

INTERNATIONAL SOCIETY FOR SOIL MECHANICS AND GEOTECHNICAL ENGINEERING



This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

<https://www.issmge.org/publications/online-library>

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.

Stabilization of a Quick Clay Slope by Vertical Sand Drains

Stabilisation d'un talus d'argile fluente par des drains de sable verticaux

by O. S. HOLM, M.N.I.F., Associate partner Norsk Teknisk Byggekonskontroll, Consulting Engineers, Oslo, Norway

Summary

The paper deals with an investigation of a natural quick clay slope at the west side of the Oslofjord, Norway.

A retaining wall and a pond will be placed on top of the slope as part of an industrial plant. If any slides should occur, oil, water and liquid clay might flow into the fjord.

Pore pressure observations show that the ground water level at the top of the slope is 1 m below the natural ground surface, and that there is excessive pore water pressure at the toe of the slope.

A stability analysis on the unloaded slope with total stresses results in a safety factor less than unity. An analysis with effective stresses on the unloaded slope results in a factor of 1.6 and for the loaded slope of 1.4. It was concluded that the stability of the slope was unsatisfactory.

It was decided to reduce the water pressure by means of vertical sand drains and drainage trenches, and to place a counter-fill along the shore line.

Introduction

Unstable natural slopes are very common in Norway — and every year major landslides occur. The frequency of these is high particularly in the quick clay regions, and the damage and casualties may be of catastrophic dimensions. The causes are usually erosion and exceptionally high rainfall, but equilibrium may also be destroyed by human action.

The classic manner of calculating the stability of slopes is by $\Phi = 0$ -analysis in connection with the circle of rupture method. In recent years, stability calculations have been extensively based on the $C - \Phi$ -method, i.e. effective stresses, in which the shear strength parameters are determined by triaxial tests on consolidated undrained samples. It seems, however, that this method of calculation overestimates the safety factors since calculation sometimes gives them as being considerably higher than 1.9.

The authors deals with the investigation of a natural slope and the measures taken to give adequate safety against failure. The slope is 10 metres high and extends 300 metres along the Oslofjord. The average inclination of the slope is 20°. The slope was probably formed by erosion and a prevailing current along the shore. Irregular formations in the slope indicate previous slides. The farm on the site bears the characteristic name of "Landfall" — a not uncommon name in Norway.

The industrial facilities on the site include the construction of a retaining wall 2 to 3 metres high on top of the slope, which, in case of emergency, will impede the flow of oil and polluted water out into the fjord. A road is to be constructed at the toe of the slope.

Piezometers installed off-shore and on the beach proved that considerable artesian pressure existed in the bedrock.

Sommaire

La communication est relative à l'étude d'un talus naturel d'argile fluente, sur le côté ouest du fjord d'Oslo, Norvège.

Les besoins d'une installation industrielle comportent la construction en crête du talus, d'une digue en terre créant un bassin de décantation. En cas de glissement, l'huile, l'eau et l'argile liquide, seraient déversées dans le fjord.

Des mesures de la pression interstitielle ont montré que le niveau piézométrique était à 1 m sous le sol naturel au sommet du talus, mais que la surpression était considérable au pied du talus.

Pour le talus non chargé, une analyse de la stabilité en tensions totales, conduit à un coefficient de sécurité inférieur à 1, et une analyse en tensions efficaces conduit à un coefficient de 1,6. Cette valeur passe à 1,4 lorsque le talus est chargé. Il a été considéré que la stabilité du talus n'était pas satisfaisante.

Il a été décidé de réduire la surpression de l'eau au moyen de drains de sable verticaux et de tranchées drainantes, et de placer un contre-remblai le long du rivage. L'auteur discute sur l'efficacité et l'écartement des drains.

Since the stability conditions of the natural slope were considered to be unfavourable, the application of loads on the crest of the slope could perhaps cause a failure. It was therefore decided to install vertical sand drains to reduce the excessive pore pressure, so that the stability of the slope would not be undermined during application of loads at the crest of the slope.

Soil Conditions

Extensive soil investigations have been carried out on the site. The investigations include many soundings, test borings with piston samplers, vane borings, and pore pressure installations. The location of the borings and the piezometers are shown on the plan (Fig. 1). The investigations show that below a layer of organic top soil the ground consists of 3 to 4 m of sand and gravel along the crest of the slope. The material becomes gradually more silty with increasing depth. The thickness of the layers of the granular material decreases gradually towards the sea and becomes insignificant at the toe of the slope. At greater depths a deposit of soft quick clay extends into the sea. Seams of silt, sand and gravel are embedded in the clay.

In general, there appears to be a layer of dense sand and gravel overlying the bedrock.

