The landslide at Tuve

by BENGT-ARNE TORSTENSSON*, DSc

On November 30, 1977, between 4.05 and 4.09 pm, a huge landslide occurred in Tuve, located in the vicinity of the city of Gothenburg, Sweden. The landslide covered an area of 27 hectares. A total of 65 homes was affected by the sliding soil masses. Some houses moved as much as 180m. The landslide claimed nine lives.

This Paper summarises the results of geotechnical investigations made in the slide area. Methods that were tested for a quick stabilisation of the slide masses are described and analysed. Some works carried out for stabilising and rebuilding of the area are briefly described.

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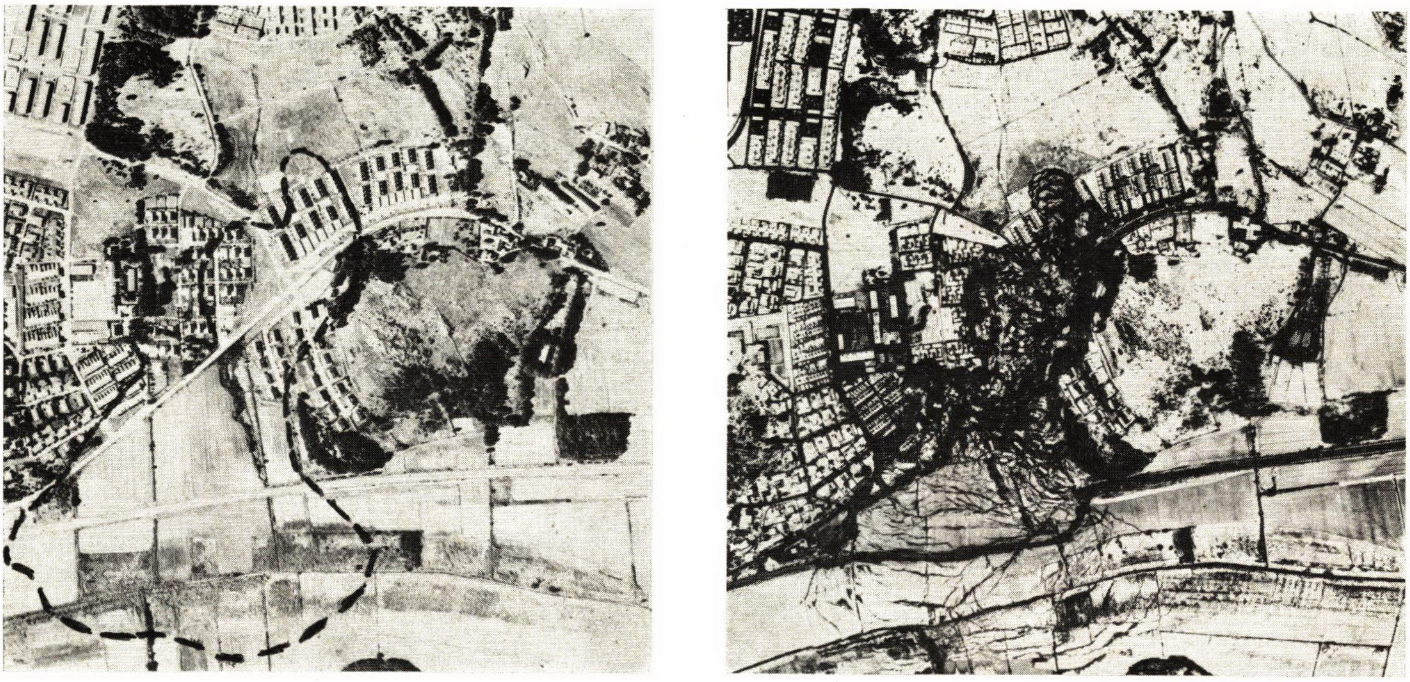
The landslide — some facts

The landslide at Tuve occurred on November 30, 1977, its duration being four minutes, between 16:05 and 16:09. The exact time is based on automatic registration when some electrical cables were cut. Eyewitness reports state that the slide started in the intersection between Tuve Kyrkväg and the small stream which ran through the area. After this initial failure the slide rapidly spread towards the sides and backwards. The total area of the landslide is about 27ha (Fig. 1).

In the upper "active" zone of the slide, the original ground surface sank a maximum of 10m. In the lower "passive" zone, the original soil was pressed upwards due to the energy of motion of the sliding soil masses. In the "passive" zone the ground heaved a maximum of 5m.



0 250 500m

Fig. 1. The landslide in Tuve covered an area of 27 ha. In the upper "active" zone of the slide, the original ground surface sank a maximum of 10m. In the lower "passive" zone, the original soil was pressed upwards a maximum of 5m due to the energy of motion of the sliding soil masses. The volume of soil involved in the slide can be estimated at 3-4 million m³

Fig. 2. Heroic rescue work was carried out by firemen working the whole night in the treacherous and slippery clay masses.
(photo, "Pressars Bild")

Fig. 3. Aerial view of the slide area

The volume of soil masses involved in the slide can be estimated at 3 to 4 million m³.

The landslide occurred in the afternoon when it was just getting dark and as a result the rescue work was difficult and dangerous (Fig. 2). On the following morning, the daylight exposed the extensive nature of the disaster in Tuve (Figs. 3 and 4) — 65 homes were carried away by the sliding clay masses, several houses had moved 100m to 180m, and close to the edge of the slide, about 30 houses sustained damage of differing degree.

The total costs of the damage caused by the landslide have been estimated to be about 140 million Swedish crowns. Nine persons were killed in the slide.

Development of the landslide

From eyewitness reports it has been possible to reconstruct a relatively accurate picture of the development of the landslide in Tuve (U. Fält, 1978). An initial fissure was observed in Tuve Kyrkväg where it crosses the small stream which ran through the area (see Fig. 5) and which was culverted west of Tuve Kyrkväg. From a topographical point of view, it is quite clear that the initial slide occurred in the weakest part of the area.

In general terms, the main cause of the landslide is likely to be found in the gradual urbanisation of the area. This process has involved both an increased loading of the weak clay masses and a disturbance of the natural ground water balance. These effects have tended to reduce the stability of the clay slopes. Vibrations from increasing heavy traffic along the Tuve Kyrkväg may also be a factor that has gradually reduced the strength of the sensitive clay. Yet another triggering factor may have been the extremely heavy rainfall during a prolonged period before the landslide.

