

STABILITY INVESTIGATIONS OF THE WHAMPOE SLIP

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SYNOPSIS

This paper describes a slip in soft, blue clay along a length of the Whampoe River bank which had been excessively loaded with stacks of granite stones. A total stress stability analysis, based on undrained shear strengths obtained by field vane, is carried out for both the slip and the unloaded stable slope. The minimum factor of safety obtained for the slip is 0.97 and that for the stable slope is 1.45. The results show that the slide was caused by the external loading and that the ' $\phi = 0$ ' method of analysis gives a correct estimate of the stability of the river bank.

INTRODUCTION

In March 1967 the Public Works Department of Singapore undertook to deepen and widen a stretch of the Whampoe River between May Road and St. Francis Road. The design called for the steepening of the existing, comparatively flat slope to a slope of 1 : 2 which was to be lined with riprap. It also provided for 4 in. diameter bakau (timber) piling at regular intervals of 5 ft to a depth of 9 ft at the toe of the slope. By August 1967 the job was completed. On 24th September 1967 a slide took place in the soft, blue clay stratum along a length of the river bank adjacent to which were stacked piles of granite stones. This was followed by a second slide immediately upstream of the first on 7th March 1968. An investigation was initiated to determine the causes of the slip, and this paper presents the results of the investigation.

DESCRIPTION OF THE SLIDE

The slide occurred on 24th September 1967. It was a deep-seated one which stretched for a length of approximately 137 ft with a maximum width, measured from the edge of the bank, of about 40 ft (see Fig. 1). The slide destroyed the riprap lining completely and chunks of soil appeared to have been pushed up from the river bed to the opposite bank. The extreme landward end of the slide surface sank about 3 ft. The slip seemed to be very sudden. At the time of the slide stacks of granite stones about 7 ft high, which had been placed there sometime in June or July, were seen on the bank;

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the water in the river was then only about 6 in. deep. The external loading, coupled with the low water level, was probably the main cause of the slip.

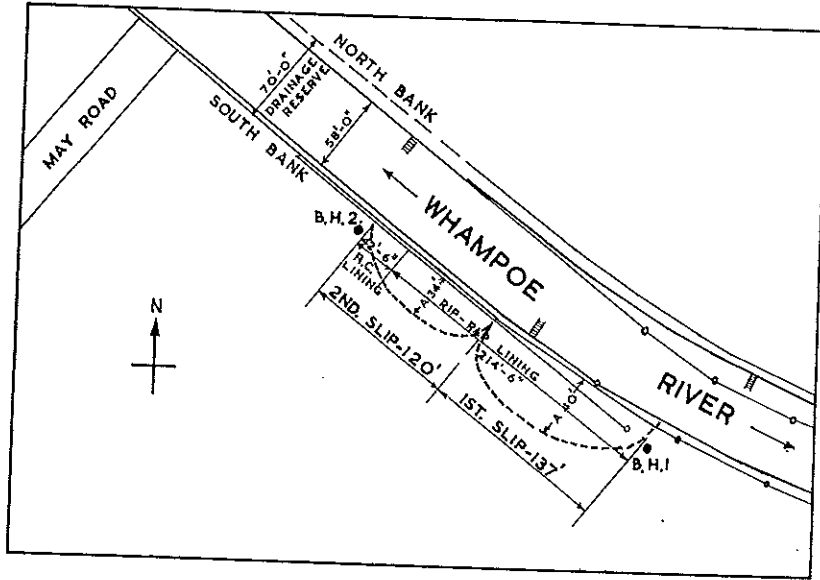


Fig. 1. Site plan of slip area showing positions of borings.

The second slip took place on 7th March 1968 immediately upstream of the first slip. Like the first slip, it was deep-seated and measured 120 ft long with a maximum width of 34 ft. It also destroyed the riprap lining and choked up the river. The stacks of granite stones remained in their original positions except that their height was diminished to about 3 ft. In neither of the slips were tension cracks observed.

FIELD AND LABORATORY TESTS

Two borings were sunk in the positions shown in Fig. 1. For Borehole 1 field vane tests using Bishop-type vane equipment (SKEMPTON, 1948) were carried out and 4 in. diameter undisturbed samples were taken, generally at 5 ft intervals, by manual boring. Unconfined compression tests, consolidated undrained tests with pore pressure measurement, consolidation tests and classification tests were performed for these samples at the P.W.D. Engineering Laboratory. For Borehole 2 only field vane tests were made. No continuous core sampling was carried out in the clay strata because of

