoir No. 4 in August. Clarke (1904) gives ample evidence that the slide had been moving before the reservoir excavations were made. This was explained in detail due to lawsuit investigations at the time. It was concluded that the movements were taking place intermittently before the construction of the reservoirs had started.

In 1895 the head scrap of the landslide was found in a marshy depression 518 m upslope of the reservoirs (see Figure 2). The extent of the sliding mass was becoming apparent. The graben feature at the top of the slope was semicircular, 90 m long and 9-18 m wide. The main body of the slide was about 122 m wide near the head scarp and it was 335 m wide at the base near the toe. The volume of the moving mass has been estimated to be 2.6 million m^3 (Clarke 1904).

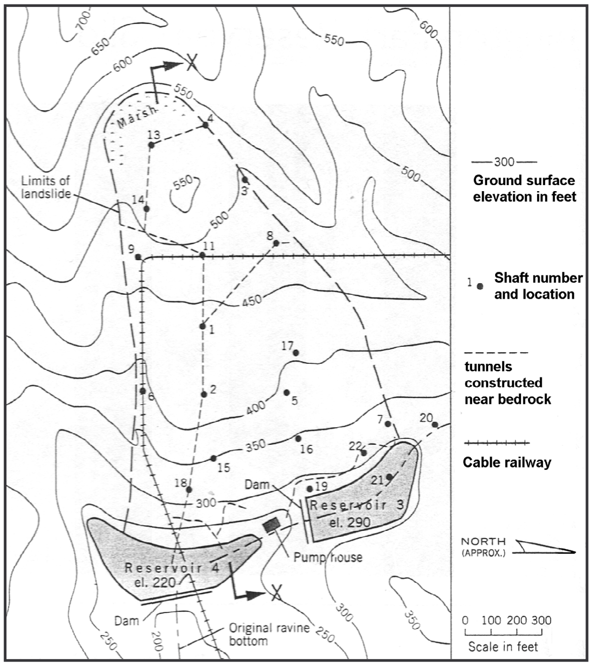

Figure 2. Washington Park reservoir slide (Clarke 1904, Cornforth 2005)

2.1 Geology and Stratigraphy

West Hills form the western boundary of the Portland Basin. They consist of uplifted, layered basalt flows of the Miocene age Columbia River Basalt Group with some Pleistocene to Pliocene age sedimentary deposits (Rice 1998, Myers et al. undated, Burns et al. 2006). Overburden materials are primarily wind-deposited silts called Portland Hills Silt Formation that covers most of the West Hills to depths ranging up to 13 m. These silt deposits are believed to be derived from glacial sediments of the Columbia River drainage system (Rice 1998, Burns et al. 2006) and sediments consist of intermixes of gravels, sands, silts, and clays. A multi-colored plastic clay deposit correlated with the Sandy River Mudstone of Miocene to Pleistocene age underlies the Portland Hills Silt and overlies the basalt. The upper basalt is highly weathered (Myers et al. undated).

During a landslide inventory study in Portland, it was discovered that most of the landslides (48%) surveyed in the study occurred in the West Hills within the Portland Hills Silt or loess (Burns et al. 1998). Earthflows and slumps were the typical type of slides observed in the region, making up 69% of all the landslides (Burns et al. 1998).

Related to Washington Park Reservoir slide, in early 1900’s a subsurface investigation was carried out by 22 deep shafts and 9 wash-drill borings. The shafts were each 1 m square, dug by hand and dewatered by pumps. They were made during July 19, 1897 to January 24, 1899. In 16 shafts the base of the slide was reached and ranged from 15 to 34 m below the ground surface (average 24 m). With the help of shafts a seam of blue clay overlying the bedrock, forming the bed of the ancient slide, was identified (Clarke 1904). Location of these shafts is shown in Figure 2 with black dots and in Figure 3 with black lines. Borings were 10 cm diameter holes, cased with driven wrought-iron pipe.