Evacuation zone

Immediately after the slide there was naturally considerable uncertainty about the stability of the houses remaining close to the edge of the slide. During the first week, effort was concentrated on the determination of an evacuation zone and acute risk area. Stability calculations were made on the basis of undrained shear strength values from field vane tests, and the topography of the firm bottom was determined by numerous soundings.

The calculations showed that the factor of safety was close to unity for slip-circles in the clay near the edge of the landslide — a result that naturally was not surprising. The investigations also showed that the irregular shape of the landslide was reflected in large measure by the topography of the firm bottom.

The geotechnical investigations and calculations carried out during the first week after the landslide determined an evacuation zone that included a total number of 120 homes (see Fig. 6).

Various temporary measures were taken to save some of the buildings that remained on the edge of the slide (Fig. 7).

Geotechnical investigations in the slide masses

Thirteen days after the landslide, geotechnical investigations were started in the clay masses involved in the landslide. The aim of these investigations was both to obtain a basis for the design of temporary support fills etc. and to study in detail the condition of the slide masses.

The thickness variations of the clay within the slide area were determined by some 500 soundings. The depth to firm bottom increases gradually eastwards. To the north-west of the slide area, the thickness of the remaining clay varies from about 2-8m. In the central part of the slide area, the depth to firm bottom is about 20m. Within the passive zone of the landslide, the clay reaches a thickness of about 40m (cf. Fig. 8).

In order to study the state of stress in the more or less remoulded clay masses,

Fig. 4. The total damage costs for the landslide are estimated to be about 140 million Swedish crowns

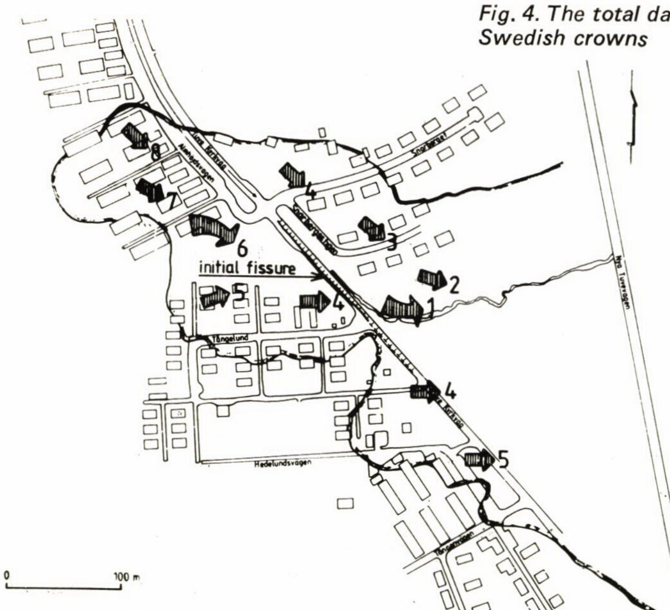


Fig. 5. Development of the landslide in Tuve. An initial fissure was observed in Tuve Kyrkväg where it crossed the small stream which ran through the area. After this initial failure the slide rapidly spread towards the sides and backwards

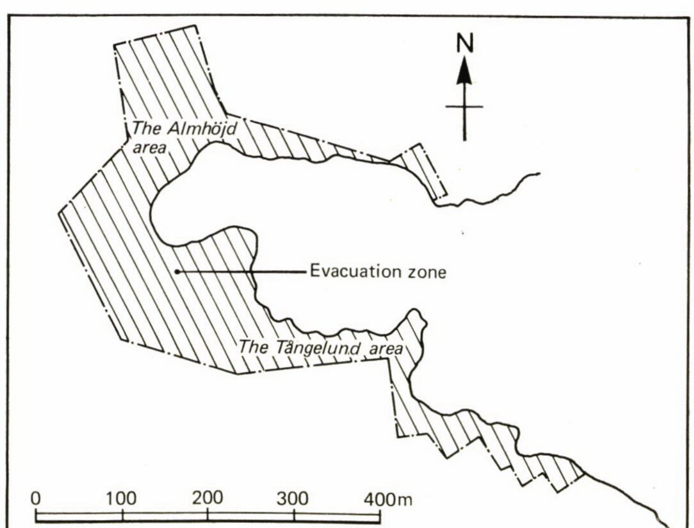


Fig. 6. 120 homes were included in the evacuation zone

Fig. 7. Temporary struts supporting a building which is hanging over the edge of the slide

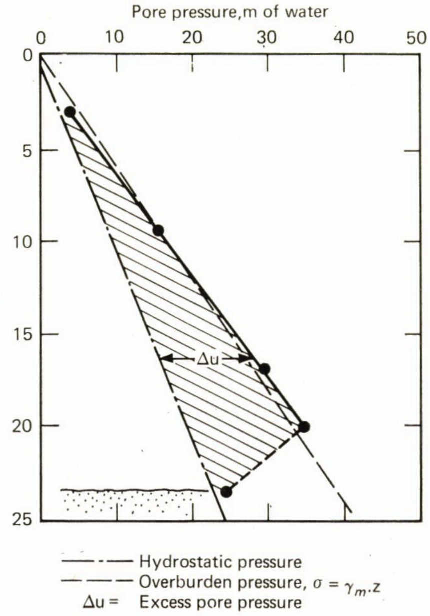


Fig. 9. Results of pore pressure measurements in a soil profile, located in the central part of the slide area. At a depth of 20m the excess pore pressure amounts to 13m of water column which corresponds to an effective stress equal to zero

