Slip Failure of Embankment on Soft Marine Clay

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SYNOPSIS The results of an investigation carried out at a site before and after the slip failure of an embankment on soft marine clay are presented. An evaluation of the possible causes for the slip failure and the remedial measures taken after the failure are also included. It appears that overstressing of the ground might have led to the movement of the soft soil. The compressible soils were subjected to an increase in stress without a comparable increase in foundation shear strength. This increase in horizontal stress resulted in the horizontal displacement of the soil as evident by the presence of the tension cracks in the fill just before slip failure occurred.

INTRODUCTION

On July 29, 1982, during the peak of construction activities on a site, its embankment slipped into the adjacent canal causing part of the canal walls to collapse. The construction site, separated from the canal by a 30 m wide drainage reserve, is for a proposed development of a vehicle depot with offices and workshops as shown in Fig. 1. All the buildings at the site are supported by piled foundations and are in various stages of construction when failure of the embankment occurred. Placement and compaction of fill for the vehicle parking area were also actively in progress at that time. Site observations indicated that the failure encompassed a length of about 100 m along the canal and extended across the drainage reserve to about 15 m into the construction site. Numerous tension cracks opened up in the fill soil on the site and drainage reserve. A pipeline, which runs parallel to the canal and supported by pad footings at about 10 m intervals, moved laterally by as much as a metre towards the canal at some locations before bursting at one point. It appears that overstressing of the ground due to the construction activities and placement of fill led to the movement of the soft soil.

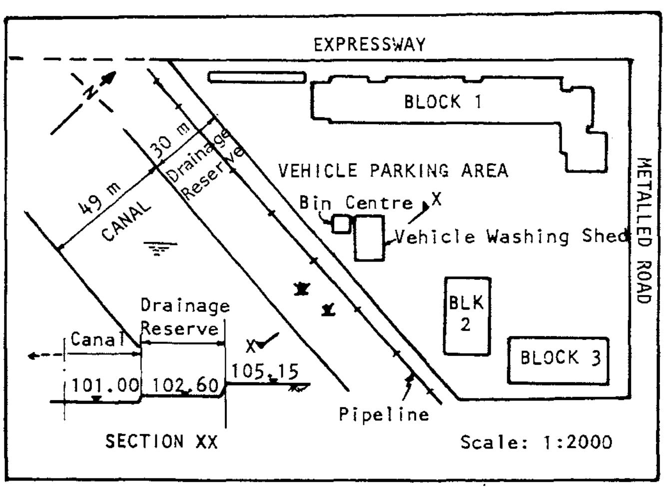

Fig. 1. Site Plan of Proposed Vehicle Depot

To prevent further movement of the slip and additional damage to the canal, the slope in the drainage reserve was regraded and about 1.0 m of fill adjacent to the drainage reserve was removed by the site contractor. Construction activities also ceased in the vicinity of the drainage reserve boundary.

SUBSOIL CONDITIONS

Based on earlier site investigation, a typical section of the soil profile at the site is shown in Fig. 2. The subsoil consists generally of a top layer of fill deposits of medium stiff silty clay. Below this top layer is soft marine clay overlying medium dense to very dense clayey sand. Ground water-table observed from the boreholes was about 1.5 m to 2.5 m below the existing ground level at the site.

From the soil information, it appears that the vehicle depot development is founded on soft ground where the thickness of soft clay is typically about 10 m. The shear strength of this layer is so low that large settlement and low stability can be expected.

