CONTROLLED FAILURE OF AN INSTRUMENTED CUT SLOPE IN SOFT CLAY

RUPTURE CONTROLEE D'UNE COUPE INSTRUMENTEE DANS UNE ARGILE MOLLE

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SYNOPSIS: An area of soft clay was instrumented and subsequently a slope was constructed by step-wise excavation and by filling behind the crest of the excavation. The slope was finally brought to failure by pumping out water from the excavation and by applying a pressure behind the fill. The aim of the project was to study changes in stresses and strains in a slope close to failure and to test an automatic monitoring system. Continuous measurements of pore pressures, horizontal stresses, vertical and horizontal displacements and inclinations were made during the entire construction period and the failure stages.

The paper briefly describes the monitoring system. Results from measurements during the construction phase and the failure stages are presented and comparisons are made between results from stability calculations and observed behaviour of the slope.

INTRODUCTION

Every year, landslides occur in clay slopes in Sweden. During the last 40 years, one major landslide (>100,000 m^2) has taken place each decade, causing injuries to the inhabitants and great costs for the community (on average 10 - 20 million US$/year). At present, several thousand people in Sweden live and work in landslide risk areas.

To remedy all these areas and to bring the stability to a satisfactory level would be enormously expensive. In order to achieve higher safety in areas where the calculated safety factor is insufficient, the development of accurate and reliable remote monitoring and alarm systems has been considered necessary. A system of this kind, monitoring parameters such as pore pressure, vertical and horizontal movements and horizontal earth pressure, has been developed at the Swedish Geotechnical Institute (SGI), (Möller et al 1989). The main drawback of the system at present is the limited knowledge of what processes occur in the soil mass when failure is imminent. For this reason, the SGI has carried out a research project, sponsored by the Swedish Council for Building Research, in which a well-instrumented slope was gradually brought to failure.

The project may be considered as a full-scale pilot test aimed at obtaining a realistic idea of relevant magnitudes of stresses and strains, as well as trying out the monitoring system under long-term field conditions. The results have been reported in detail by Möller & Åhnberg (1992a).

TEST SITE

The project was carried out in one of the Institute's test areas with soft clay, located just outside the city of Norrköping. The layers of clay in the area are 14 - 16 m deep.

At the top, the soil consists of about 2 m of clay with remnants of vegetation and root threads. The upper metre has the character of a dry crust. A test pit showed that the upper 2 m had vertical cracks and fissures and was highly permeable. The soil below this top layer consists of grey clay on top of grey varved clay down to about 14 m depth. From a depth of about 7 m, the soil contains thin silt layers, which become thicker with depth. Fig.1 shows the water content, density and shear strength of the soil.

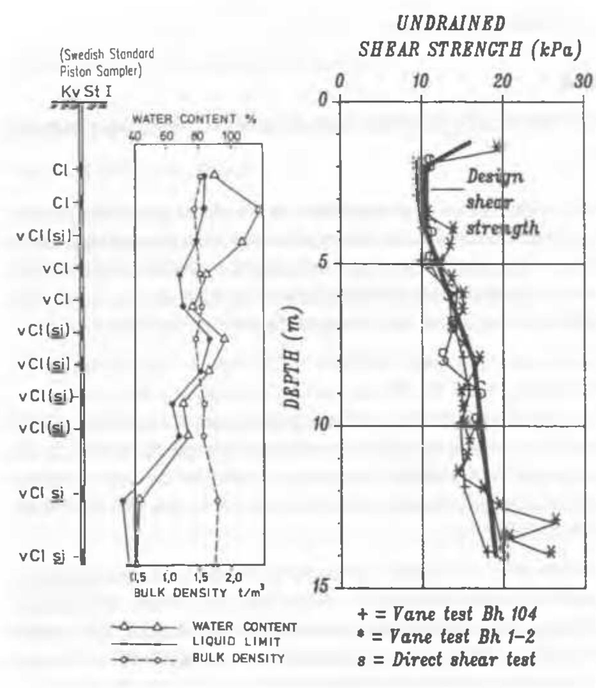

Fig. 1. Results from soil investigation. a) Density and water content. b) shear strength.