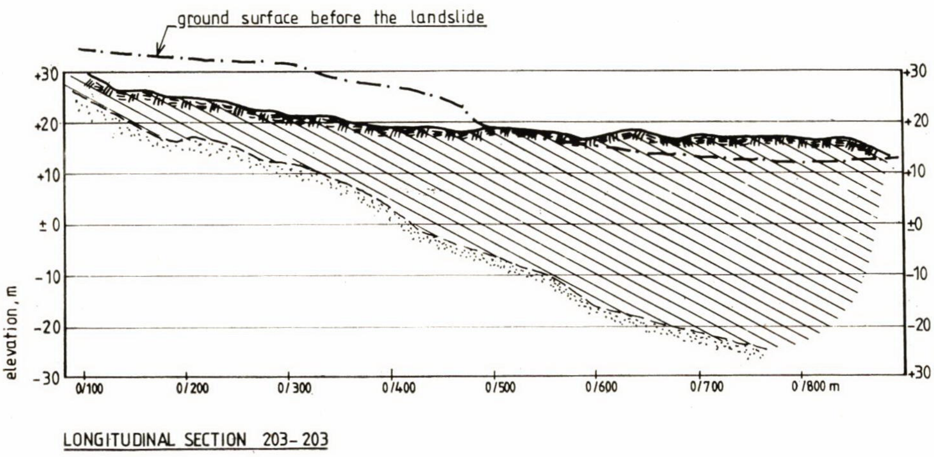


Fig. 8. Longitudinal section through the landslide. The thickness of the clay layer increases gradually towards the passive zone of the landslide. In this zone the depth to firm bottom is about 40m

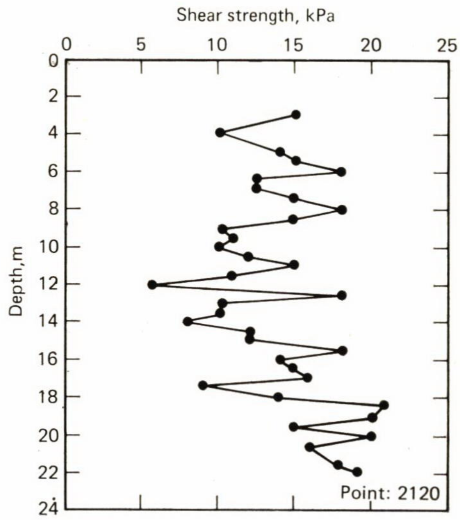


Fig. 10. Results from field vane tests in the "active" zone of the landslide. The erratic variation of the shear strength values indicate the presence of several zones of remoulded clay in the soil profile

several BAT type piezometers were installed. The pore pressure measurements showed that due to the slide movement, extremely high excess pore pressures had developed in the soil. In fact, in the active zone of the landslide the pore pressures were to a great depth about equal to the total overburden pressure. For example, at a depth of 20m the excess pore pressure amounted to 13m of water column (Fig. 9). The very high excess pore pressures can no doubt be explained by the fact that the clay had been so severely remoulded that it behaved in large measure as a heavy liquid.

Several vane tests were carried out in order to study the shear strength variation of the slide masses. The results of these tests mostly showed an erratic variation of the shear strength values within the soil profile. It could be concluded that in the "active" part of the landslide, zones of weakness (i.e. zones with more or less remoulded clay) were present within the whole depth of the slide masses (Fig. 10).

Special field vane tests were carried out to study the influence of strain rate on the shear strength values. In these tests the testing rate was only about one thousandth of the standard testing rate. The time needed for mobilisation of maximum shear stress was of the order of 17 hours. For example, at point 2016, where the clay had only been moderately disturbed by the landslide, the results showed that the strain rate had a pronounced influence on the mobilised shear strength (see Fig. 11*a*). At point 2034, however, where the

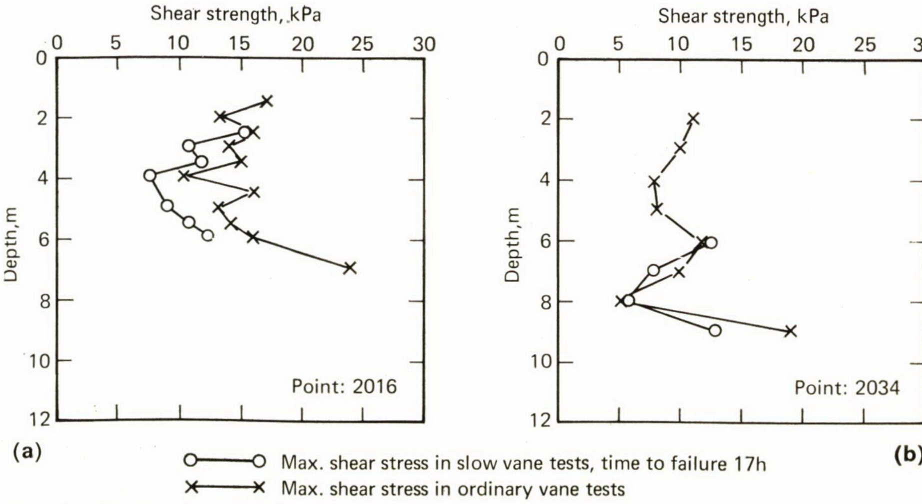


Fig. 11. Results of special field vane tests which were carried out in order to study the influence of strain rate on the shear strength values

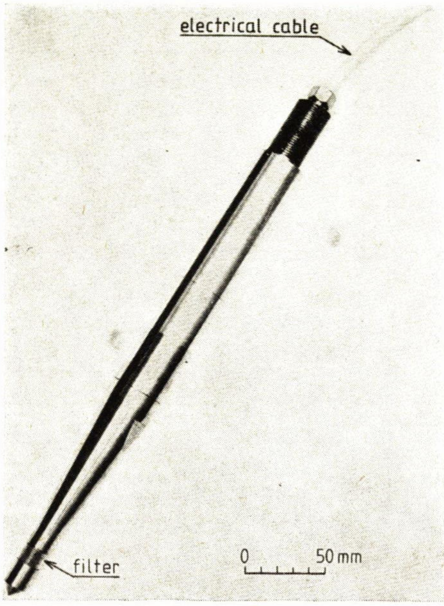


Fig. 12. The BAT pore pressure probe consists of a conical piezometer tip furnished with a small cylindrical filter. Pore pressure is measured with the aid of a flush-diaphragm pressure transducer

