STATENS GEOTEKNISKA INSTITUT
SWEDISH GEOTECHNICAL INSTITUTE

RAPPORT
REPORT No 18

The Landslide at Tuve November 30 1977

ROLF LARSSON
MATS JANSSON

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PREFACE

This report deals with the geotechnical aspects of the landslide at Tuve in 1977.

In June 1978 the Swedish Government commissioned the Swedish Geotechnical Institute to investigate the geotechnical conditions in Tuve prior to the slide and if possible, to determine what factors caused the slide.

The investigation was led by the Director of the Institute Dr Leif Andréasson. After the tragic death of Dr Andréasson the concluding part of the investigation was delayed but has now been finished in a joint effort by the staff of the Institute.

The investigation has been performed in cooperation with a number of Swedish institutions and individuals as well as colleagues abroad. The investigators have greatly benefited from their help and experience.

All institutions and individuals do not necessarily share all the views expressed in this report.

Linköping November 1982

Rolf Larsson   Mats Jansson

CONTENTS

1. INTRODUCTION

The landslide at Tuve occurred on November 30 1977 just after four o'clock in the afternoon. The initial slide occurred suddenly and the events that followed took place in rapid succession. In about five minutes the appearance of an area of 27 hectares (270,000 m^2) changed completely. As a consequence of the slide 65 one-family houses were completely destroyed. A further 84 houses, of which many were severely damaged, lay close to or were hanging on the back edge of the slide and were in the danger zone for further slides. Nine people died from injuries caused by the slide. The reason that the death toll was not higher was that many people who lived in the area had not yet returned from their work or from school. The direct costs resulting from the slide amounted to about 150 million Swedish crowns (corresponding to about US $ 30 million).

The length of the slide area was 800 m and the greatest width was 600 m. In the upper central parts the ground level was lowered up to 10 m and in the lower parts the ground heaved about 5 m. Horizontal displacements of nearly 200 m took place.

The landslide at Tuve is no unique occurrence. Another large landslide in an urban area took place in September 1950 at Surte 8 km north-east of Tuve (Jacobsson, 1952). Tuve, however, was fairly densely populated and therefore the consequences were unusually severe. Some similarities between the Tuve landslide and a number of slides in soft clays can be found. Larger landslides with an area exceeding 1 hectare occur in Sweden every 2 or 3 years on the average, though only a few of them take place in urban areas. Similarities to the landslide at Tuve can also be found in a number of landslides in Norway. The frequency of landslides in clay is also high in the province of Quebec in Canada.

The frequency of landslides in Sweden has been increasing during the last century as a result of the building or railways and roads, urbanization and other human activities (Viberg, 1982).

Landslides constitute a considerable danger and great efforts have been made to build up the knowledge required to evaluate the stability of slopes and to take suitable steps to improve it. The mechanisms of landslides and the external factors that release slides have not yet been made sufficiently clear.

After the landslide at Tuve the Swedish Geotechnical Institute (SGI) was commissioned by the Government to investigate the geotechnical conditions prior to the slide and, if possible, to determine what factors might have started it. The investigation was made in cooperation with the Geological Survey of Sweden (SGU), the Geotechnical Department and the Geological Department at Chalmers University of Technology in Gothenburg and the Municipal Authority of Gothenburg.

A number of research projects concerning slope stability were started with grants from the Swedish Council for Building Research. A special grant to investigate the geohydrological conditions at Tuve was given to the consulting firm VIAK (SGI Report No 11c).

The landslide at Tuve attracted much interest and discussion among soil mechanics engineers in Sweden. To benefit from their experience consulting engineers were invited to SGI to take part of the results from the geotechnical investigations and to give their view of the cause of the slide (SGI report No 10). Their contributions as well as those from all others who have taken part in the discussion and given other kinds of help and information are gratefully ackknowledged.

To benefit from the experience in other countries with similar problems a symposium was held in Linköping in March 1982 with specially invited participants from Norway and Quebec (SGI Report No 17). The contributions to this symposium have been very helpful in the investigation of the landslide at Tuve.

Some symbols used in geotechnical investigations.

On maps

Static cone penetration or weight sounding

Undisturbed sampling with foil sampler or standard piston sampler

Field vane test

Penetration test to firm bottom

Measurement of pore pressure

In profiles

Type of soil

S Sand

Si Silt

M, M_s } Coarse silt

M_f, M_j } Fine silt

L Clay

G Gyttja

Sk Shells

Vx Organic matter from plants

v varved

m layer of coarse silt

Capital letters are used for the main type of soil. si L means silty clay.

In diagrams for undrained shear strength and sensitivity

Undrained shear strength determined by fall cone tests

Undrained shear strength determined by field vane tests

Sensitivity determined by fall cone tests

In diagrams for water contents and bulk density

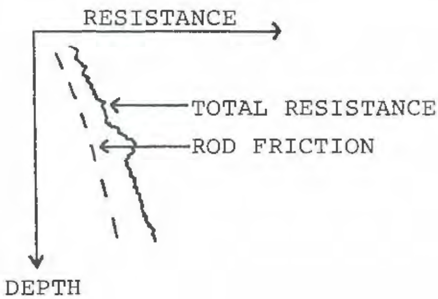
Natural water content

Liquid limit determined by fall cone

Plasticity limit

Bulk density

In diagrams for cone penetration tests



2. DESCRIPTION OF THE SLIDE AREA

2.1 Location

Tuve is a suburban area within the municipal region of Gothenburg. It is situated in the middle of Hisingen island about 10 km north of Gothenburg city centre, Fig 1. The landslide took place in a side valley of the main valley of the Kville brook. The main valley stretches across Hisingen island from north to south from Nordre River to Göta River. There are no visible scars from earlier landslides in the Kville valley. Probably earlier slides have occurred but so long ago that their sharp edges have been smoothed out with the passing of time. The Tuve area is located within one of the regions in Sweden hardest hit by landslides in soft clays, Fig 2.

2.2 Topography

The topography within the area is typical for this part of the country, Fig 3. The lowest parts of the slide area are within the main valley of the Kville brook. The ground level here is only a few metres above sea level. The ground surface in the main valley is almost horizontal. The side valley in which the slide took place was surrounded by higher areas with exposed bedrock. The valley was filled with clay on a layer of silt and sand on bedrock.

A smaller brook flowed through the side valley. The brook had created a gully a couple of metres deep. The banks of the gully were steep in places with an inclination of between 1:5 and 1:3. The upper part of the brook had recently been put in a conduit and the gully had been filled in. The small brook discharged its waters into the Kville brook.

The topography of the side valley is shown in detail

FIG 1. Location of Tuve.