From the shaft excavations, the general character of the materials forming the mass of the slide has been determined to be largely of broken rock of small size mixed with clay matrix. The detailed notes taken during the logging of shafts and reported by Clarke (1904) are very useful in identifying the materials involved, the depth and character of the slickensided slide surface. For example: Shaft 1: *“Blue sedimentary clay without any admixture of broken rock. Some pieces of clay having smooth upper faces. A plainly defined movement seam was uncovered, having 13 cm of dark blue clay above the line of cleavage, with fine broken rock between the clay and the solid bedrock”*, *“the line of cleavage in the stratum of clay was defined very clearly. The upper and lower portions of the stratum could be easily separated along the line of cleavage, and the faces, which had been in contact presented uniformly a smooth and glazed appearance.”* From the descriptions of the other shafts, the thickness of the blue clay seam above the basalt bedrock and the exact location of the slide plane can be identified. For example, Shaft 2: *“dark blue clay, 30 cm thick, on top of the rock, the line of the slide was defined clearly near the bottom of the clay*” or Shaft 5: *“2.5 cm thick blue clay, the under face of which was smooth, next came 15 cm of mixed clay and rock fragments, below which the bedrock was found smooth and without fissures. The total thickness of the distinctly clay seam was about 8 cm, through which a line of cleavage, indicating the movement plane was defined clearly”* (Clarke 1904).

2.2 Material Properties

The red clays were quite plastic and contained little sand. At some locations, the material was coarse, containing grains and small fragments of the rock incorporated with the clay, but the finer materials were all of the same reddish or yellow color. The blue clay differed from the red. When found in thin seams near bedrock blue clay was tough and plastic but when found in considerable bodies at higher levels it contained quite a significant amount of fine sand. When dry it was hard to pick and a vertical bank would stand without support, when placed in water it soon crumbled into an incohesive mass. Clarke (1904) described it as “blue quicksandy clay”.

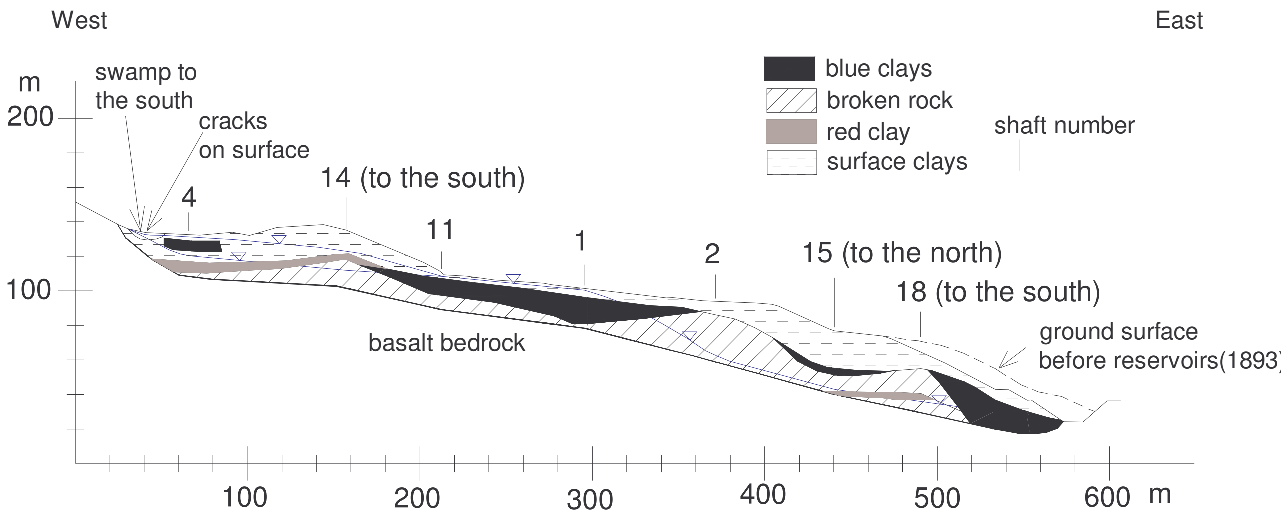

Figure 3. Cross section X-X in Figure 2 (modified from Clarke 1904 and Cornforth 2005)