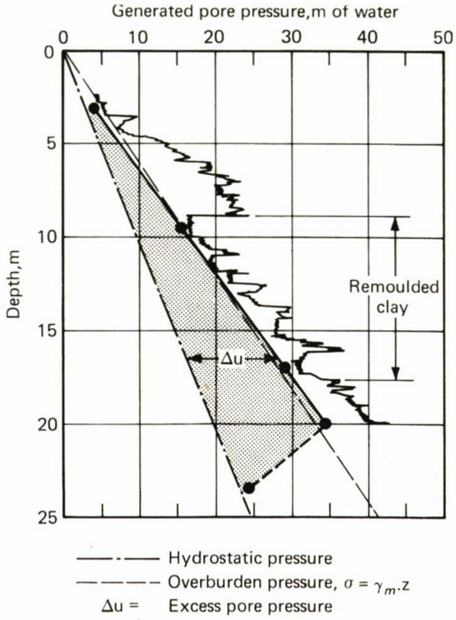


Fig. 13. Pore pressure sounding diagram for a soil profile in the "active" zone of the slide area. At depths greater than 10m the results of the pore pressure sounding indicated the presence of several zones of weakness in the slide masses; i.e. in these zones the generated pore pressures were at a minimum equal to the static pore pressure

clay was severely disturbed, the results showed that the influence of strain rate on the shear strength had almost vanished, (cf. Fig. 11*b*).

By using the BAT pore pressure probe seen in Fig. 12, it proved possible to obtain continuously qualitative information on the variation of the shear strength in the slide masses. The system utilises the pore pressures that are generated when the probe penetrates the soil at a constant speed. The excess pore pressures measured during penetration provide continuous information both of the soil type and of the shear strength of soft clay.

When making pore pressure soundings in the slide masses, the presence of zones of remoulded clay are registered on the sounding diagram as sudden pressure drops. From the pore pressure sounding diagram shown in Fig. 13, it can be seen that within the depth interval of 4-9m there is a block of relatively undisturbed clay (i.e. high excess pore pressures are generated in this zone). At greater depths the results of the pore pressure sounding show that the slide masses contain several zones of weakness. In fact, the generated pore pressures are at a minimum equal to the static pore pressure. This indicates that the shear strength of the slide masses is locally equal to zero. The presence of weakness zones means that the slide masses may have a very low shear strength irrespective of the depth below ground surface.

Based on the different investigations, the average shear strength of the slide masses was judged to be equal to 7kPa. Stability calculations and the design of temporary support fills etc. were made on the basis of an allowable shear stress equal to 5.5kPa.

One method of stabilising parts of the slide area would be to eliminate the very high excess pore pressures in order to make possible a rapid reconsolidation and healing of the remoulded clay masses.

Laboratory testing

To study this healing effect (i.e. increase in shear strength) which could be achieved by a reconsolidation of the clay masses, a series of special oedometer tests were carried out.

The tests were made on samples having an initial shear strength of 5-8kPa. Before testing, all samples were given a well-defined degree of distortion by pressing them through a conical piece of pipe, resulting in an area reduction of the sample of 35% (cf. Fig. 14). In cases where the samples were already severely disturbed in-situ, this treatment did not result in any further reduction of the shear strength. Altogether 34 samples were consolidated to different effective stress levels varying from 20-160kPa. The subsequent shear strength measurements showed that the ratio $\Delta\tau/\Delta\sigma'$

in which $\Delta\tau$ = increase in shear strength

$\Delta\sigma'$ = increase in effective stress

decreases with an increasing stress level. Within the stress interval of 45-160kPa, the ratio $\Delta\tau/\Delta\sigma'$ is on average equal to 0.20. This value is in good agreement with the empirical relation given by Hansbo (1957)

$$\Delta\tau/\Delta\sigma' = 0.45\ w_F$$

in which w_F = fineness number ($\approx$ liquid limit).

The settlements caused by a reconsolidation of the remoulded clay masses in the active zone have been estimated on the basis of the special oedometer tests described above. Fig. 15 summarises the results of the settlement calculations. Depending on the thickness of the clay masses, the final settlement due to reconsolidation of the clay is estimated to vary from about 0.1m (5m clay layer) to about 0.7m (25m clay layer). The time needed for a reconsolidation of the clay may be estimated assuming a value of the coefficient of consolidation c_v of 5×10^{-8} m^2/s. At an average degree of consolidation $\overline{U}$ equal to 80%, the following consolidation times can be calculated:

thickness of clay layer, m	10m	15m	20m
time of consolidation ($\overline{U}$ = 80%).	10 years	20 years	30 years

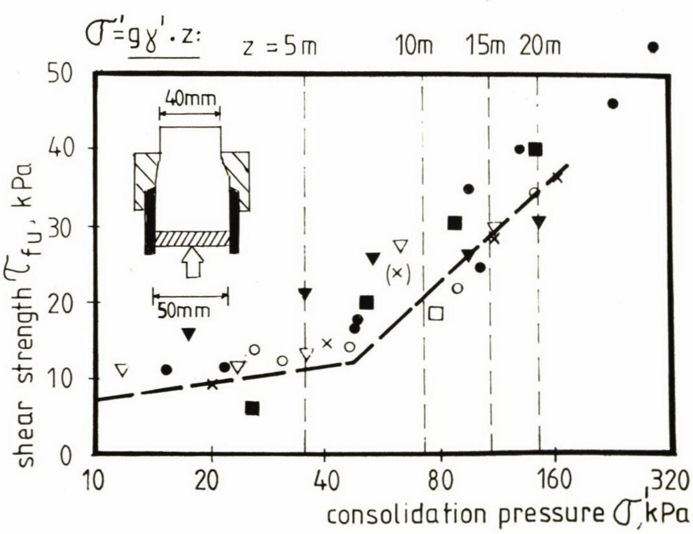


Fig. 14. Undrained shear strength values, consolidation pressure — results from tests on 34 reconsolidated samples from the 3-18m depth intervals