The results from oedometer tests indicate that the clay is slightly overconsolidated by about 10 kPa down to 8 m depth. Below this level, the overconsolidation ratio increases from about 1.3 to 1.5. Below the clay, there is a transition layer of loose friction material on top of till or bedrock.

CONSTRUCTION AND STABILITY

Construction Stages

The 40 m wide slope was constructed from the level ground surface, using excavation by steps and filling of excavated masses behind the crest. In this way, a gradually increasing shear stress level was achieved.

At the end, the excavation reached 4 m below the original ground level and the fill had a height of 1.6 m (20 kPa) above this level. The natural ground water level was about 1.25 m below the original ground level and the water in the excavation was kept at this level during the different construction stages. The excavation depth and the duration of the construction stages are shown in Fig. 2.

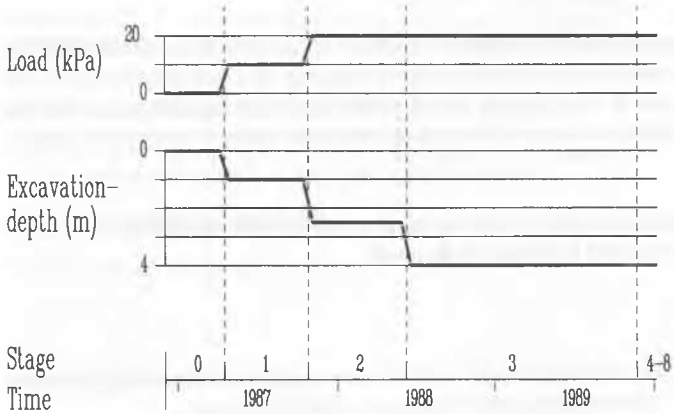

Fig. 2. Construction stages.

Stability

The calculated safety factor had been lowered to 1.2 in the last construction stage.

Stability calculations were made for the various stages in the project and were reexamined at times as more information on the soil behaviour was obtained. The results were used to estimate probable safety factors and shear surfaces and, prior to the failure test, also to localize a trench through the fill at the estimated back-scarp of the critical slip surface.

The analyses comprised estimates of increase in shear strength due to consolidation under the fill and, since the slope was fairly steep and the inclination of the shear surfaces for large parts corresponded to that of active shear, also the anisotropy of the shear strength was taken into consideration. The conventional stability analyses, as well as the use of advanced numerical methods, have been more extensively described by Möller & Åhnberg (1992b).

As a result of the calculations, it was decided that a 1.6 m deep trench should be dug through the fill about 5 m behind the crest where, according to the calculations, the most critical shear surface was located. The calculated safety factor, using 3-dimensional analyses, was then 0.98 - 1.06, depending on the assumptions made for the shear strength in the crust.

Almost three years after the start of the project, the slope was gradually brought to failure. This was achieved by using pumps to lower the water level in the excavation by steps and finally water filling the open trench in the fill. The variation in the calculated safety factor with the water level in the excavation is shown in Fig. 3. In the diagram, the calculated effect of water filling the trench, using different assumptions on the resulting water pressures in the crust, is also shown.

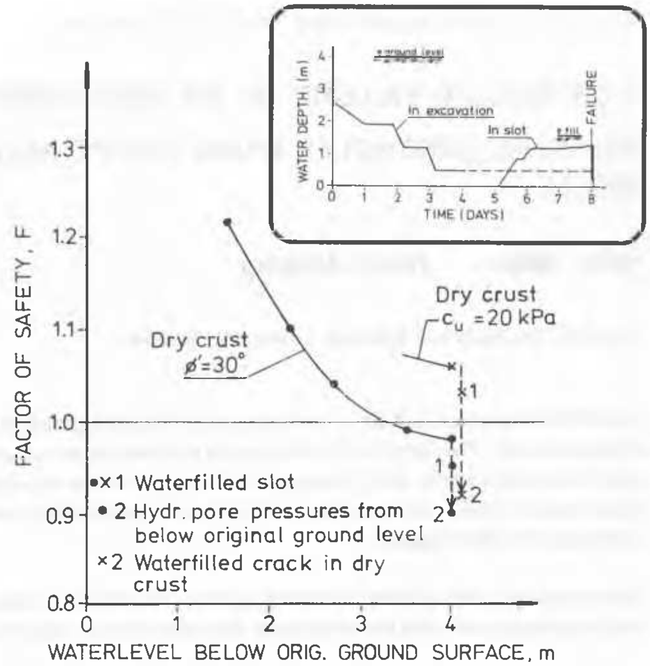

Fig. 3. Variation of safety factor with water level in excavation.