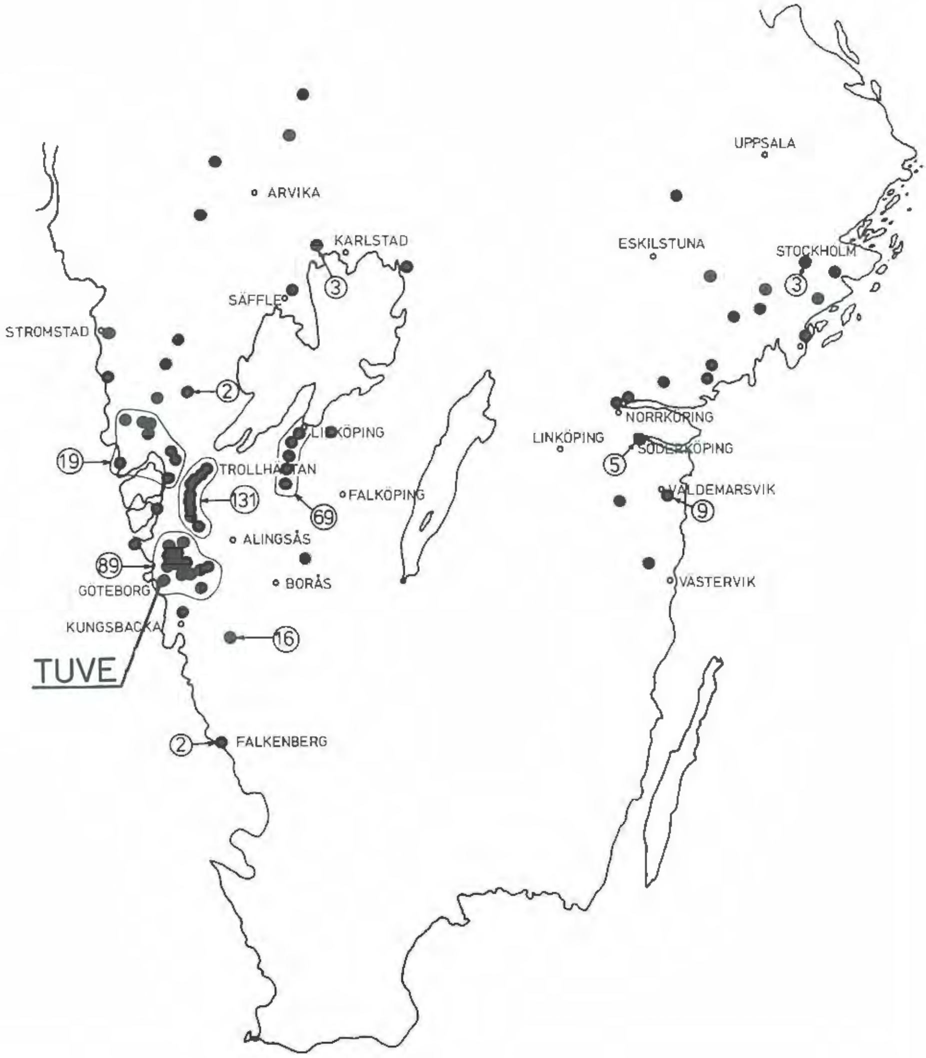


FIG 2. Located landslides in southern Sweden. The investigation is not complete and the number of slides in some areas probably far exceeds the numbers given in this map. (From Inganäs & Viberg, 1979.)