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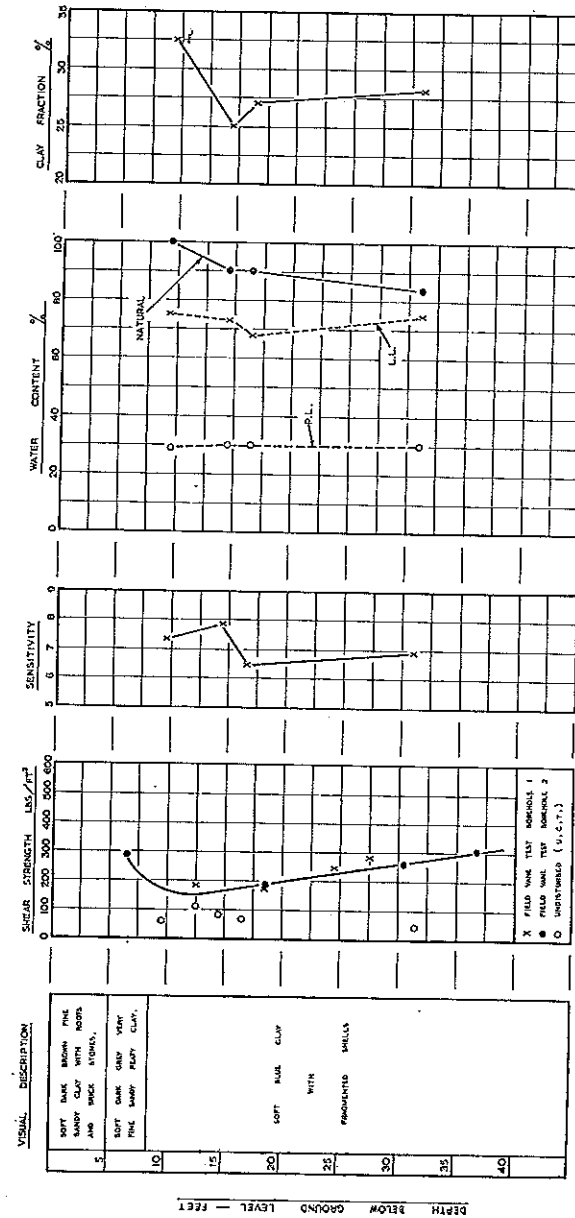


Fig. 2. Borehole log and test results for Borehole 1.

the lack of suitable equipment. Consequently, the exact position of the slip surface is unknown.

The results of boring and laboratory tests for Borehole 1 are shown in Fig. 2. Except for the first few feet of surface soil, soft, blue clay with fragmented shells predominates up to the end of the boring. The moisture content is high (about 100%) and decreases slightly with depth. The liquid limit drops slightly with increase in depth but the plastic limit remains almost constant; it should be noted that the Atterberg limits were determined on oven-dried samples. This gives a decrease in plasticity index with depth which is reflected in the observed decrease with depth of the clay fraction. Vane test results show an approximate linear increase in undrained shear strength with depth, the higher values for the top few feet presumably being due to desiccation. The results for Borehole 1 agree well with those for Borehole 2, suggesting consistency and uniformity. The value of c_u/p' is 0.27. For the plasticity index shown this value conforms with the well-known relationship between c_u/p' and plasticity index given by SKEMPTON (1957). This observation suggests that the clay is normally-consolidated, which is confirmed by the results of consolidation tests, although the preconsolidation pressures for these tests are poorly defined.

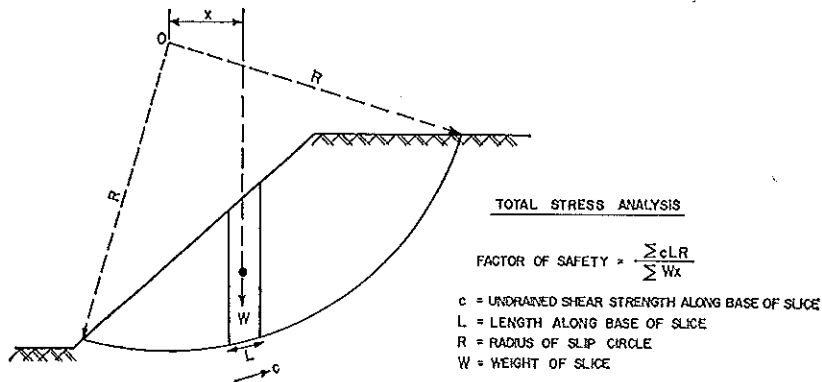


Fig. 3. Outline of the ' $\phi = 0$ ' analysis.

Unconfined compression test results lie below those of field vane tests. It is felt that the numerous shells present in the laboratory samples tested are responsible for the lower unconfined strengths. Consolidated undrained tests give $c' = 0$ and $\phi' = 22^\circ$; these parameters are useful for effective stress stability analysis in cases where the total stress method is unsuitable.

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The sensitivity, which is defined as the ratio of field vane strength to remoulded vane strength, shows a slight decrease with depth and ranges from 6 to 8, indicating that the material is very sensitive. The bulk density, in general, is fairly constant with depth at a value of 93.8 lb/ft.³

' $\phi = 0$ ' ANALYSIS

The outline of the ' $\phi = 0$ ' analysis for the slide is shown in Fig. 3. This method is based on the assumption that there is no change in soil water content on loading (i.e. failure takes place under undrained conditions). When a fully saturated clay is sheared under such conditions, it will behave as if it were a purely cohesive material with $\phi = 0^\circ$. The undrained shear strength is, hence, appropriate for use in the stability analysis. In the stability calculations for the Whampoe slip the undrained shear strengths determined by the field vane test were used, since it was felt that these results were the most reliable and representative.