MONITORING SYSTEM

One of the causes of landslides is changes in the action loads, both in the short term and long term. This means that if it is possible to measure changes in reactions in the soil to external or internal loads, it should also be possible to acquire some indication of the risk of landslides. Such a monitoring system has been developed at the SGI during the last decade.

The remote data acquisition system consists of three main parts:

1. On-site control computer
2. Remote measuring unit
3. Transducers

The on-site computer is an ordinary PC able to communicate and be controlled by a telephone and modem. This allows the operator to remotely control and run the monitoring programme.

The remote measuring units are devices which convert the electric signals from the transducers to digital units and send them to the control computer. Each unit can handle 16 channels with 12-bit resolution. The units convert 0-20 mA analogue signals to digital units between 0-4000.

Some transducers used in the project were developed at the SGI, while others were well-known commercial transducers suited to the specific type of measurement. A total of more than 60 electronic instruments were used in the project for automatic measurement of the following parameters:

- settlement
- lateral movement
- inclination
- earth pressure
- pore pressure

A large number of conventional, manually read instruments were also used for measurement of the above parameters.

The measurements were concentrated to the mid-section of the slope. The instrumentation in this section is shown in Fig. 4.

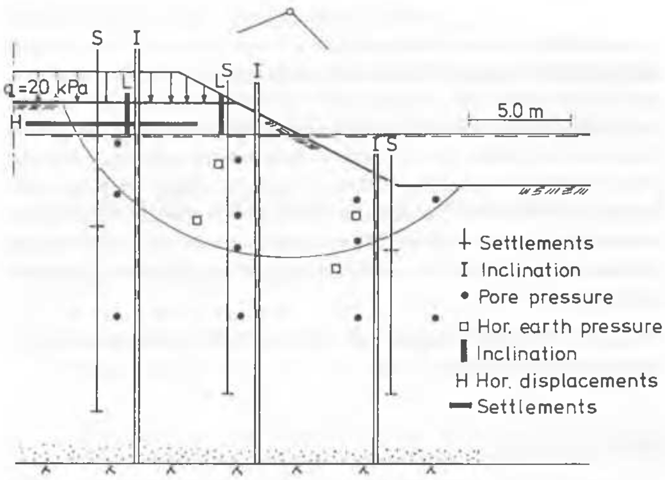

Fig. 4. Section of the slope with the principal instrumentation.

MEASUREMENTS PRIOR TO FAILURE

Stage 0. Before Construction

During the first stage, the natural soil was heavily instrumented with transducers for automatic recording and with equipment for parallel manual measurement of stresses and deformations.

When all the instrumentation had been installed, calibration and reference measurements were performed during a period of more than 4 months. This was done to verify the operation of the measuring system and to obtain an overall picture of the natural variations in measured parameters in order to separate these variations from effects of increasing load during later stages. The measuring interval for the automatic system was mainly 1 hour and the results were displayed as mean 24-hour values.

The results from this initial reference period did not cause any greater changes in the monitoring system. However, one valuable observation was that the atmospheric pressure caused significant variation in recorded pore pressures and horizontal earth pressures. This resulted in changes in the programme so that values corrected for barometric pressures were recorded and displayed. On the other hand, rapid changes lasting less than a day did not seem to influence the measurements to the same extent as did longer periods of atmospheric pressure changes. At times, an overcorrection therefore resulted in a small flutter in the readings, which had to be disregarded. Fig. 5 shows examples of uncorrected and corrected measurements of pore pressures, together with barometric pressure readings.