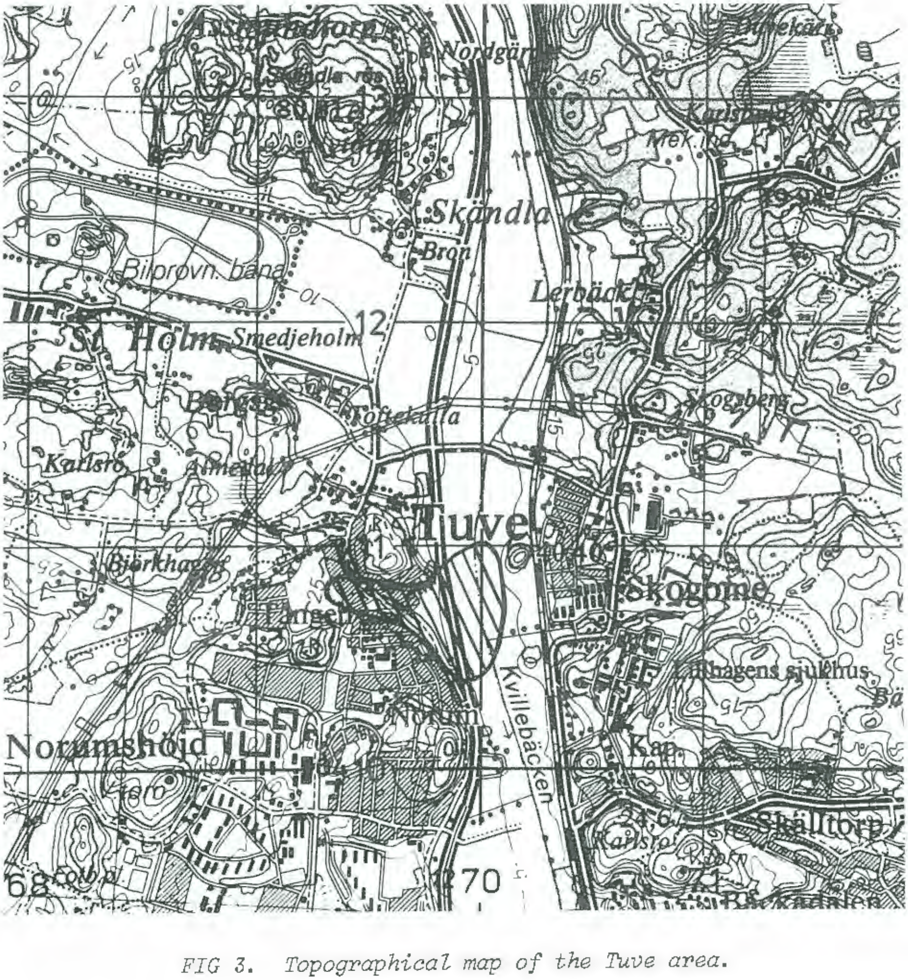


FIG 3. *Topographical map of the Tuve area.*

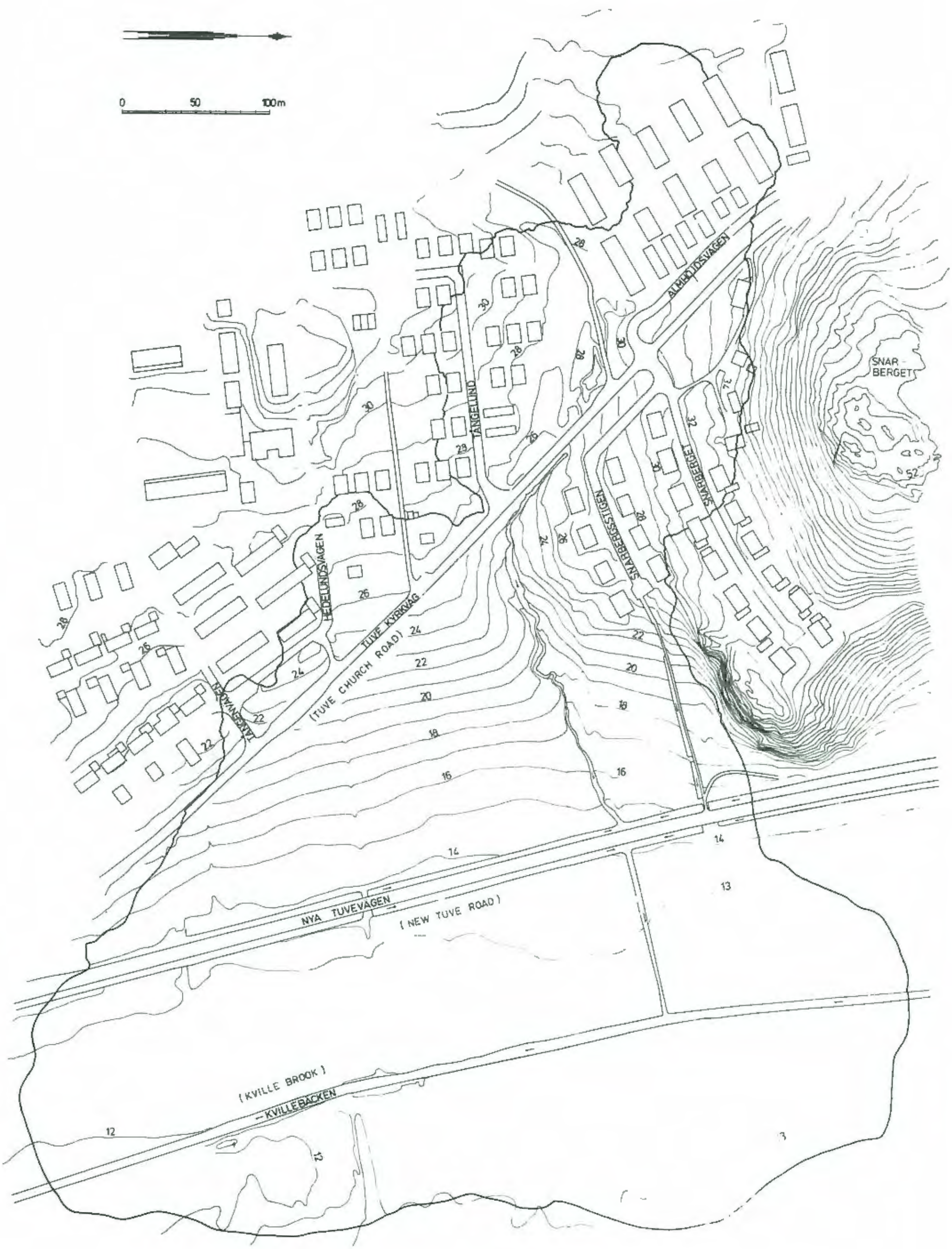


FIG 4. Topography of the side valley before the slide. The contour of the slide area is shown. Levels are given in the local Gothenburg system.

in Fig 4. The Kville valley is at the bottom of the figure. The Kville brook is in the middle and New Tuve Road (Nya Tuvevägen) runs along the west side of the Kville valley. The ground inclination between the road and the brook was 1:50.

In the area between Tuve Church Road (Tuve Kyrkväg) and New Tuve Road the inclination was steeper and amounted to 1:12.

The steepest parts were the gully banks just east of Tuve Church Road where there were inclinations of 1:5 and even 1:3 locally. The difference between the bottom of the gully and the surface of the road has been estimated to 4 or max 5 m from aerial photographs taken in 1970 (Fig 5). West of Tuve Church Road the ground level flattened out partly due to the infilling of the gully. The maximum difference in ground level between the highest and the lowest points in the slide area was 22 m. The horizontal distance between these two points was 700 m.

2.3 Development of the area

The area was developed mainly during the last two decades before the slide.

The location of Tuve Church Road had been largely unchanged for a long time (Fig 6). Formerly the road was called "Old Kungälv Road". Extensive reconstruction work was carried out in 1936. The road was widened and raised by between 0.4 and 1.0 m. After reconstruction the road ran on an embankment for a distance of about 80 m where it crossed the small brook. The largest rise in road level was at the crossing of the brook where a new culvert was installed. The embankment had an average height of 3 m at this point. According to the information received only minor adjustments had been made since 1936.

FIG 5. Tuve in 1970.

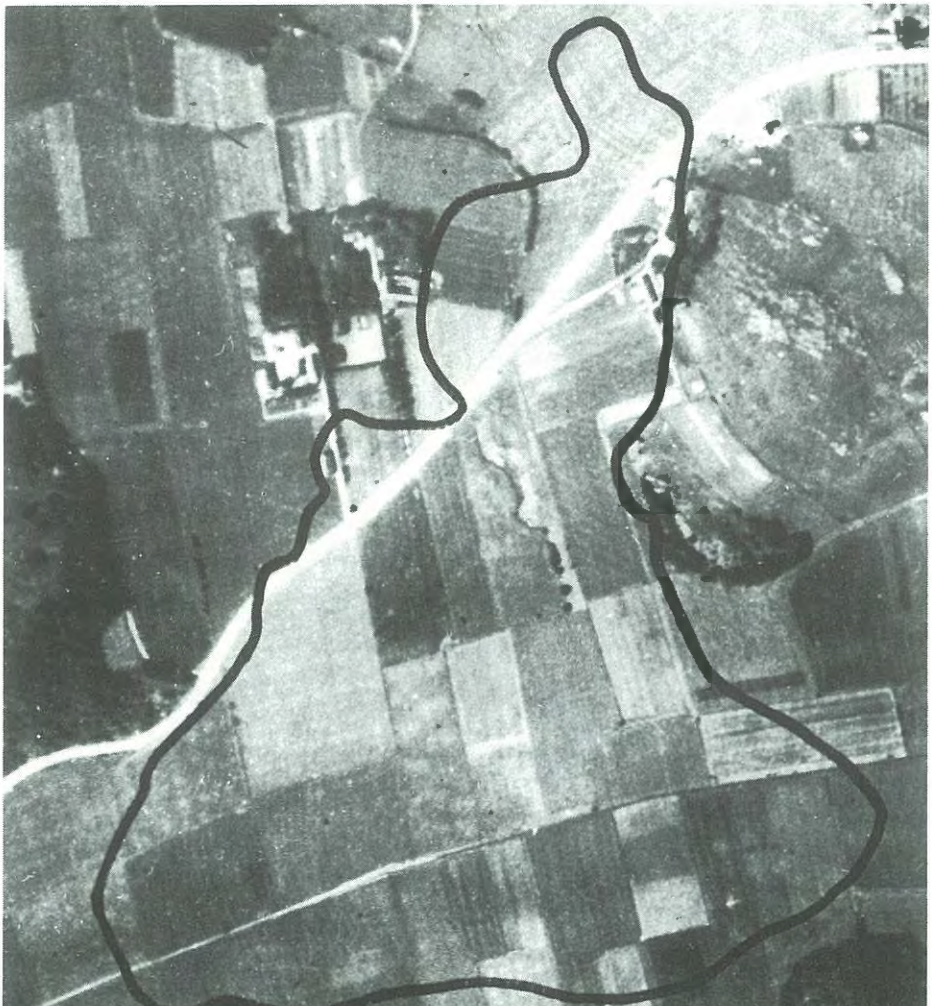


FIG 6. Tuve as a rural area in 1930.

New Tuve Road, which also was affected by the slide, was constructed in 1962 to take over the through traffic previously using Tuve Church Road.

The urbanization of the area was started in 1957 when rows of two-storey one-family houses without basements were built at Tångenvägen. (Names of roads are shown in Fig 4. Short list of words: vägen = road, berget = hill, stigen = path or road, lund = grove, bäck = brook.)

Development was continued in the middle of the 1960's when the Tångenlund area was built with linked 1½ storey houses without basements. A couple of years later the houses in the Almhöjd area and the Snarberg area were built. At Almhöjdsvägen rows of 1½ storey houses were constructed with basements and in the Snarberg area detached one-storey houses were built with and without basements.