Cornforth (2005) analyzed the Washington Park Reservoir slope and back-calculated a friction angle of 13.3 degrees. Unfortunately from 1900's no laboratory data is available on the weak blue clay that is causing the movements. But there was another landslide in late 1990's nearby (within 1.6 km distance) discovered during the construction of the Washington Park Station of the Westside Light Rail Project in Portland. The sliding materials were geologically similar (Cornforth 2007). Vessely and Cornforth (1998) took a block sample of the clay within the shear zone at the Washington Park Station slide at a depth of 18.3 m. The landslide was occurring within a layer of decomposed basalt near the contact with less-weathered basalt bedrock. It was moving with an average rate of 0.8 to 20 mm/year. The samples of decomposed basalt consisted of stiff to very stiff, mottled, gray-brown-red silty clay with scattered coarse sand-sized nodules. Samples were highly slickensided. The properties of the material were clay-size fraction of 30%, liquid limit of 85%, plastic limit of 59%, and plasticity index of 26% (Vessely and Cornforth 1998).

3 RATE OF MOVEMENT AND RELATION WITH RAINFALL

The surface movement data are recorded starting from December 1894 (Figure 4). Until December 1899, movement data is obtained from the average movement of 14 survey points established on the surface of the slope, along the central portion of the sliding ground, monitored monthly. Since December 1899 more points are installed for monitoring. The average of the readings at 51 of these points along a central belt 100 ft wide, is used in plotting the movement during 1900-1903. Although it is known that there had been movements before the reservoir excavations were made, it is not possible to know the amount of previous movements. Therefore the cumulative movement data plotted in this study (see Figures 4 and 5) starts from zero movement in December 1894.

To determine the depth of the moving mass, 2.5 cm diameter pipes (connected in 3 m segments) were inserted into the boreholes (Clarke 1904). In most of the borings, the movement was found to be taking place at or near the surface of the bedrock, and at depths varying between 15 and 34 m below the surface of the ground. In some of the borings water would rise in the casing after they were completed, indicating the presence of water pockets at various places in the sliding ground (Clarke 1904).

In this slide as well as in other slides in West Hills in Portland, it is observed that the rate of movement increased during the winter or rainy months of November to May, and decreased during the remaining months of the year when the rainfall was less (Clarke 1904, Cornforth 2005, Myers et al. undated, Burns et al. 2006). In spring 1897 it was understood that the water in underground springs fed by percolating water from the surface was a major factor. A comprehensive drainage system was planned (Clarke 1904). The effectiveness of reducing water pressures is shown (in Figure 4) by the decrease in the rate of movement between December 1897 and November 1898, during which period the underground reservoir connected with Shaft No.1 was drained by pumping.

When landslide movements are monitored the uniformity of the movement of the sliding mass with depth becomes important because the movements are monitored at the ground surface and the basal shear plane may or may not be experiencing the same amount of movements. In the landslide analyzed in this study, the movement at the surface and at bedrock level was found to be uniform. For example, in Shaft No.6, between October 25, 1897 and November 22, 1899 the total surface movement was 14.9 cm, while the movement at the bottom of the shaft was 14.3 cm during the same period (Clarke 1904).

It can be seen from Figure 4 that with each recurring dry season there was a corresponding decrease in the movements and that with the beginning of winter rains the movement increased. It is also seen that a time lag of about one month elapsed between a change in the