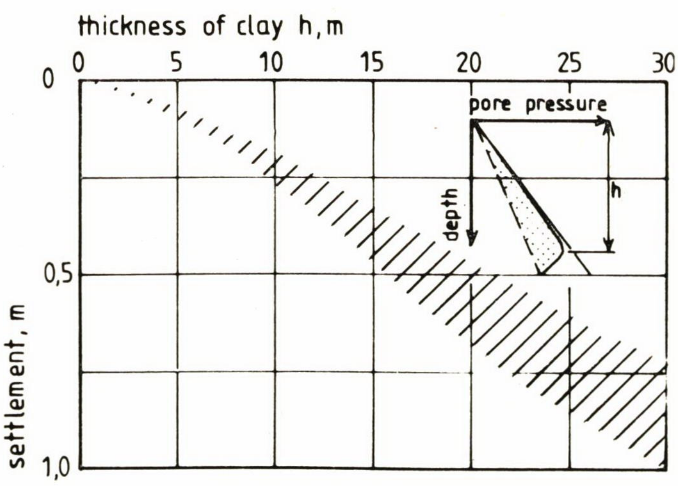


Fig. 15. Calculated settlements due to a reconsolidation of the remoulded clay

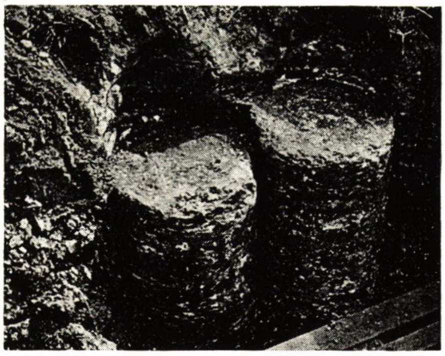


Fig. 16. Lime columns with a diameter of 0.5m

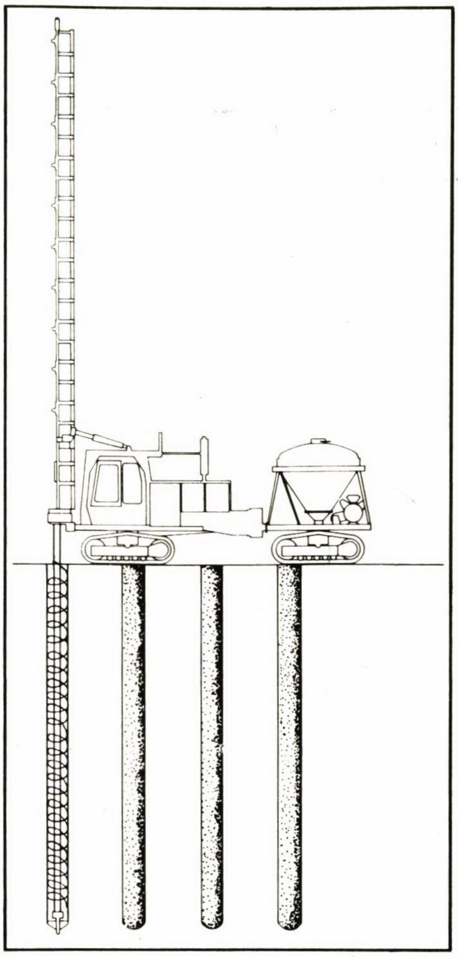


Fig. 17. Lime columns are manufactured in-situ by mixing unslaked lime and clay. The LPS equipment consists of a mixing tool mounted on a special carrier, the unslaked lime being stored in a tank mounted on a trailer

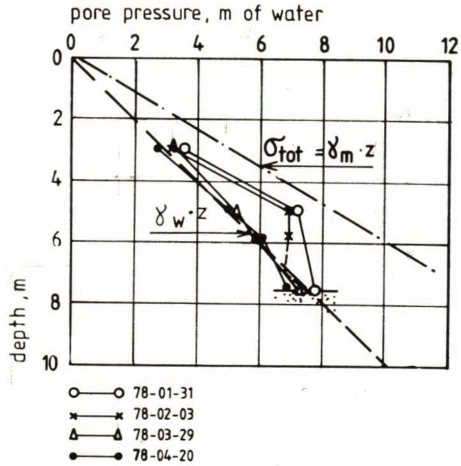


Fig. 18. Results of pore pressure measurements in the lime columns test area

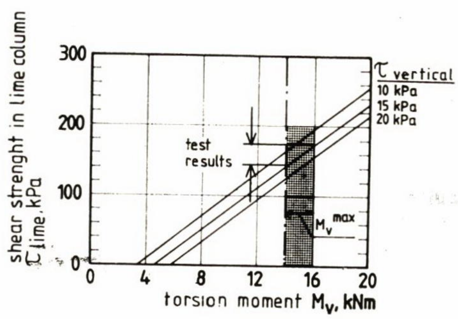


Fig. 19. Results of shear strength tests on three-months-old lime columns. The tests were made with the aid of a giant vane tester having a diameter of 0.5m and a height of 0.5m

Testing methods for stabilisation of the slide masses

Lime columns

One possible solution to a quick stabilisation of the slide masses would be the use of the so-called lime column. The lime columns (Fig. 16) are manufactured in-situ by mixing unslaked lime and clay. The LPS equipment, developed and marketed by Linden-Alimak, consists of a mixing tool mounted on a special carrier, the unslaked lime being kept in a storage tank on a trailer towed behind the carrier (see Fig. 17).

The mixing tool is screwed down into the ground at a maximum speed of 0.5m/s. When the mixing tool is retracted, the rotation is reversed and unslaked lime is then forced into the soil by compressed air. In this way lime columns with a diameter of 0.5m are formed. About 300m of lime columns can be installed in eight hours.

The strength of the lime column will gradually increase as a result of chemical reactions. Because of pozzolanic reactions, this process will continue over a long period.

About a month after the landslide, a test field was set up in order to investigate the applicability of the lime column method in Tuve. The columns were installed with a spacing of 1.4m and the clay was instrumented with piezometers to study their drainage effect. Due to the tight time schedule the piezometers were unfortunately not installed until 20 days after the installation of lime columns. However, the results from the pore pressure measurements, which are summarised in Fig. 18, clearly demonstrate that the lime column effectively functions as a vertical drain. Thus, 3.5 months after the installation of lime columns, the measured pore pressures corresponded practically to a hydrostatic condition; i.e. the initial excess pore pressures had been completely eliminated. The combined function of the lime column as both a reinforcing element and a vertical drain is naturally a desirable feature.