During the same period two areas with multifamily houses were built west of and above the slide area. During the construction of these areas the storm water system was connected to the small brook flowing through the slide area. This increased the catchment area of this brook considerably compared to what it was before.

Development also involved laying large areas of impervious surfacing which further increased the maximum discharge of the brook.

Parallel to the brook a waste water sewer was laid straight through the slide area. This sewer served not only the houses in the side valley but also some surrounding areas.

During development of the area the upper part of the small brook was laid in a conduit and the gully was filled in. This was done in two stages. The first stage was performed in 1965 and comprised the part of the brook located west of the houses at Tångenlund. In 1970 the work was continued south of the Almhöjd area on to Tuve Church Road. This part of the work also involved a raising in the ground level west of the Tuve Church Road 1-2 m to ensure local stability for the south row of houses at Almhöjdsvägen.

3. GEOLOGY OF THE SITE

The geological conditions at Tuve prior to the slide have been investigated by a group of geologists from the Geological Survey of Sweden and the Geological Department at Chalmers University of Technology. The results from this investigation have been reported in detail in SGI Report No 11b (Cato et al, 1981).

The different quarternary deposits and their distribution in the area of interest are shown in Fig 7.

The thickness of the clay layer increased from 5-10 m at the back scarps of the slide to 20-25 m in the central parts of the slide area. Down in the Kville valley clay layers more than 35 m thick have been found. In the bottom parts of the clay and between clay and bedrock there are layers of sand and silt.

The bedrock at Tuve is characterized by trends in north-south directions dipping towards the west with mainly gneissic structure and granitic composition. A diabase dike, the Tuve dike, also runs through the area in the direction west-northwest to east-southeast, Fig 8. The distribution and orientation of existing cracks and jointed zones are important for the hydrology in the bedrock and the ability to

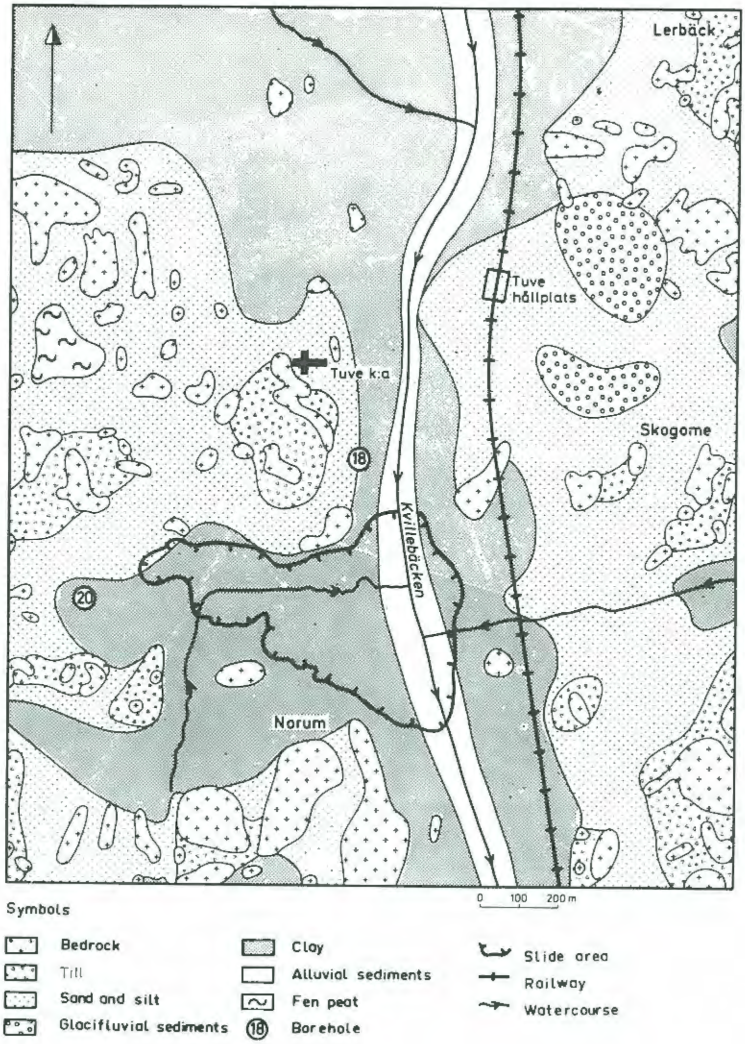


FIG 7. Surface soils in the Tuve area (after Sandegren & Johansson, 1931).

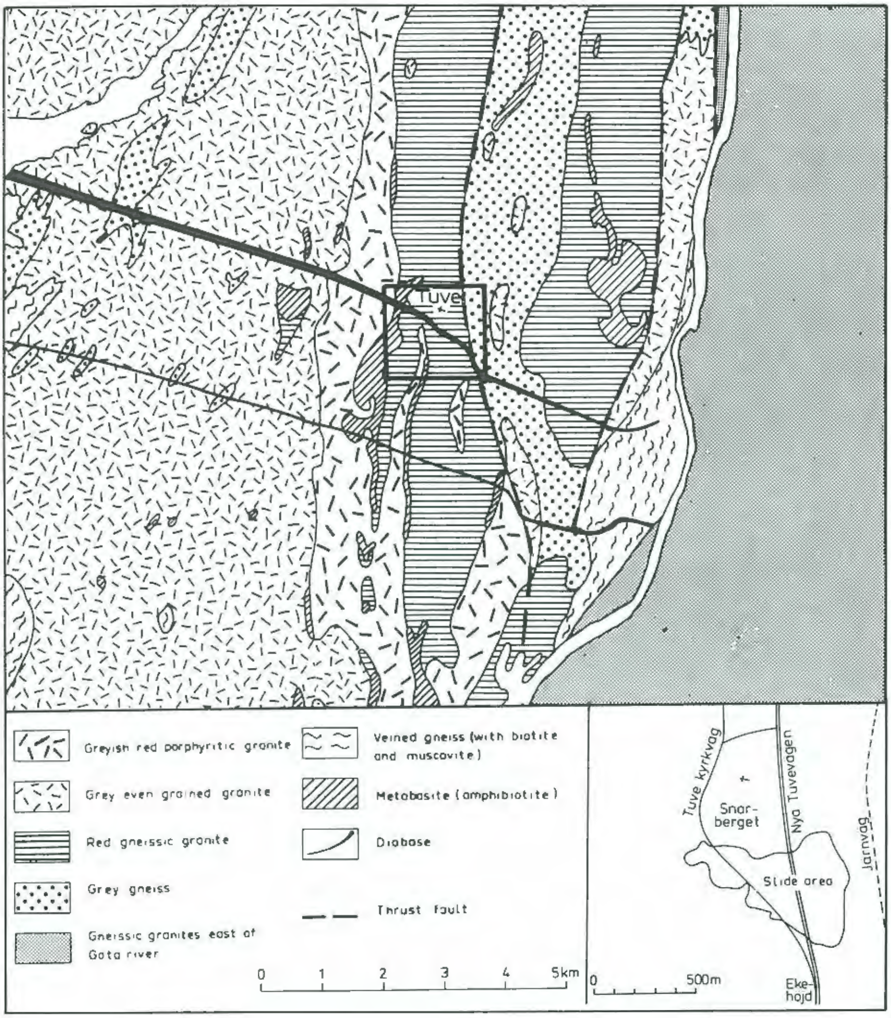


FIG 8. Bedrock in the Tuve area (after Lundegårdh, 1958).