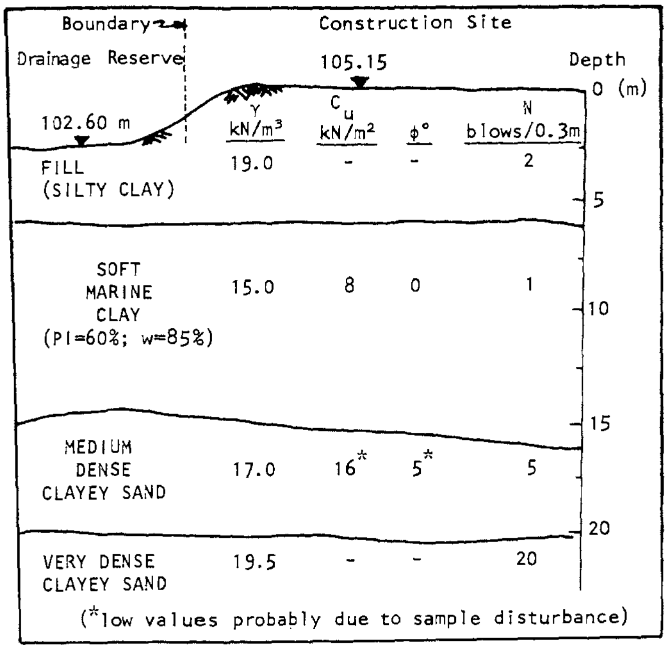

Fig. 2. Typical Soil Profile

SLIP FAILURE OF EMBANKMENT

The plan and typical sections of the site at various stages of events, before construction of the car park formation to the removal of collapsed earth and regrading of slope, are illustrated in sequence in Fig. 3. Ground levels for various stages were measured by surveying techniques.

To construct the parking area for vehicles, additional placement of fill was required for the whole site. It can be seen from Fig. 3(a) and (b) that an additional layer of fill of about 0.3 to 0.5 m was placed at the site. In addition, fill of about 2.85 m high was placed at the boundary of the site and drainage reserve, extending about 2 m from the toe of the original slope. Unfortunately, there was no available records to indicate the actual quantity and the rate of placement of fill except the fact that the fill placement and compaction began about two weeks prior to the day of failure.

Eye-witnesses at the site indicated that tension cracks were observed on the fill on the morning of the day in which failure occurred. Moments after this observation, the pipeline burst resulting in water overflowing into the fill at the drainage reserve. This was followed subsequently by the slip failure and upheaval of the banks of the canal as shown in Fig. 3(c). Failure encompassed a length of about 100 m along the canal and extended across the drainage reserve to about 15 m into the construction site. The eye-witnesses further noted that failure took place suddenly with out any perceptible vibration. At the end of failure, settlement of the fill at the