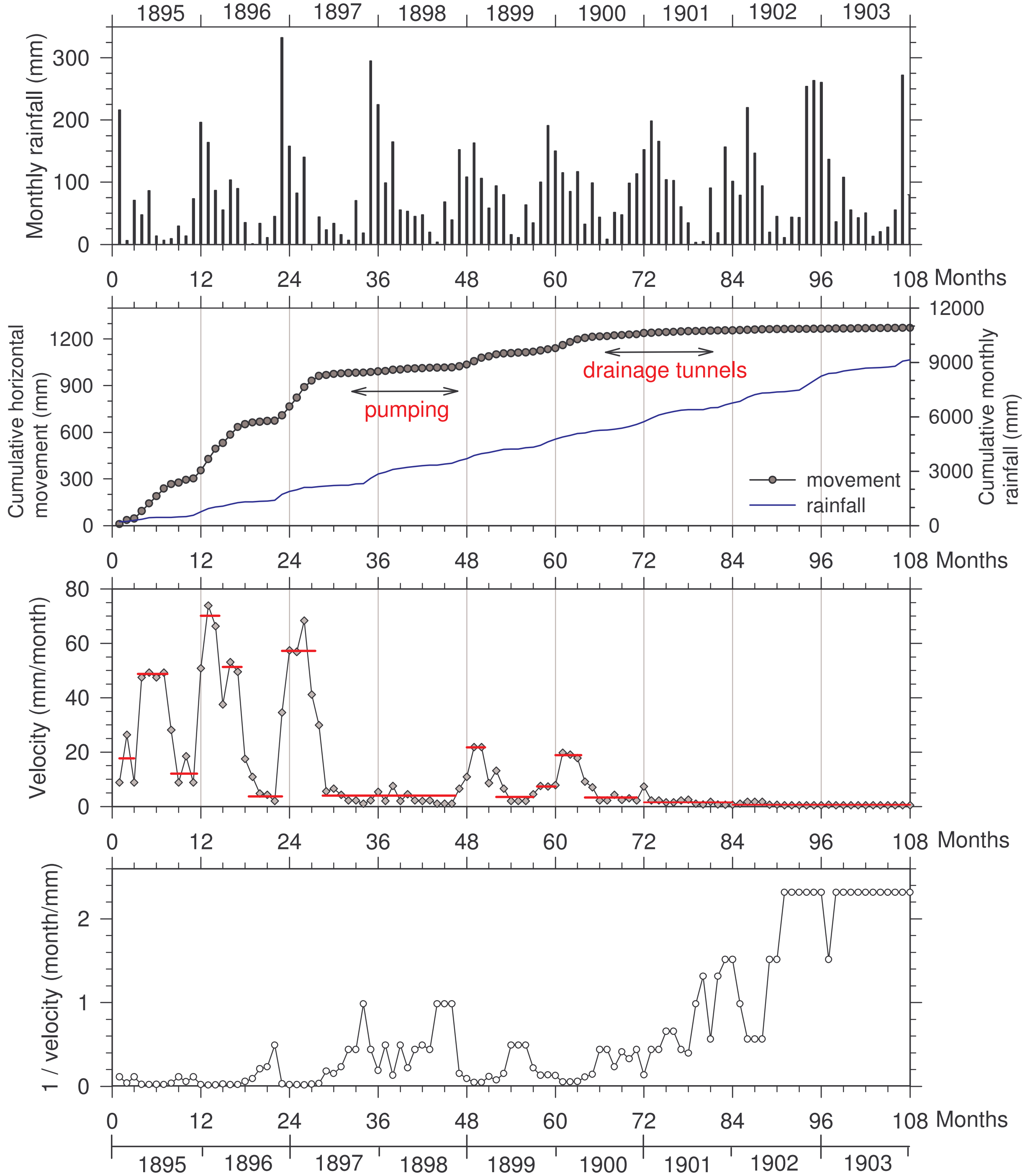

Figure 4. Measured rainfall, movements, and velocities for January 1895 – December 1903 period. Rainfall records are taken at 2.4 km distance from the slide location. Pumping from Shafts No.1, 11 and 18 was carried out during August 1897 – November 1898. Construction of drainage tunnels in progress from July 1900 to September 1901 (data from Clarke 1904).

volume of rainfall and the corresponding change in the rate of movement. When pumping from Shaft No.1 was in progress the movement didn't increase as rapidly during the winter months as it had during the former winter seasons. Decreased movement during 1898 was due in part to the pumping of water from Shaft No. 18 (July-