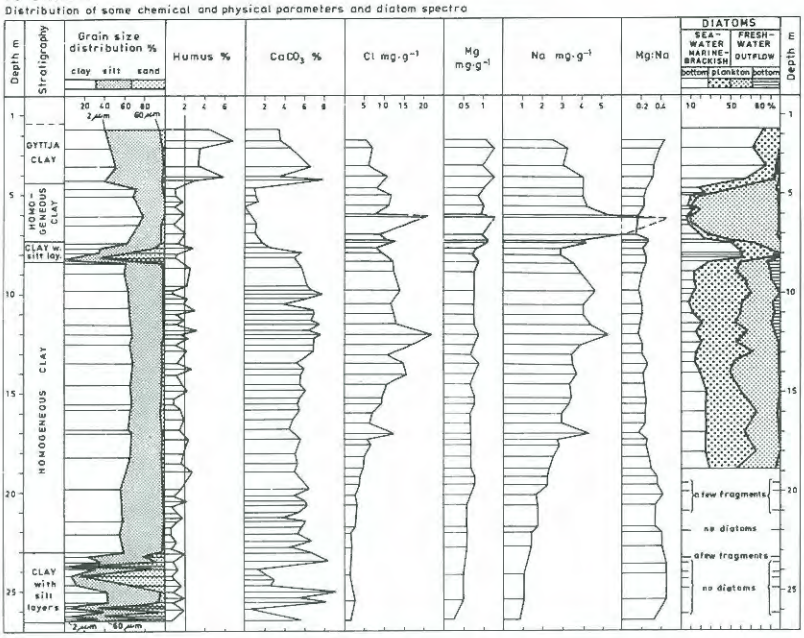
transport water. From knowledge of the bedrock at Hisingen island in general and geological experience from the field it can be assumed that water-carrying zones exist in a north-south direction some ten metres west of New Tuve Road and along one or both of the contact zones between the Tuve dike and the surrounding rock. In the middle and eastern parts of the slide area the bedrock is heavily jointed and often weathered. The permeability ought to be relatively high and hydraulic connection with a large drainage area may exist through large intersecting crack systems.

The geology of the soil in the slide area has been reconstructed by means of grain size analyses, chemical-physical analyses and bio-stratificational analyses on samples taken inside and outside the slide area. Exhaustive studies have been made on two cores (18 and 20, see Fig 9) taken outside the slide area. The first of these cores was taken in the Kville valley and the other was taken behind the back scarp of the slide. From the results of the investigations it is evident that the stratigraphy of core 20 corresponds to sequence of the lower part of core 18. The stratification in core 5, taken inside the slide area, is similar to the sequence of core 20. This implies that the clay layers in core 5 were originally located in the upper part of the slide area.

Based on the results from the investigations and present knowledge of the geological development in western Sweden during the last 13000 years a good idea can be obtained of the deposition of the various soil layers.

During the last glaciation gigantic ice sheets covered millions of square kilometres in northern

TUVE: core 18
Distribution of some chemical and physical parameters and diatom spectra



TUVE : core 20

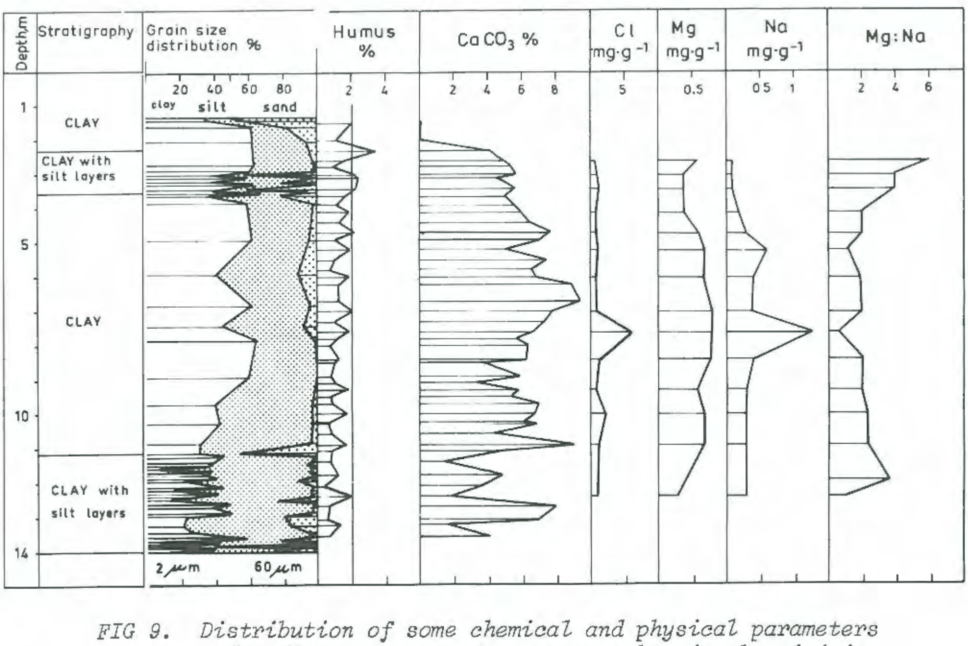


FIG 9. Distribution of some chemical and physical parameters and a diatom spectra from cores taken in the vicinity of the slide area at Tuve (Cato et al, 1981).

Europe. Deglaciation started about 18000 years ago as the climate became milder. The area of Tuve became clear of ice about 13000 years ago. During the deglaciation huge volumes of melt-water raised the sea level and areas such as Tuve, which had been depressed by the load of the former ice sheet, were submerged by the sea. In the lowest parts of the Kville valley the water depth was nearly 150 m. The main part of the clay sediments was deposited during a period of 4000 years. Because of the sedimentation and the isostatic uplift of the depressed crust which exceeded the rise in sea level, the water depth gradually decreased and large areas emerged out of the water until the whole Tuve area finally became land. The istostatic uplift is still going on but at a decreasing speed.

The sediments have been deposited in a marine environment with varying salinity, depths, currents and distances to the shore line. The thickness of the sediments is greatest in those areas where the water depth was greatest. New particles were deposited on the sea bottom and the previously deposited soil consolidated for the increasing overburden pressure. The inclination of the different layers depended greatly on the bottom contour and consequently the clay layers sloped towards the lowest part of the bedrock surface. At Tuve this part is located below New Tuve Road. A generalized picture of the stratigraphy at Tuve is shown in Fig 10 and the distribution of some parameters in cores 18 and 20 in Fig 9.