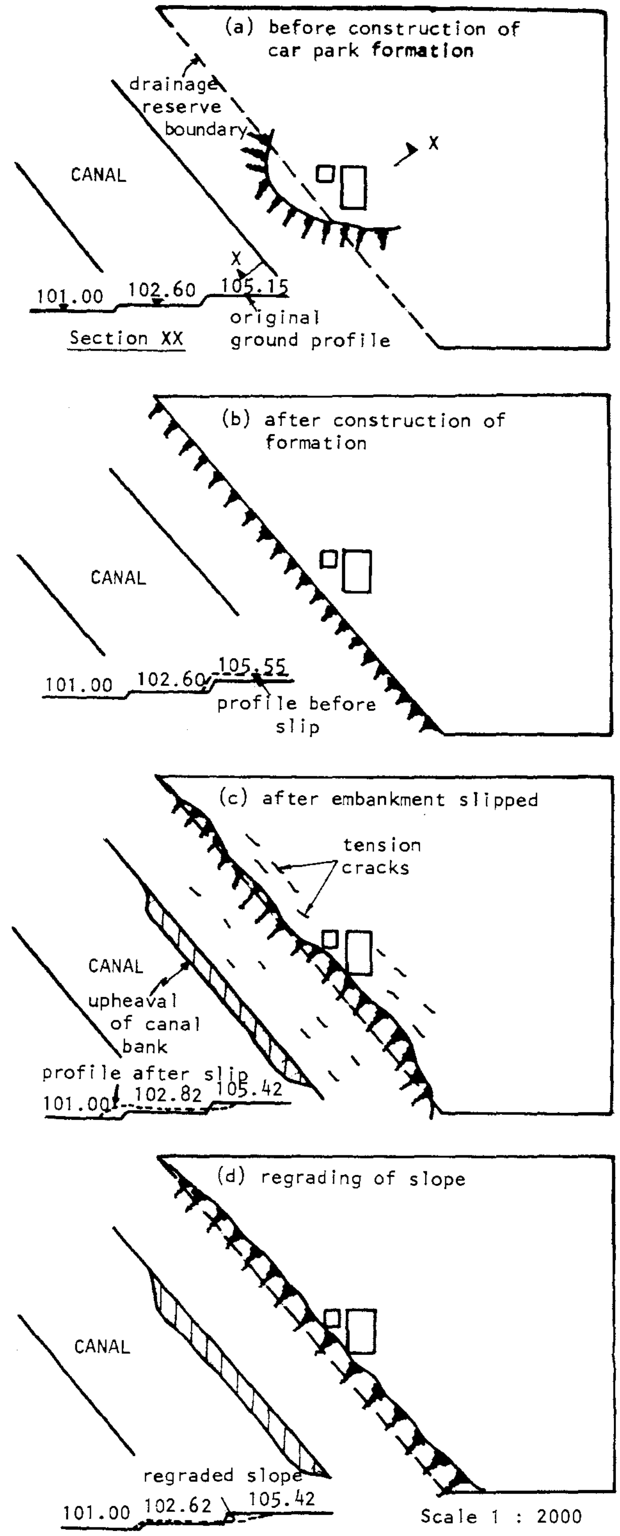

Fig. 3. Sequence in Slip Failure of Embankment

boundary had reached over a metre, while the floor of the canal heaved close to a metre. The bin centre and vehicle washing shed, which are supported by tanalised timber piles showed no structural damage apart from some minor cracks in the floor slabs. The rest of the buildings, which are supported on precast concrete piles, appeared to be unaffected by the slip failure.

To prevent further movement of the slip and additional damage to the canal, construction activities ceased in the vicinity of the drainage reserve boundary and the collapsed soil was removed by the site contractor. The removal of the collapsed soil and the regrading of the slope shown in Fig. 3(d) was completed in five days. Although no records were available on the amount of earth removed, the site contractor estimated that about 480-lorry loads were involved.

STABILITY ANALYSIS

Five days after failure, six additional boreholes were carried out in the slipped area. The diameter of the boring was 150 mm and casings were used to prevent the collapsed of the soft clay and loose sand formation. In addition, vane shear tests were also conducted. Results of the field and laboratory tests were consistent with the findings given in earlier soil reports.

In establishing the reasons for the failure, factual evidence rules out a deep seated circular slip. The existence of a zone of very soft soil directly below the fill suggest a preferential plane of failure in this layer. Embankments on soft clay induce zones of overstressing. Elsewhere along the potential failure surface, stress levels can be sufficiently high that creep strains become important. Natural sedimented clays possess critical strain levels at which clay structure breaks down. This breakdown in clay generates the excess porewater pressure and a progressive undrained failure can occur rapidly (Bjerrum, 1972).

Stability analysis have been carried out considering critical circles using total stress analysis. The approach to stability analyses are by the method of slices and the design charts given in Navfac DM-7 (1982). Results of the analyses are summarised in Fig. 4. The standard slip circle analysis, using undrained shear strength calculated from results of triaxial test and vane tests, is adequate for the prevailing site conditions. In the analyses, the following conditions and assumptions were made:

(i) The fill surcharge is applied rapidly on the clay stratum where no provision is made to drain porewater during stress changes.

(ii) The vane shear strength is corrected for anisotropic conditions and strain rate (Bjerrum, 1972).

(iii) Soil parameters used in the stability analysis are based on minimum strength condition.

It can be seen from the results that the critical slip failure passes through the soft clay stratum giving factors of safety ranging from 0.91 to 1.21. This circle is in good agreement with the exterior signs of failure.