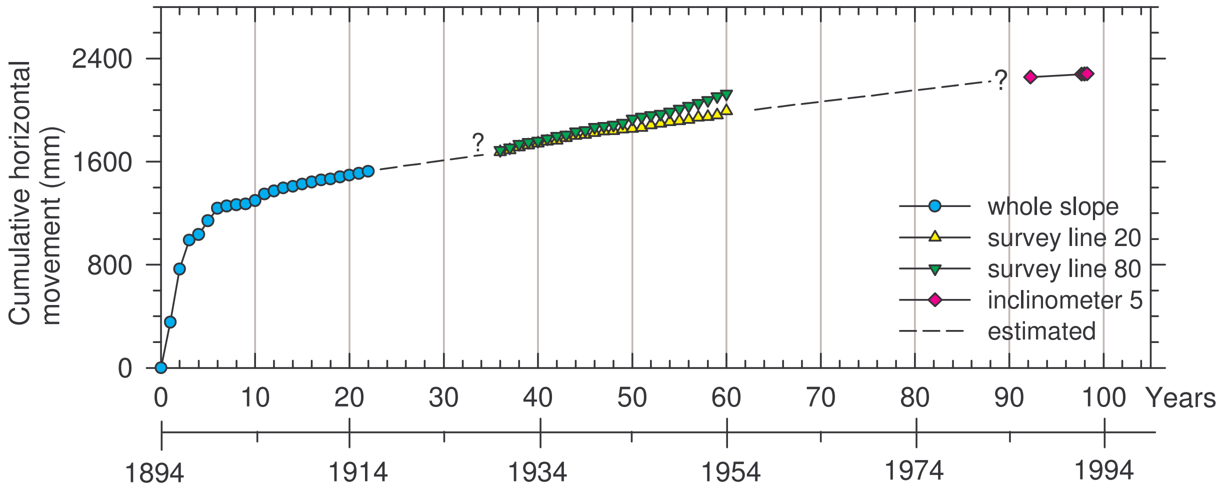

Figure 5. Cumulative movements of the slide between 1894 and 1993 (data from Clarke 1904, Clarke 1918, Cornforth 2005).

August) and No.11 (October-November), but it is believed to have been due principally to the drainage of the underground reservoir connected with Shaft No.1 (until January 1898) (Clarke 1904).

Slide was remediated with a system of drainage along bedrock. Drainage tunnels were completed in September 1901. It be can seen that the drainage tunnels have been very effective in draining away the water pockets and bringing a condition of stability in the sliding mass. In 1925-26, drainage tunnels damaged by landslide movements were repaired or replaced. Extensive surface drains were added (Cornforth 2005).

In movement data shown in Figure 5, survey line 80 is near the top of the slide (at about 550 ft ground elevation in Figure 2), and survey line 20 is near the toe of the slide close to the reservoirs. Both survey lines are stretching the width of the slide in approximately north-south alignment. Movements are average annual horizontal movements of all points within the slide boundaries on the specific survey line. Between 1930 and 1954 survey line 20 moved with a rate of 13 mm/year, and survey line 80 moved slightly faster, with movement rate 18 mm/year.

9 inclinometers were installed in 1987 near the toe of the slide close to the reservoirs at the ground surface elevations of about 250-300 ft to measure the rates of movement at the discrete shear zone more accurately than is possible from surface survey hubs. These inclinometers have been read at three-month intervals until 1993. Inclinometer SI-5 (located near Reservoir 4) shows long term creep movements on the order of 4.6 mm/year (Cornforth 2005).

4 STABILITY ANALYSES

Various researchers have proposed empirical correlations between the residual shear strength and index properties. However most of them have significant scatter (the reasons for which were explained by Mesri and Shahien 2003) decreasing their reliability and applicability to real life problems. The empirical correlation developed by Mesri and Shahien (2003) between the secant residual friction angle and the plasticity index is used in this study. In this method, for a given I_p, the secant residual friction angles at three different effective normal stresses, namely 50, 100 and 400 kPa, are estimated from the empirical information and used to establish a non-linear relationship between shear strength and effective normal stress for the material. The plasticity index of the blue clay in the shear zone above the bedrock is estimated to be 26% (Cornforth, 2007).

Before discussing the slope stability analyses, the difference between the high and low ground water levels should be noted in Figure 3. From the top of the slide to Shaft No.11 there are two blue lines, the higher one indicating the groundwater level before any pumping was carried out. The lower line indicates the groundwater level after pumping. From Shaft No.11 down to the toe of the slide, higher and lower groundwater levels join and are shown by the same blue line. Two-dimensional limit equilibrium slope stability analyses are carried out using Spencer's method in UTEXAS3 computer program.

Considering that there were indications of instability and slope movements before the reservoirs were excavated, it is assumed that the slope was initially marginally stable, and the factor of safety (F.S.) was 1.00. Using the ground surface before the reservoir excavations (see Figure 3) and high groundwater level, the mobilized strength of the blue clay overlying the bedrock was back-calculated for a F.S. of 1.00. The results are shown in Figure 6 as secant friction angles and a non-linear relationship between shear strength and effective normal stress.