The shear resistance of the lime columns was measured with the aid of a specially designed giant vane tester that had a diameter of 0.5m and a height of 0.5m. With this tool it was possible to measure at different depths the horizontal shear resistance of the lime columns, and tests were carried out 100 days after their installation. Fig. 19 summarises the re-

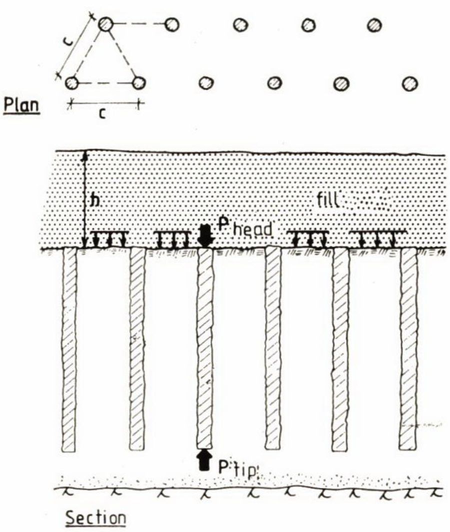


Fig. 20. Load distribution within a soil block reinforced with lime columns

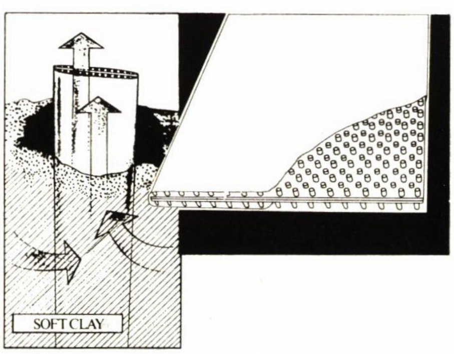


Fig. 21 (above). The Alidrain consists of a studded plastic core which is covered with a synthetic filter. (Right). The Alidrain test area. The drains were installed with a spacing of 1.2m

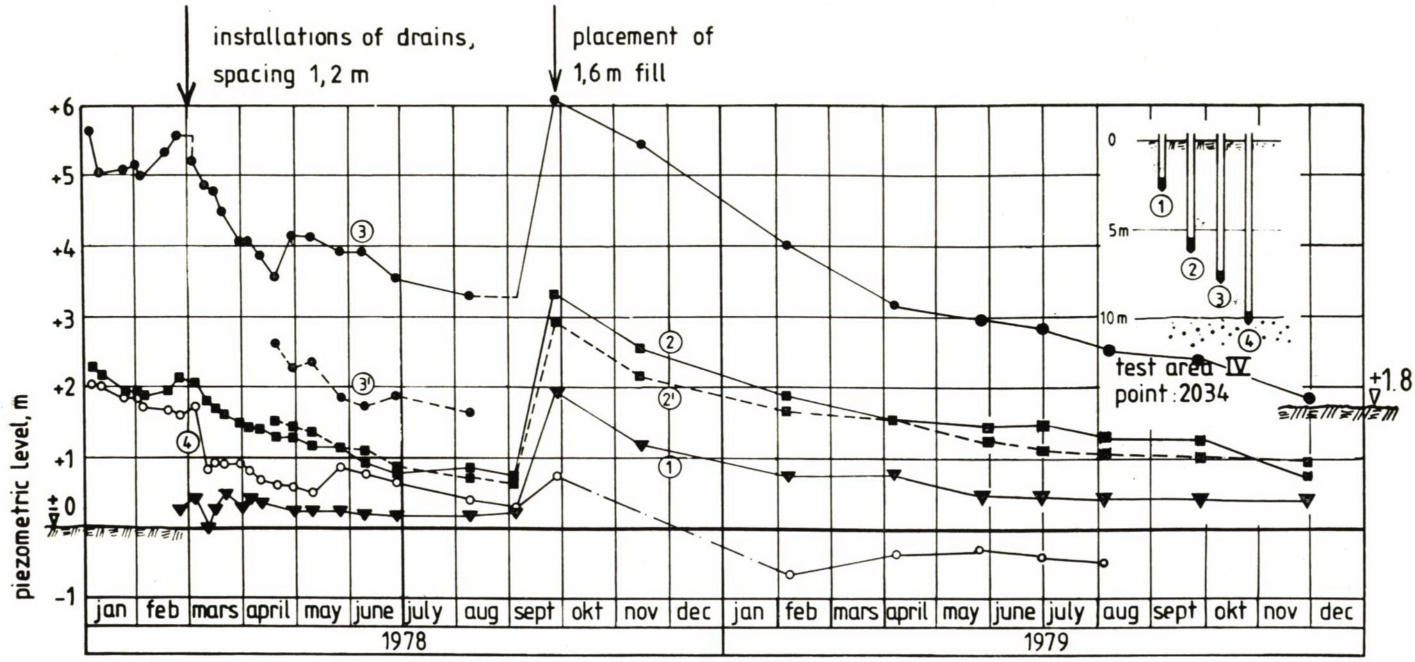


Fig. 22. Results of pore pressure measurements at different depths in the Alidrain test area

sults from 12 tests made in three columns and at different depths. The vane tests gave consistent values of the shear resistance with an average value of 150kPa. Considering the creep behaviour of the lime column material, a design value for the shear resistance of 100kPa for three-month-old columns was recommended.

When designing a lime column installation, the load distribution between the lime column and the surrounding soil must be taken into account, cf. Fig. 20. The load carried by the lime column is a function of both the strength and the spacing of the columns and the magnitude of loading on the soil surface.

Vertical drains

Four test areas were established to study the efficiency of vertical drains for a quick stabilisation of the slide masses. In one test area, Alidrain was installed with a spacing of 1.2m (Fig. 21).

The instruments used for checking the rate of consolidation included piezometers, installed at different depths, and bellows hoses for studying the distribution of settlement within the clay layer. In addition, vane tests were carried out at different times after the installation of the drains in order to study the gradual regain in shear strength of the remoulded clay.

The results of the pore pressure measurements showed that the excess pressures immediately started to dissipate significantly after the installation of vertical drains, (see Fig. 22). This effect was especially noticeable within the lower part of the clay layer. For example, in test area IV (Alidrains at a spacing of 1.2m) the average degree of consolidation after six months amounted to 60-70%.

The results of the bellows settlement meter (Fig. 23) show that in test area IV about 65% of the total compression had occurred within the lower one-third of the clay layer. This part of the soil profile had also suffered the highest degree of disturbance during the landslide.