The thickness of clay deposit is, on the average, about 10 m, with a maximum thickness of approximately 18 m. On Fig. 2 is shown a soil profile together with the results of the laboratory tests on undisturbed samples taken at the shore line, and the results of a nearby field vane test. The soil consists of very sensitive quick clay. The field vane test indicates an undrained shear strength as low as 1 ton per sq. m., whereas unconfined compression tests on undisturbed

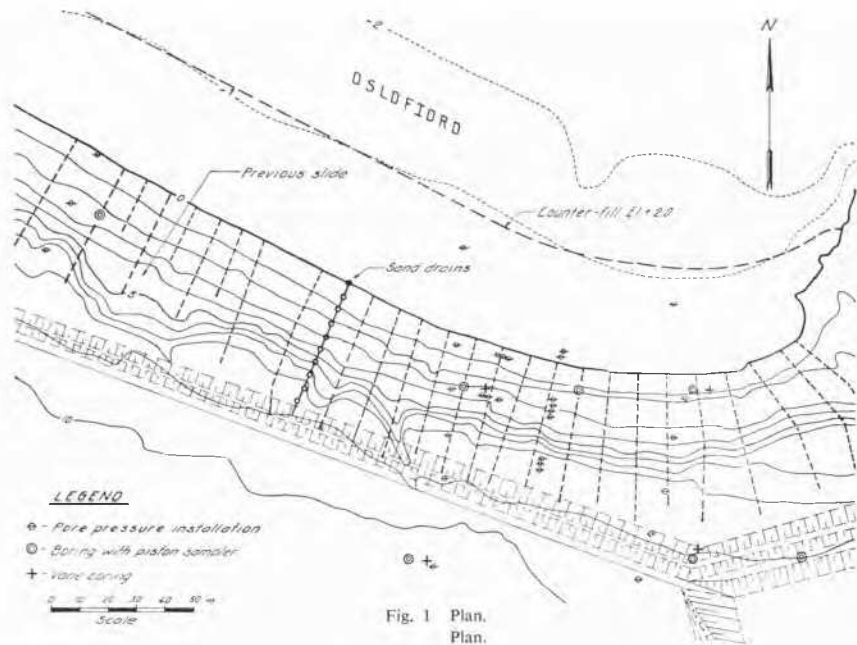


Fig. 1 Plan.
Plan.

samples show a shear strength of 1.7 to 2.0 tons per sq. m. This discrepancy is probably due to a considerable content of coarse material in the clay.

The natural moisture content varies between 30 and 40 per cent, which is 6 to 17 per cent above the liquid limit. The clay is therefore liquid in the remoulded state. The plasticity index is 5 to 9 per cent. These values are typical of Norwegian quick clays.

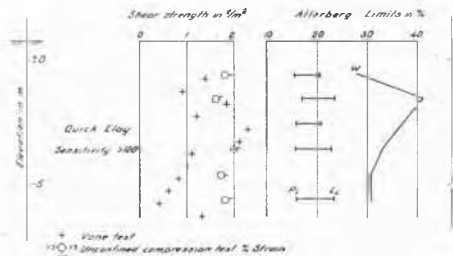


Fig. 2 Typical soil profile.
Profil des caractéristiques du sol.

Consolidated undrained triaxial tests with measurements of pore pressures on undisturbed samples have been carried out, and the tests indicate an apparent cohesion of $c = 0.5$ ton per sq. m. and an apparent angle of friction of $\Phi = 25-8^\circ$.

Pore pressure measurements

The piezometer consists of sintered bronze filters mounted on a hollow stem connected to a plastic tube contained within steel tubing.

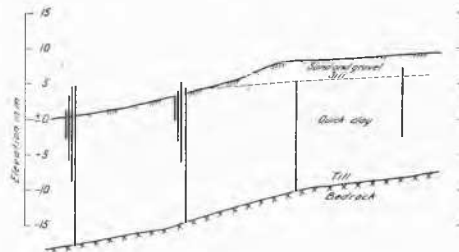


Fig. 3 Section. Pore pressure installations. Piezometric heads at measuring points prior to drainage installation. Coupe. Mesure de la pression interstitielle. Charges piézométriques avant drainage.

The piezometric head in the plastic tube is measured by lowering a thin electric cable connected to a micro-ammeter down the tube until it touches the meniscus.

A small amount of an anti-freeze fluid having electrolytic properties is poured down the plastic tube. A Bourdon gauge is fitted for measuring excess pressures.

On Fig. 3 is shown a section through the slope and the corresponding piezometers. The piezometers are indicated by full lines drawn from their points to the original piezometric heads.

Behind the crest of the slope the free ground water level varied between ground surface in wet periods, and 1.5 m below ground surface in dry periods. On the shore line, and off-shore, an excess pressure of 5 m was measured at bedrock, gradually decreasing with decreasing depth. Water leaked steadily out of the ground in the lower half of the slope.

Stability

Stability calculations of the natural slope in terms of total stresses indicated a safety factor of 0.7. Stability calculations in terms of effective stresses, based upon the measured pore pressures in the ground, resulted in a safety factor of 1.6 (Fig. 4). Assuming the additional loads to be the retaining wall at the crest of the slope and the oil-filled basin behind the wall, and assuming that these loads result in 100 per cent pore pressures in the clay, the safety factor is reduced to 1.4, which is considered unsatisfactory.