We can see that there is a significant nonlinearity to this relation indicated by decreasing secant friction angles with effective normal stress. We also see that the mobilized back-calculated strength in the slide is close to the residual strength of a material with I_p = 33% (instead of I_p=26%). This could be reasonable considering the effects of sample preparation on the measured plasticity index and the residual shear strength (Mesri and Shahien 2003). Also it should be recalled that I_p = 26% was reported from a nearby, geologically similar landslide (Vessely and Cornforth 1998), and not from the shear zone of the Washington Park Reservoir slide.

In the next slope stability analysis, the new ground surface (after the reservoir excavations) is used, and the F.S. is calculated using the back-calculated mobilized strength, and high groundwater levels. This would indicate the effect of the reservoir excavation on the stability of the slope. The F.S. is calculated to be 0.92 in the current study. For a slightly different water conditions, Cornforth (2005) calculated F.S. to be 0.95.

After reservoir excavations were made, and after pumping was carried out, the water levels in the shafts were given by Clarke (1904), that would indicate a lower groundwater level at the top of the slope as shown by the lower blue line. Using this condition, F.S. is calculated to be 1.01. This means that the lowering of the groundwater counteracted the drop in the factor of safety due to reservoir excavations. These groundwater levels reported in the literature may be approximate.

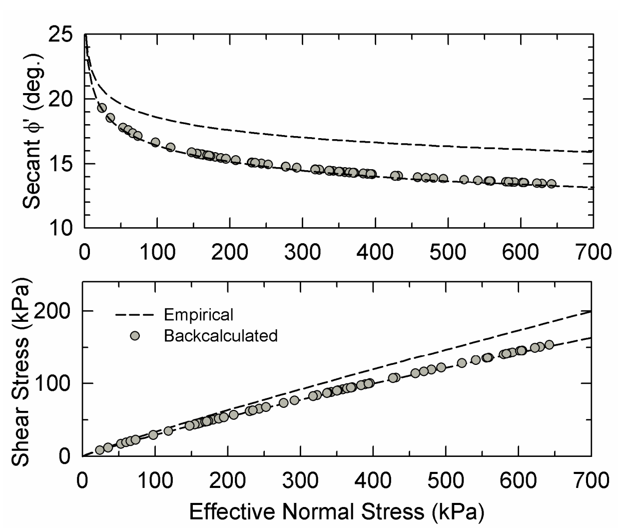

Figure 6. Back-calculated mobilized shear strength for blue clay together with residual shear strengths from Figure 2 of Mesri and Shahien (2003) shown by dashed lines (in both figures upper dashed lines are for I_p = 26%, lower dashed line I_p = 33%).

5 FAILURE TIME PREDICTION

The prediction of the failure time of a landslide is a very challenging and important subject in mitigating landslide hazards. Various researchers proposed methods to predict when a landslide would fail, such as Saito (1965) and Fukuzono (1985) among others.

The inverse-velocity method is used increasingly as it is a useful tool for estimating the failure time of a landslide from the measured rate of movements (Rose and Hungr 2007, Eberhardt 2008). The concept of inverse-velocity for predicting slope failure time was developed by Fukuzono (1985) based on laboratory simulations of accelerating creep conditions. The inverse of observed displacement time rate ("inverse-velocity") was plotted against time, the values approached zero as velocity increased towards failure. A trend line through values of inverse-velocity versus time could be extrapolated to zero value of inverse-velocity (where the velocity would be highest), predicting the approximate time of failure. Fukuzono (1985) presented three types of plots fitted to the laboratory data, concave (α<2), convex (α>2) and linear (α=2), defined by the following equation:

$$V^{-1} = [A(\alpha - 1)]^{1/\alpha - 1} (t_f - t)^{1/\alpha - 1} \qquad [1]$$

where t is time, A and α are constants and t_f is the time of failure. In the laboratory measurements preceding failure, α was found to range between 1.5-2.2. Fukuzono (1985) based on the laboratory simulations concluded that a linear trend fit through inverse-velocity data usually provided a reasonable estimate of failure time as supported by other researchers (Rose and Hungr 2007).