The vane tests carried out at different time intervals showed that the consolidation of the clay had resulted in a significant increase in the undrained shear strength (see Fig. 24). This effect was especially noticeable in the zones of weakness in the lower part of the clay layer. Thus, in this zone the lowest shear strength value after seven months of consolidation was 15-20kPa, compared with an initial value of 5kPa.

The conclusion of the tests was that vertical drains would be an effective means of quick stabilisation of the slide masses.

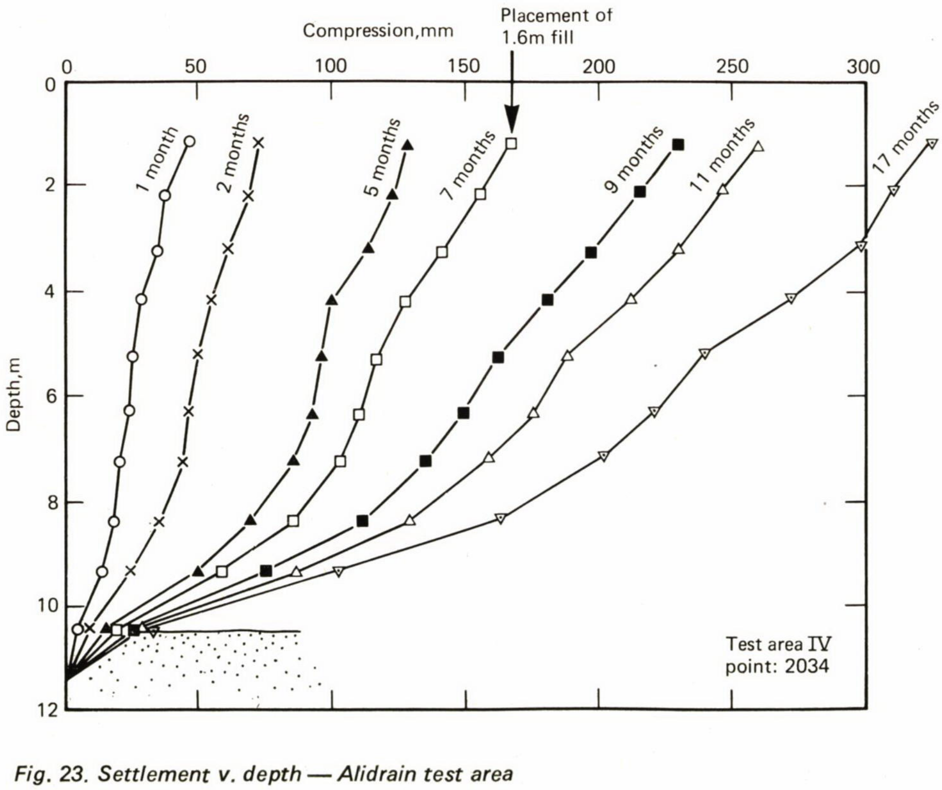


Fig. 23. Settlement v. depth — Alidrain test area

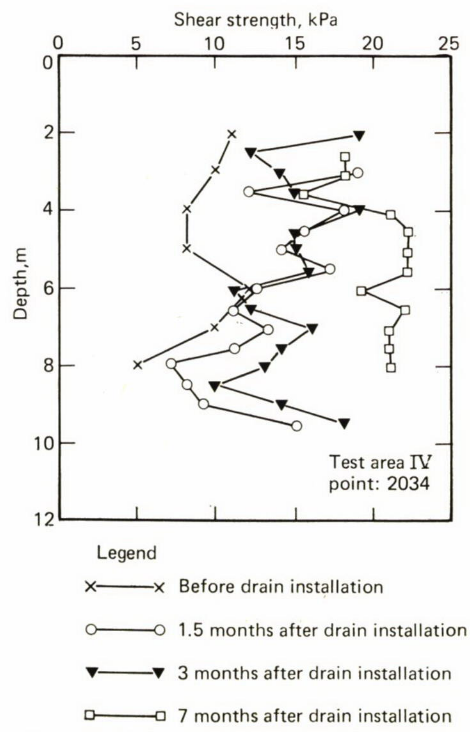
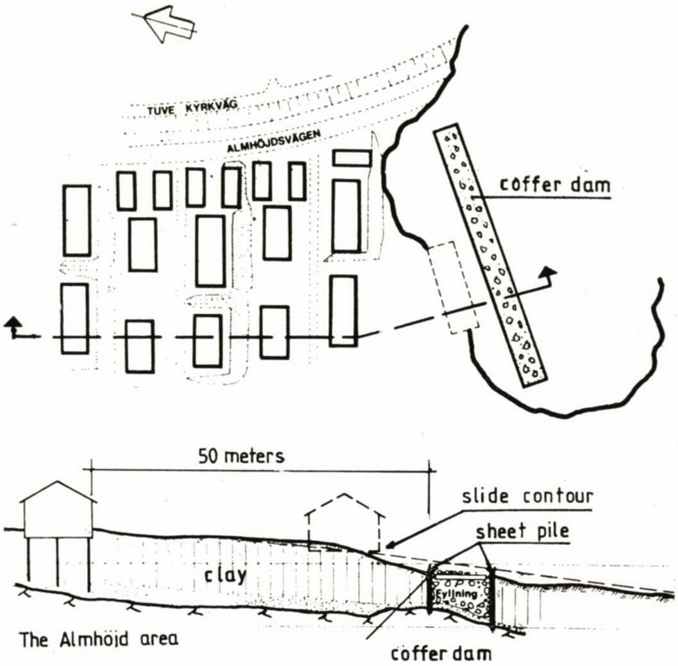


Fig. 24. Undrained shear strength v. depth Alidrain test area



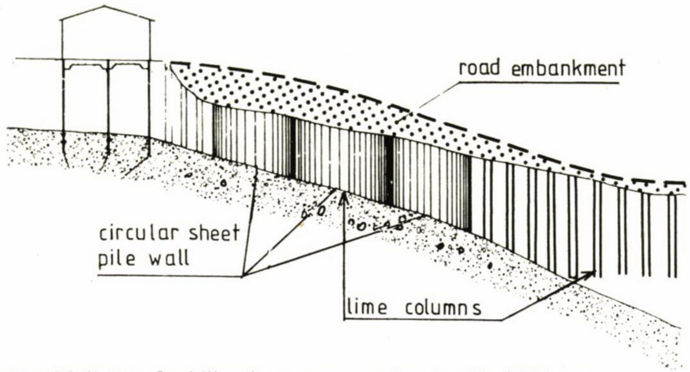


Fig. 25 (left). Stabilisation measures for the Almhöjd area
(from P-O Sahlström, 1978)