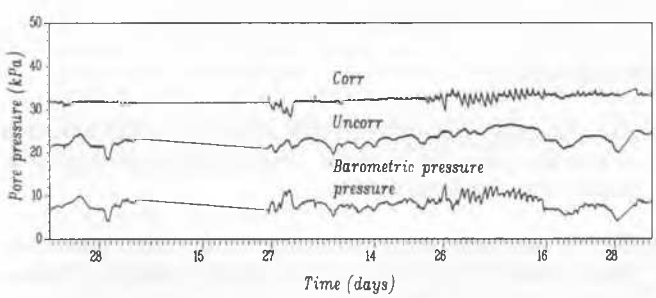

Fig. 5. Influence of atmospheric pressure on pore pressure readings.

Stage 1 - 3. Excavated Slope 1-4 m Below Ground Level

The excavation and filling was performed in three stages that together lasted for more than 2.5 years. The measuring interval for the automatic system was the same as during the preceding reference period, i.e. one hour. Manual measurements were made 4 to 5 times during each of the three stages.

The operation of the system and interpretation of the measurements during these stages demanded careful inspection of the results in order to estimate whether the recorded values were realistic or not. Smaller adjustments, as well as reinstallation of instruments and changes in computer programmes, were made.

During the excavation and loading process, the measurements did not show any changes in the slope that could be interpreted as indications of reduced stability of the slope. The deformations and changes in stresses could be considered related to consolidation processes due to loading/unloading. Long-term readings of pore pressure under the fill are shown in Fig. 6.

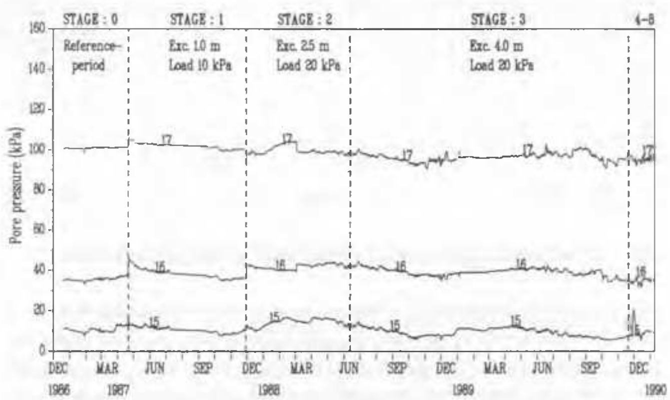

Fig.6. Recorded pore pressures under the fill.

THE FAILURE TEST

Development Of Slope Failure

Pumping out of the water in the excavation started in late November, 1989. The water level was lowered in three steps, with the level kept constant for about 24 hours after each step. In the next stage, water was pumped into the trench in the fill. The water level in the trench was kept constant or very slowly rising at about 0.3 m under the top of the fill for 3 days until failure occurred eight days after the start of the failure test.

The failure did not develop quite as predicted. Because of the comprehensive instrumentation in the middle section, it had not been possible to excavate this part completely, and the section had assumed the shape of a small and slightly reinforced beam at the bottom of the excavation. As a result of this, two failures developed, one on each side of the middle section.

Measurements

Monitoring of the slope continued with an increased frequency of the readings during the failure test. The measuring interval was about 10 minutes, decreasing to 1 minute during the last hours before failure.

Many interesting measurements were obtained when the slope was finally forced to failure. Fig. 7 shows the measurements from two extensiometers during the last stages constituting the failure test. The extensiometer rods were 8 m and 12 m long respectively and were anchored beneath the crest of the slope.

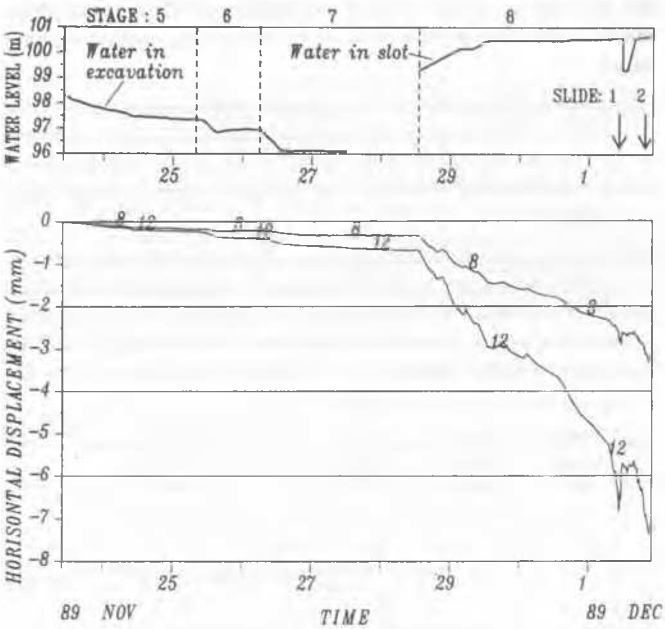

Fig. 7. Horizontal displacements versus time during the failure test.