For Washington Park Reservoir slide the rate of movements and inverse-velocities are shown in Figure 4. It can be seen that the movement rate is decreasing and inverse-velocity is increasing with time, therefore it is not an accelerating creep type of movement and it is not possible to apply Fukuzono's inverse-velocity approach for the long-term measured data. Only small incremental time can be used where the inverse-velocity seems to decrease toward zero. If we use the data at months 1 and 2 and a linear relationship between inverse-velocity and time, we would predict a time of failure at 2.5 months. Since the velocities decrease and slide becomes stable with time, Fukuzono's method would not be very useful in this kind of non-accelerating movement data.

When we look at the movement data in terms of the long-term creep movement rates, Saito's method might be applicable. Saito's time to failure prediction method is also used commonly (Picarelli et al. 2004, Bonnard and Glastonbury 2005) and predicts the time to failure using this equation:

$$\log t_r = 2.33 - 0.916 \times \log \dot{\varepsilon} \pm 0.59 \qquad [2]$$

where the time to rupture (t_r) in minutes is obtained from the strain rate (in units of 10^{-4}/min) of the slope. To estimate a constant creep displacement rate we can look at typical slow constant rate movements, such as during 84-108 months where the displacement rate is 0.43 mm/month (5.2 mm/year). At inclinometer SI-5, which is located at about 300 feet ground surface elevation near the toe of the slide to the west of reservoir 4, the average creep rate was 4.6 mm/year between 1987-1993. Therefore the average creep rate of Washington Park Reservoir slide can be taken as about 5 mm/year. Also it can be noted that in Washington Park Station slide (within 1.6 km distance to Washington Park Reservoir slide) with similar geologic and environmental conditions the creep rates were 5.6 mm/year between 1975-1976 and 0.8-1.8 mm/year between 1987-1994 (Myers et al. undated).

As was done by Saito (1965) we can use the length of the slope (520 m) to convert measured horizontal deformation rate of 5 mm/year to a strain rate. Therefore creep strain rate of this slide is about 9.62×10^{-6}/year. When used in Saito's equation in terms of the units required, average time to failure can be obtained as 604 years (with a range 155-2349 years) with the assumption that the slope will continue to deform at 5 mm/year constant creep rate without any change in other parameters influencing the stability of the slope. If we had used the highest deformation rate observed in the history of this slide (such as in month 13 in Figure 4) 74 mm/month, we would estimate a time to failure of 5 years (with a range of 1-20 years) if the slope had continued moving with this rate. Saito's method could be a useful tool for setting up threshold deformation rates and corresponding alert levels for landslide hazard mitigation.

Another way of looking at the deformation rates to estimate failure time could be in terms of the acceleration of the movements. Picarelli et al. 2004 presents a plot between the acceleration of the slope movements and the time to failure from slope failures in the literature. In Washington Park Reservoir slide, there have been relatively high accelerations at month 4 (39 mm/month^2) and at month 23 (33 mm/month^2). But the maximum acceleration occurred in month 12, and it was 42 mm/month^2 (or 4.7×10^{-5} m/day^2). That could be arguably the closest condition the slope came near failure. If the movements had continued with that acceleration, it would have lead to the failure in about 30-100 days (Picarelli et al. 2004 Fig.10). The acceleration decreased in the dry season following month 12. In the long-term, the drainage remedial measures prevented the slope from accelerating, and helped stabilize it at a constant velocity creep deformations. Estimating time to failure based on acceleration values could be a useful approach in establishing warning systems in landslide prone areas.

For active landslides with very slow (< 1.6 m/year) to extremely slow (< 16 mm/year) movements (IUGS 1995) concentrated within a narrow shear zone, where the rate of movement is more or less uniform with depth, the factor of safety of the slide is often in the range of 1.20-1.01 and movements are often controlled by rainfall-induced pore pressure fluctuations (Calvello et al. 2008, Bonnard and Glastonbury 2005, Eshraghian et al. 2008). Relating the displacement rate of a slope to the factor of safety has been an interest for geotechnical engineers, especially in the recent decade. For example, Calvello et al. 2008 proposed "R-u-F-v" approach where changes in the pore pressures (u) can be estimated from rainfall (R), and it is used to calculate factor of safety (F) at different times, and correlated with the displacement rates (v). In addition, they have successfully applied an optimization algorithm for calibrating the parameters used in the analysis. As emphasized by Calvello et al. 2008, this method needs a good understanding of the movement mechanism(s) of the slide, and considerable geotechnical engineering judgement as parameters are being calibrated. To be able to use "R-u-F-v" approach, one needs the pore pressures in the ground with time in addition to the movement rates. Since pore pressures in the ground were not measured at different times in the slide analyzed in this study, this promising method could not be applied.