A varved glacial clay (Zone E), characterized by several layers of sand and silt rests directly on the Precambrian bedrock. This zone was deposited close to the retreating ice front. After the landslide it was found that the silt/sand layers were highly permeable (SGI report No 11c). Very high pore

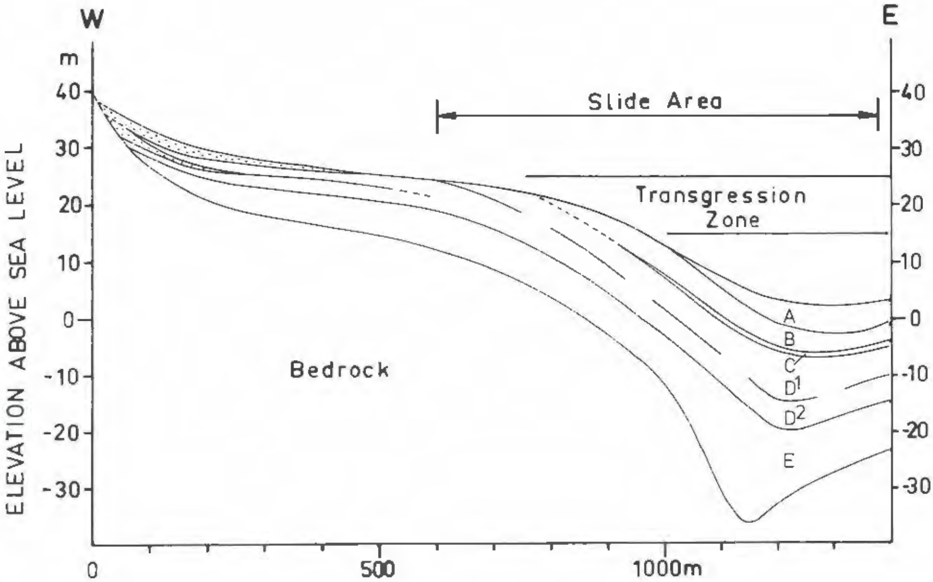


FIG 10a. Generalized soil profile through the slide area at Tuve. Geological zones. (Cato et al, 1981.)

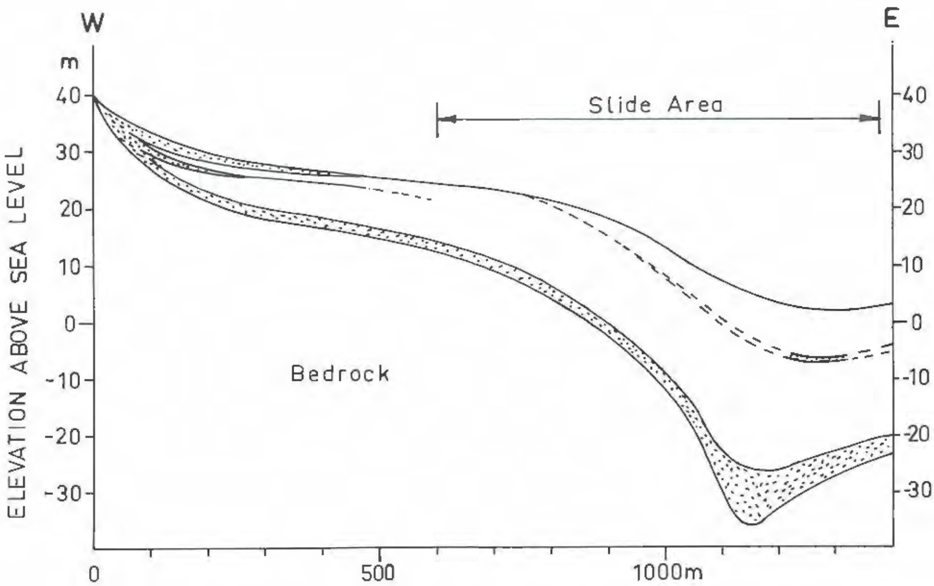


FIG 10b. Generalized soil profile through the slide area at Tuve. Zones where layers of sand and silt occur (dotted).

pressures were measured in the upper part of this zone after the slide and the slide surfaces seem to have followed the zone.

As the ice front retreated towards the north due to the change in climate the sedimentation conditions in the area became more stable. The more homogeneous clay in zone D was then deposited. The presence of marine-arctic planctic diatoms in the upper and middle part of the clay shows that it has been deposited in a marine-arctic environment at a great water depth.

In zone C, which is about one metre thick, there are several thin silt layers. Zone B consists of a homogeneous clay characterized by a very high clay content, 75-80%. Both zones C and B have very low calcite contents, below 1.5%. The deep water planctic diatoms in zone D have been replaced in the lower part of zone C by species living close to the shore line. In the upper part of zone C there is a pronounced decrease in salt water species among the microorganisms and a corresponding increase for the fresh water species. In zone B there is an extremely high incidence of planctic fresh water diatoms (*Melosira islandica*), originating mainly from the region of the Lake Vänern basin in central Sweden. The sediments and diatoms have been eroded and transported from this region and redeposited in the Gothenburg area. This process is a result of the melt-water flow and the drainage of 10000 km^3 of water from the ice-dammed Baltic basin into the Skagerrack about 10000 years ago.

As the water containing the particles flowed out into the Skagerrack it was spread over vast areas. The current decreased and the particles and organisms came into contact with the higher density underlying sea water and sedimented. In spite of the fresh water character of the diatoms the clay was thus redeposited in a relatively shallow (estuarian) environment dominated by sea water.

The youngest soil, zone A, is present only in the lower parts of the Kville valley. It consists of gyttja-clay which has been formed mainly during the transgression of the sea 9000-7000 years ago.

During the transgression the sea level rose and previously dry areas became flooded. The top soil was eroded and organic material and clay particles were transported to lower parts where they sedimented. At this time the Kville valley was a relatively shallow, warm and calm bay with a rich organic life. The lowest part of the Tuve area emerged from the sea about 1000 years ago.

Many of the clays in western Sweden and among them the clays in Tuve have thus been deposited in a marine environment. The clay particles of the non-expanding minerals have sedimented in a flocculated state (aggregates) due to the salts in the sea water. The clay has therefore become a very open structure with large pores and high water contents.

As a consequence of the topography at Tuve and the permeable layers at the bottom of the clay an artesian pressure often existed at the soil bedrock contact. If a hydraulic connection existed through the jointed parts of the bedrock with larger areas this situation may have been further accentuated.

Due to the artesian water pressures, diffusion and percolating surface water a leaching process has taken place and has reduced the salt content in the clay to various degress. Chemical changes such as reduction and complexing of several ions man have occurred. The original structure of a clay may remain after leaching but it will have lost some of its strenth and become much more sensitive to mechanical disturbances.

The salt concentrations in the samples confirm that leaching has occurred. In the upper part of the slide area where the thickness of the clay is only 10-15 m the whole soil mass seems to be leached. In the lower parts where the clay thickness is up to 35 m only the bottom and top layers are obviously affected. This confirms that leaching has occurred from the permeable layers below and from percolation at the ground surface.

Geological investigations of landslides in western Sweden have previously been made for instance in connection with the landslide at Surte 1950 (Mohrén, 1956). The methods and techniques used in the two investigations differ but in general there are certain similarities in geology and soil stratification between the areas at Tuve and Surte.

4. HYDROGEOLOGY AND HYDROMETEOROLOGY

It is well established that the frequency of landslides in western Sweden has a peak during the rainy months in the autumn (Viberg, 1982). High water pressures are considered as one of the main factors in the initiation of landslides.

After the landslide at Tuve the consulting firm VIAK received a grant from the Swedish Council for Building Research to investigate the geohydrological conditions at Tuve prior to the landslide.

The investigation, which has been reported in SGI report 11c, comprised test pumpings at three locations. Hydrometeorological data has been collected from meteorological stations nearby and have been compared to tong term observations of groundwater level and pore pressures. Other data have been obtained from the borings and observations after the slide.