During the pumping stage (5-7) there was only very small displacement, at most 0.7 mm for the 12 m long extensiometer, which could be related to elastic deformation of the soil. In the next stage, when water was pumped into the trench behind the crest of the slope, the rate of displacement increased significantly. Afterwards, the time for the first significant change in displacement rate could be interpreted as an early indication of imminent failure. The day before the actual failure took place, there was a further increase in the rate of displacement.

Changes in pore water pressure during the failure were caused both by changes in the external load situation and changes in shear deformations. Another factor which affected the pore water pressure was the climate situation during the failure test. During this period, the outdoor temperature varied between +1 and - 14 °C and the barometric pressure by 5.5 kPa.

Fig. 8 shows the results from station 10, located close to the crest in the middle section of the slope. Here, transducers were placed at 2.8, 5.5, 7.2 and 10.5 m below the original ground surface.

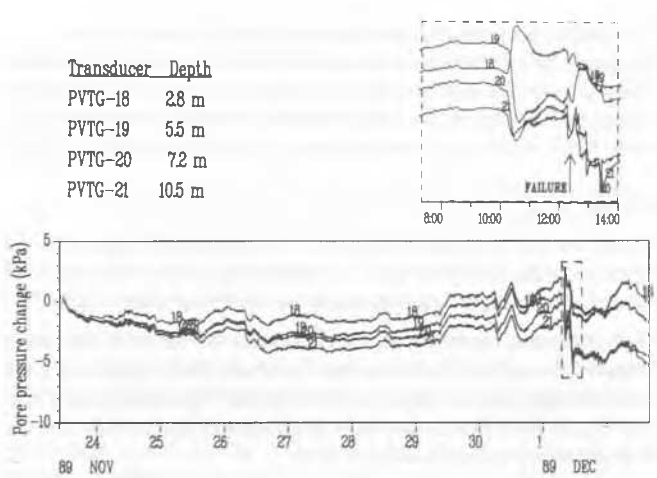

Fig. 8. Pore water pressure versus time during the failure test.

During the end of stage 6 and beginning of stage 7, there was an extreme low barometric pressure which affected the results, in spite of a correction being made. In stages 5-7, when the water level in the excavation was gradually lowered, there was a decrease in the pore water pressure directly proportional to the decrease in water level in the excavation. When the trench was filled with water in stage 8, there was an increase in the pore water pressure. The last day before the slide occurred, the pore water pressure was unstable. On a magnified scale, sudden changes in pore water pressure can be seen two hours before the slide. At one level, there was an increase and for the rest of the piezometers there was a decrease in the pore water pressures.

Small but measurable changes thus developed both in deformations and pressures in the failure test.

CONCLUSIONS

The measurements provided the following information, among other things:

- The system for automatic measuring of various parameters in slopes functioned very well and valuable information was obtained on natural variation in the soil.
- Instruments placed in the passive zone of the slope gave earlier and clearer indications of forthcoming failure.
- Changes in velocity of horizontal movement gave a warning that a slide was imminent. A significant change occurred about 3 days before the slide.
- Changes in pore water pressure and horizontal earth pressure in connection with slide initiation were diffuse and difficult to interpret. However, the measurements show that two hours before the slide, stress changes occurred on a level that could possibly be interpreted as a plasticization of the soil.
- Pore pressure transducers in the active zone of the soil became "unstable" when the velocity of movements changed three days before the slide.
- The measurements also show that all transducers became "unstable" at least two hours before the first slide.

The project also shows that uncertainties in the measuring results are to a large extent due to climatic influence.

The measurements did not show any significant changes in the measured parameters during the gradual decrease of the safety factor down to 1.2. The changes in stresses and strains that were recorded were mainly proportional to the changes in external loads.

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