Fig. 26 (above). Construction of a road embankment using a combination of lime columns and circular sheet pile walls.
(from P-O Sahlström, 1978)

Fig. 27. The circular sheet pile walls were primarily constructed to prevent local stability problems in the clay during placement of the road embankment

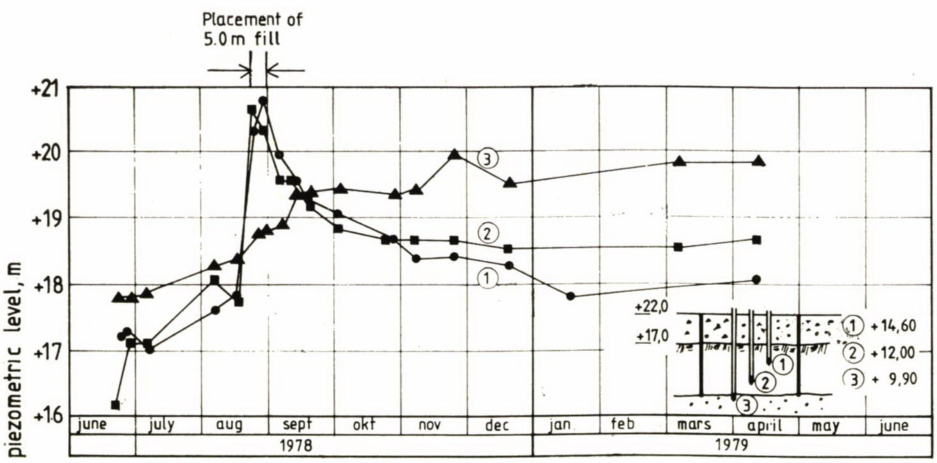


Fig. 28. Results of pore pressure measurements at different depths in the lime column stabilised soil. The results of the measurements demonstrated that the lime column functioned both as a reinforcing element and as an effective vertical drain

Some stabilisation and reconstruction works

Stabilisation of the Almhöjd area

The Almhöjd area is situated to the northwest of the slide area. In this area a total number of 35 homes had been evacuated, the houses resting on a 10m thick clay layer. The thickness of the remoulded clay adjacent to this area is of the order of 5 to 8m.

The Almhöjd area was without doubt the most urgent area to be stabilised. A safe stabilisation of the area was carried out by constructing a 108m long and 10m wide cofferdam of blasted rock (see Fig. 25). The cofferdam functioned both as a particularly reliable retaining construction and as a massive drainage ditch, keeping the ground water pressure in the underlying frictional soil at a safe level.

The stabilisation works for the Almhöjd area were completed in a relatively short period. On July 1st, 1978, the evacuees were invited to move back into their homes. Twelve families of thirty-five accepted this offer. However, those moving back to the area had one year to think the matter over before making a final decision. Those families who were not willing to move back to the houses within the evacuation zone were granted full economic compensation by the Swedish government.

Construction of a road embankment using lime columns

In the southern part of the landslide the important road connection to the areas Tangen and Tangelund was broken. For the reconstruction of the road it was necessary to place a 5m thick fill on the remaining 7-8m clay layer. In order to be able to construct the road embankment safely, the clay layer was stabilised using lime columns placed at a spacing of 1.4m.

After the installation of lime columns, a row of three circular sheet pile walls (diameter = 15m) were driven to firm bottom (Figs. 26 & 27). The road embankment was then placed on the stabilised soil.

The sheet pile walls were constructed primarily to prevent local stability problems in the clay during placement of the road embankment. The aim was that the lime columns would accelerate the consolidation of the clay so that the final paving of the road could be done only a few months after the construction of the embankment. The shear strength increase in the clay layer due to consolidation under the weight of the road embankment, together with the reinforcing effect of the lime columns, would give an acceptable factor of safety for the future

Fig. 29 (above and below). 18 months after the landslide (September 1979) almost all scars of the landslide have been removed. In general, the area was completely stabilised and reconstructed by the end of 1979

road, without taking into account the presence of the circular sheet pile walls.

The solution using lime columns for the construction of the new road embankment proved to be both a faster and cheaper method than a conventional concrete pile deck. An additional advantage of the lime column technique, in comparison with the latter alternative, is that the stabilised soil provides a smoother transition to the surrounding ground.

Pore pressure measurements

The behaviour of the lime column stabilised clay was studied by piezometers installed inside the circular sheet pile walls. The piezometers were installed at different depths in the clay and in the underlying permeable soil. As a result of the landslide, the piezometric level in the permeable soil had been lowered about 2m.

Fig. 28 summarises the results of the pore pressure measurements. During the construction, in August, 1978, of the 5m thick road embankment, the pore pressure in the clay layer increased by 30-40kPa. This pore pressure increase corresponds to about 40-50% of the weight of the road embankment. Thus, the measurements indicated that the lime columns carried about 50% of the embankment weight. After completion of the filling works, the excess pore pressure started to dissipate rapidly. This result shows that the lime columns effectively functioned as vertical drains. In about 2-3 months the consolidation process in the clay layer was completed — a result that must be regarded as most satisfactory.

The measurements also showed that in the permeable soil beneath the clay layer the original piezometric level was re-established due to the construction of the road embankment, cf. curve 3 in the pore pressure diagram.

Tuve 18 months after the landslide

Some 18 months after the landslide almost all scars of the slide had been removed (Fig. 29). On the whole, the area was completely stabilised and reconstructed by November 1979.

People are gradually moving back into the previous evacuation zone. For example, in the Almhöjd area all the homes are now re-occupied.

Acknowledgements

Most of the investigation work described in this Paper was planned and discussed in conjunction with the author's late friend and colleague, Dr. Leif Andréasson, Director of the Swedish Geotechnical Institute, whose untimely death occurred in 1980. This Paper is therefore dedicated to Dr. Andréasson's memory.

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