Contrary to the assumptions of the geologists the investigators conclude that the bedrock in the slide area in relation to overlaying layers of silt and sand can be considered impervious. The main groundwater aquifer in the valley therefore consisted of the coarser layers on top of the bedrock. It was a closed aquifer with a limited capacity. The aquifer was replenished at the sides of the valley where the coarser layers came to the surface. Due to the topography and the fact that overlaying clay which reached some way up the valley sides acted as an impervious seal the water pressure in the valley could often become artesian. This artesian pressure, however, was limited as the groundwater after the aquifer was filled to the brim (where the coarser layers surfaced) was discharged on the ground surface.

At the time just before the slide the aquifer was brimming over and the investigators have calculated that the groundwater pressure in the bottom layers in large parts of the area corresponded to a groundwater level of more than two metres and in some lower areas more than four metres above the ground surface. The accuracy of the calculation is estimated as ± 1 metre of water pressure.

Records from the meteorological station at Säve nearby have been studied. In general 1977 was a year with a high precipitation without being exceptional. The month of November, however, was the rainiest month since 1970 when the last stage of the development of the area was finished. There are other factors than the precipitation which are important for the replenishment of groundwater. If the net of precipitation (precipitation minus evaporation) is studied, 1977 is found to be an exceptionally wet year and the month of November 1977 an unusually wet month. See Tables 1a and b.

Month (mm)												
Year	1	2	3	4	5	6	7	8	9	10	11	12
1971	63	33	62	43	32	72	71	119	88	67	89	40
72	28	20	29	70	97	104	109	36	20	31	63	98
73	69	53	18	64	54	26	24	64	75	35	92	75
74	79	42	19	0	28	34	52	66	149	54	106	87
75	104	7	20	42	37	23	106	39	117	41	76	48
76	47	22	19	21	51	29	16	7	55	101	103	95
77	98	48	81	83	42	66	69	14	93	80	120	53
78	41	29	118	15	20	58	29	108	127	32	70	26
Average	66	32	46	44	45	52	60	57	91	55	90	62
Average 1945-1978	58	37	38	44	45	52	66	77	77	72	72	66

Table 1a. Precipitation measured at the meteorological station at Säve.

Month (mm)													
Year	1	2	3	4	5	6	7	8	9	10	11	12	Accumulated for the year
1971	48	18	45	19	-18	-23	-29	19	45	35	74	24	+257
72	28	5	11	46	45	34	9	-49	-18	3	43	78	+235
73	54	35	-6	40	6	-64	-76	-21	29	13	82	67	+159
74	63	24	2	-34	-20	-49	-31	-19	94	31	88	71	+220
75	104	-8	2	18	-17	-67	6	-61	53	9	59	32	+130
76	47	7	19	-4	3	-64	-84	-93	13	75	84	95	+98
77	98	48	64	64	-6	-34	-31	-86	51	48	103	43	+362
78	26	29	95	-8	-28	-37	-66	13	89	0	48	26	+187
Average	+59	+20	+29	+26	-4	-38	-38	-37	+45	+27	+73	+55	+206

Table 1b. Net of precipitation (precipitation minus evaporation).

As the groundwater level could not rise more than to the brim level the investigators assume that there should have been an extremely high surface discharge of water in November 1977. Witness reports from inhabitants in the area seem to confirm this assumption.

5. THE SLIDE EVENTS

5.1 Events according to eye witnesses

After the landslide people who lived or were present in the area at the time of the slide were interviewed about their observations and experiences before and during the slide. The interviews were made by the staff at the Geological and Geotechnical Departments at Chalmers University and have been collated and evaluated in a special report (Fält, 1978).

In spite of the relatively large number of people living in the area the number of actual eye witnesses is limited. Fortunately many people had not yet returned home from their work or from school. Most people present were indoors as it was in November and the weather was foggy.

The combination of foggy weather and dusk made accurate observations with a greater perspective impossible. In fact the whole extent of the slide became known only the next morning at dawn. The visibility at the time of the slide has been estimated as about one hundred metres.

Witnesses have reported that first the lights went out, then creaking and cracking noises were heard in the house and a short while after the house started to move and the ground cracked up. Other witnesses standing at the bus stop on the corner of Tuve Church

Road and the exit road from the Snarberget area reported that first a lamp standard for the street lighting started to swing as if it were were standing in water. A short time after this cracks opened up and the landslide started.

Some of these observations are inconsistent. Power cables usually break due to large movements. Houses do not creak and crack in an alarming way unless they are subjected to considerable strains and lamp standards do not swing in the way reported if they are standing in stable ground.

It is quite possible that the soil started to move so smoothly and slowly that it was not percepted among the more obtrusive visual and acoustic sensations at the initiation of the slide. It was later found that the slide followed the contour of the bedrock which was uneven. After limited travel the slide masses therefore had to start to break up. At this moment, when cracks opened up and houses nearby started to move in relation to each other the movements must have become obvious. There is also at least one witness in the area that was first observed to move who perceived that the whole area was moving before cracks started to open up. The eye witnesses can thus not give an exact idea of how and where the slide started but a rough picture can be drawn from a summary of their accounts.

The first obvious sign of large soil movements was the formation of a crack in Tuve Church Road. The crack ran from the bus stop at Snarberget down to the exit from Tångelund, Fig 11. In the following moments parts of Tuve Church Road were seen to move down towards New Tuve Road followed by the houses at Snarbergsstigen and Snarberget in succession. At about the same time as the houses at Snarberget were seen