Method of Analysis	Shear Strength (kN/m^2)	Factor of Safety
Method of slices	8 (Undrained Triaxial)	0.91
Method of slices	10 (Vane Shear Test)	1.21
Navfac DM-7 (1971)	8 (Undrained Triaxial)	1.02

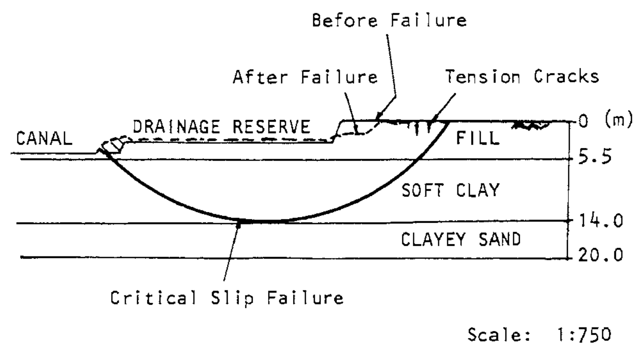

Fig. 4. Results of Slope Stability Analyses

POSSIBLE CAUSES OF FAILURE

In examining the possible causes of failure, it is worth noting that it is difficult to reconstruct the actual sequence of events that led to failure; thus factors which initiated the instability seldom can be positively identified. However, based on field observations and information provided by the site contractor and engineer, the events leading to failure can be postulated.

During the initial placement of about 5 m of fill on the marine clay, 12 months before the present contractor moved in, no sub-surface drainage was installed to dissipate the excess porewater pressure generated. Furthermore, no piezometers were installed to monitor the change in porewater pressure in the 10 m thick marine clay. The high water-table in the fill observed at the site may indicate the presence of excess porewater pressure. Therefore, when the site was possessed by the present contractor, there was no information to indicate the effective stress condition of the subsoil. The construction activities at the site and the placement of the additional fill at the boundary of the drainage reserve might have triggered the failure in view of the marginal factor of safety for slope stability.

Therefore, it appears that overstressing of the ground might have led to the movement of the soil. The compressible soils were subjected to an increase in stress without a comparable increase in foundation shear strength. The increase in horizontal stress would result in horizontal displacement of the soil. This was evident in the tension cracks in the fill before slip failure occurred. In fact, the tension cracks were a warning sign of impending instability.

The lateral movement of the ground resulted in the pipeline bursting at one of the joints. Although the control valves of the pipeline was turned off 20 minutes after the bursting, the water from the pipeline had overflowed and seeped into the fill and this undoubtedly increased the lateral stress on the walls and floor of the canal. Collapsed of the walls and heaving of the floor of the canal occurred soon after as a result of increased soil and hydro-static pressure.

REMEDIAL MEASURES AND PRECAUTIONS

Remedial measures were required as soon as possible to prevent any further movement. Survey measurements taken at the site regularly after failure indicated that ground movements towards the canal was still in progress although at a smaller magnitude. It was feared that these movements might increase during period of heavy rainfall or increase site activities.

The use of counter-berm to stabilise the movement was not possible because of its proximity to the canal. As an expedient measure, the slope was regraded gently to 1:8 as shown in Fig. 3(d) and excess soil removed to relieve some of the stresses o soft marine clay. Inclinometers were r mended to be installed at the drainage re to monitor any ground movements during regrading and the subsequent repair of canal.

CONCLUSIONS

The stability analysis given in this pape by no means comprehensive. Original g conditions were irreparably altered. Howe from field tests and observations, it ap that overstressing of the ground caused sudden failure with rapid soil displace Tension cracks in the fill are signif indication of initial ground movement and of impending failure. In view of the foundation soil at the site, some instrume tion such as inclinometers and piezome should have been installed before constru commenced at the site. Readings from devices would have forewarn any impe instability in the soil.

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