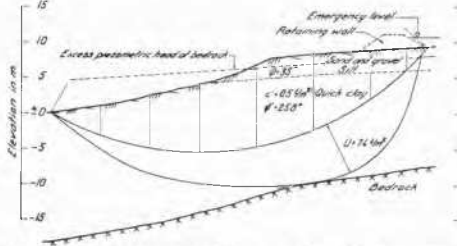


Fig. 4 Natural slope. Stability analysis. Effective stresses. Talus naturel. Analyse de stabilité. Tensions efficaces.

It was therefore decided to take steps to reduce pore water pressure permanently, so that the safety factor would not be less than 1.6.

A stabilizing system of vertical sand drains was proposed which would act as relief wells of the bleeder type. The primary effect of the drains would be to reduce the pore pressures at bedrock, and in the sand layers in the clay deposit. This reduction would both occur rapidly, and increase stability. The secondary effect would be to consolidate the clay. The rate of consolidation would be accelerated by the drains.

To try out the boring equipment, and to check if relief of the artesian pressure by means of sand drains was feasible, two test drains — each 6 in diameter — were installed at the shore line. The boring equipment consisted of diamond core drill rods connected to a wash crown with a diameter of 6 in. Water was pumped down through the rods and out through the nozzles at the circumference of the crown. Simultaneously the rods were rotated and the soil sluiced out of the borehole. Mud was blown out of the hole by compressed air after the boring was finished. Experience showed that the hole did not collapse. The hole was then filled with water and washed until there was no clay in the run-off. Finally, the full depth of the hole was filled with sand filter material. The material used for the test drains was chosen such that 95 per cent of it lay within the grain fraction of 1 to 8 mm. The run-off from the test-holes was under observation for four months, and the drains did not show any signs of being clogged by infiltration of silt and clay from the surrounding ground. The water flowed steadily at the rate of about half a gallon per minute. Nevertheless, it was decided to use a more well-graded material with a 10 per cent content of particles of diameter less than 0.5 mms to meet the usual filter criterion. The diameter of the drains was increased to 8 in to compensate for the decrease in permeability.

In the finished project some 315 drains were installed. The excess water from the drains and from the upper soil strata was collected by a total of 31 drain conduits in separate trenches. The conduits consisted of 6 in diameter concrete pipes, embedded in a filter of coarse gravel, passing over the vertical drains. The trenches were 2 to 3 m deep at the crest of the slope, and 0.5 to 1.0 m at the toe. The length

was 52 m. On the plan (Fig. 1) the trenches are indicated by thin lines at right angles to the shore line. On one of the lines is indicated the location of the sand drains installed in that particular trench.

The spacing between the drains in each trench was 5 m, and between the trenches, 10 m. Additional drains were installed between the trenches at the shore line where the highest pore pressures had been measured.

On Fig. 5 are indicated the results of the pore pressure measurements made by two piezometers, one at the shore line, and the other at the crest of the slope. The curves show that the pore pressures fell considerably during installation of the sand drains. Below the pore pressure diagram is shown the actual and normal monthly rainfall. The variation in the pore pressures and rainfall show an appropriate correspondence. In the long dry summer period of 1959 the pressures were low, but late in 1959 the rainfall was much higher than normal, with correspondingly higher pore pressures. Nevertheless, the pressures were still lower than the original excess pressures.

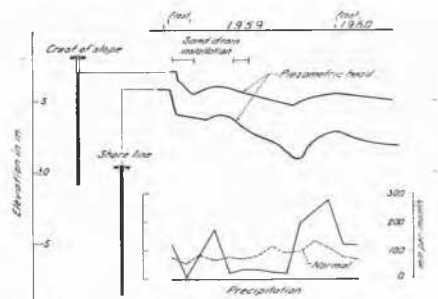


Fig. 5 Measured pore pressures and monthly rainfall. Pressions interstitielles mesurées, et précipitations mensuelles.

The curves indicate that there is a considerable variation in ground water flow.

The changes in the pore pressures during variable rainfall indicate that the spacing between the drains at the toe of the slope might have been smaller. The grain-size distribution of the filter material in the test drains might also have been used for the installed drains, as the test drains did not seem to be clogged.

Comparison between the readings of all the piezometers proves that a sufficient reduction in the excess pressures has been achieved thus giving a satisfactory stability against failure.

As an additional safeguard against failure, a counterfill of elevation 2.0 m was dumped in the sea along the shore line. The benefit of this meant a 10 per cent increase in the $C - \phi$ safety factor.

The author wishes to thank Mr. J. W. Kelly, Resident Engineer of Essoraffineriet, Norge, for permission to quote the results of this investigation which has been carried out for the company.

References

- [1] BISHOP, A. W. (1955). The use of the slip circle in the stability analysis of slopes. 5, 7-17.
- [2] BJERRUM, L. and KJÆRNSLI, B. (1957). Analysis of the stability of some Norwegian natural clay slopes. 7, 1-16.