In Fig. 4 the results of the stability analysis for the slip are presented. The minimum factor of safety obtained is 0.97. This is less than the value

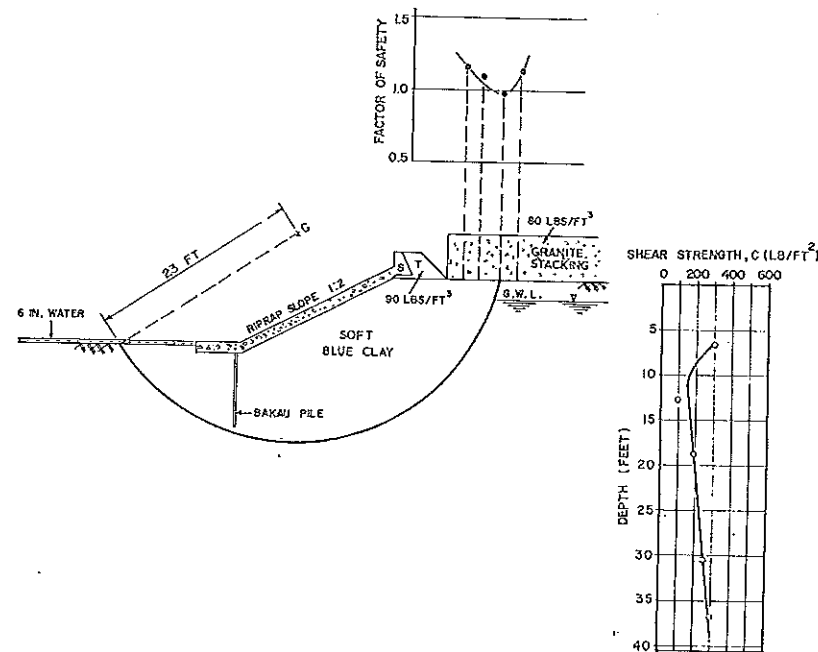


Fig. 4. Stability analysis of slip ($\phi = 0$).

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of 1.0 at which failure should occur, and the difference is probably due to the fact that the effects of the forces on the ends of the cylindrical soil body under consideration were ignored in the calculations. Nevertheless, the critical factor of safety of 0.97 is very close to unity, from which it is evident that the total stress analysis gives a good estimate of the stability of the Whampoe River bank.

The stability analysis for the unloaded slope is shown in Fig. 5. The minimum factor of safety is 1.45. Therefore, the slope is stable so long as there is no excessive surcharge loading on its bank, which accounts for why no failure of this slope has occurred.

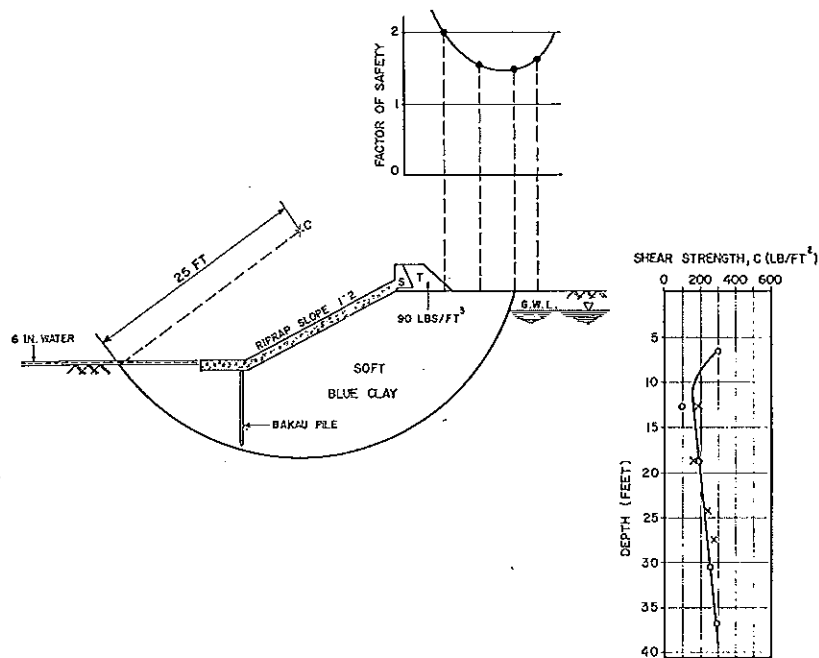


Fig. 5. Stability analysis of stable slope ($\phi = 0$).

CONCLUSIONS

The ' $\phi = 0$ ' method of stability analysis gives an accurate determination of the stability of the Whampoe River slope. The critical factors of safety of approximately 1.0 for the slip and 1.45 for the unloaded slope show that the main cause of the slide that occurred was the external loading in the form of the stacks of granite stones.

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