Bonnard and Glastonbury (2005), based on Glastonbury and Fell (2002), presented relations between the F.S. and slide velocity for earthflows and debris slides from the literature (Figure 7). They used infinite slope analysis approach. Due to uncertainties in the calculated F.S., they present data in terms of "relative change in the F.S." For each slide, the condition of highest groundwater level (the least stable condition) was considered to represent a relative F.S. of 1.0. The other conditions are normalized against this value.

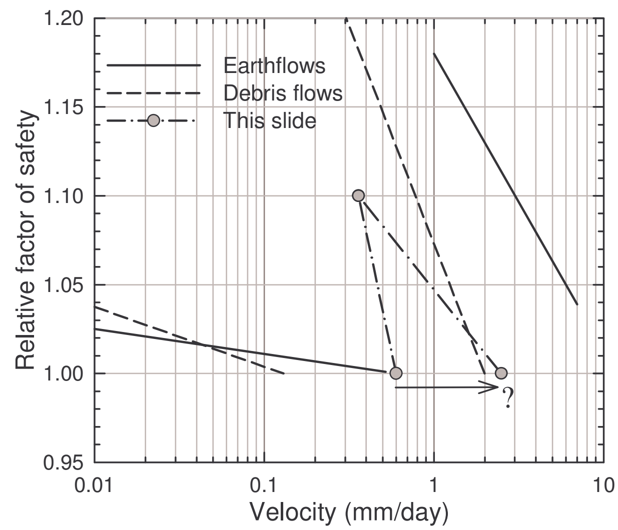

Figure 7. Relation between relative factor of safety for earthflows and debris slides taken from Bonnard and Glastonbury 2005, shown with estimates for the landslide in this study.

For earthflows where the majority of the movement is uniform between the ground surface and the basal sliding plane, the sensitivity of the slide to fluctuations in groundwater level is possibly related to velocity of movement of the earthflow. Therefore, Bonnard and Glastonbury (2005) calculated F.S. for changing groundwater conditions and related that to the movement rate of the slope. They present similar relations for debris slides, which are reported to have internal deformation (i.e. non-uniform movements on the ground surface and at depth), relating measured groundwater levels and calculated relative F.S. to observed slide velocities.

For Washington Park Reservoir slide, F.S. was calculated as 0.92 after the excavations were made for reservoir construction and the groundwater level was high. Slope stability analysis by Spencer's method and a nonlinear shear strength envelope is used in UTEXAS3. Although there is uncertainty in the time when the groundwater levels were this high and corresponding rate of movement, for the sake of reaching to an estimate, this time is assumed to be close to January-February 1895 where initial movements were being measured (average about 18 mm/month). The second case is after the reservoir excavations were made, and water level is lowered by pumping (in March 1899, cross-section and water levels reported by Clarke 1904) for which the F.S. is calculated to be 1.01. During that time, movement rates were slower (about 10 mm/month). These factor of safety values can be plotted in Figure 7 using a "relative factor of safety approach", whereby the values are normalized by the lowest one (F.S.=0.92). The highest movement rate of the slide, observed in January-February of 1896 in Figure 4, is also shown in Figure 7 for the possibility that the movements might have been the highest at that time. More information about the piezometric levels at different times and corresponding rate of movements for Washington Park Reservoir slide is needed to improve such an initial estimate. However, plotting even an initial estimate would help to compare with the typical range of velocities for other slides, and see the sensitivity of the slide to changes in groundwater levels.

6 SUMMARY AND CONCLUSIONS

Close relation exists between the rainfall and the movement of the slide. With each recurring dry season there was a corresponding cessation of movement, and with the beginning of winter rains the movements increased. The pumping and drainage tunnels were very effective in reducing the movement rates to extremely slow creep rates.

Mobilized shear strength in the slide was backcalculated. The empirical relation between plasticity index and the residual shear strength is found to be valuable especially when there is not enough laboratory investigation on the shear strength of the material.

Various failure time prediction methods are investigated and it is discussed that they can be very useful in establishing threshold values in warning systems.

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