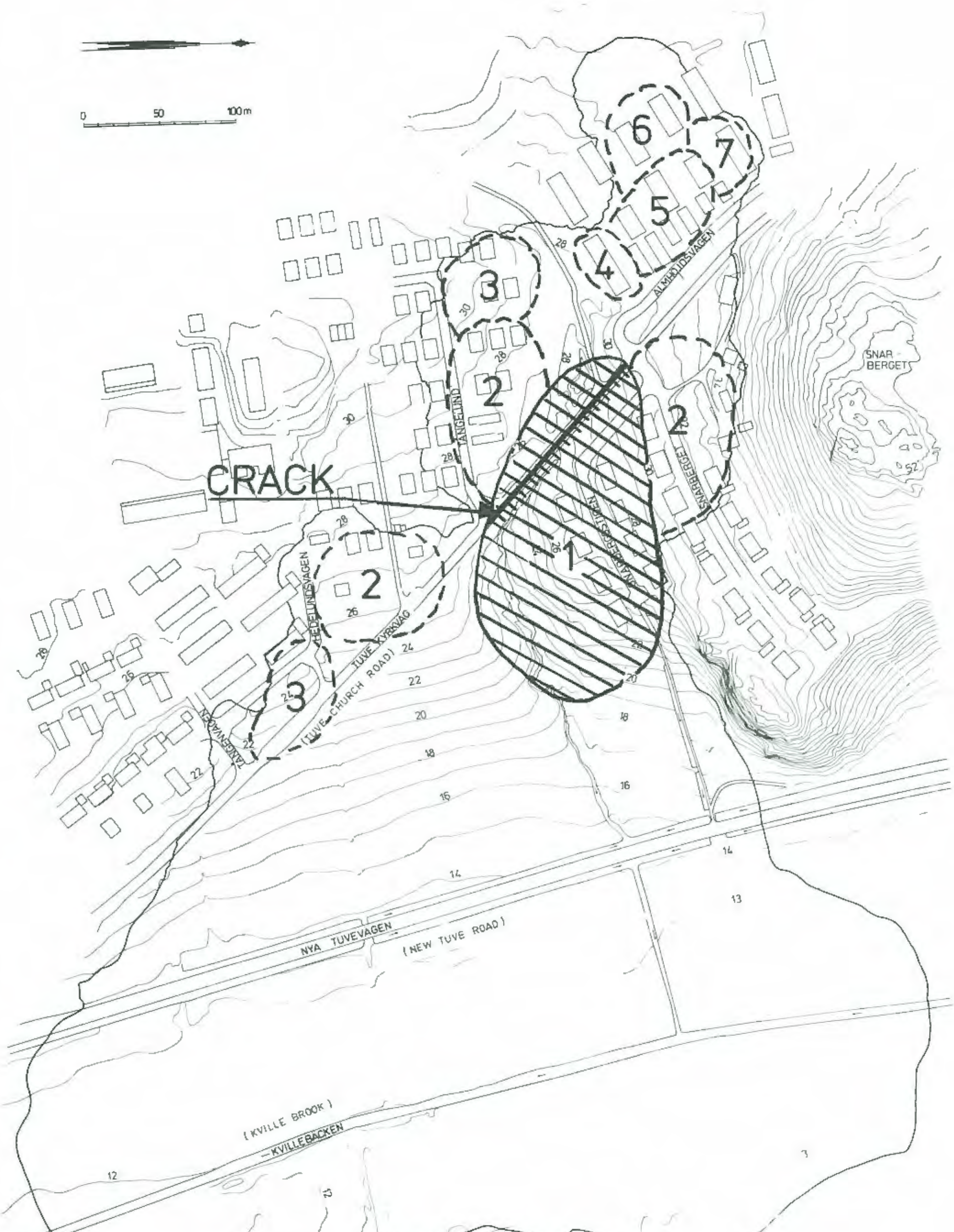


FIG 11. Sequence in which housing-areas started to move according to eye witnesses.

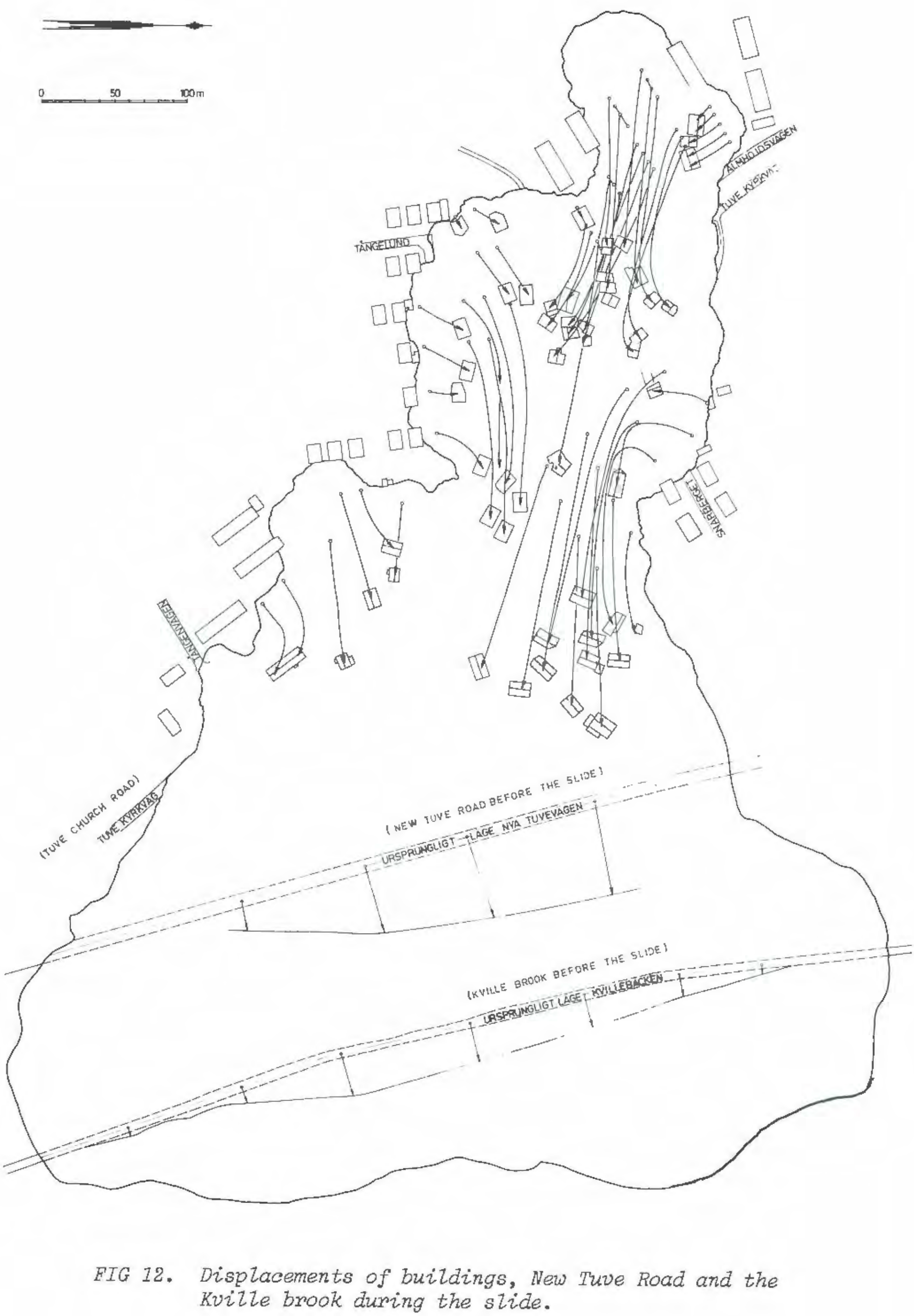


FIG 12. Displacements of buildings, New Tuve Road and the Kville brook during the slide.

to be in motion the eastern houses in Tångelund and the houses at Hedlundsvägen started to move. The slide then retrogressed and the houses at Tångenvägen and the rest of the houses in Tångelund followed. The slide area then reached Almhöjdsvägen and the rows of houses started to slide away one after another. The sequence in which the houses were observed to set in motion is shown in Fig 11 and the horizontal displacements of the individual houses are shown in Fig 12.

The retrogressive slide in many places reached firm ground before it stopped.

There are reliable witness accounts for the final phase of the retrogressive slide and the time has been established by automatic recordings of power cable failures. No less than seven power cables were broken between 16.05.33 and 16.09.29. These cables served not only the Tuve area but also large parts of the surroundings. The first cables to break may have been involved in the initial slide but can also have been broken during the retrogression of the slide.

The last cable failures occurred in the Almhöjdarea and coincide with the final phase of the retrogressive slide. The first signs of anything happening were observed "a few minutes" past four o'clock. All larger soil movements during the landslide thus seem to have taken place within about five minutes. Nobody observed what happened below Tuve Church Road. It is therefore impossible to conclude from witness accounts whether some part or parts in the shaded area (area 1 in Fig 11) moved first or if the initial slide comprised all this area.
There is a general feeling about the observations before the slide that there was much water in the area, in fact more water to be seen at the surface and in the brooks than ever before.

5.2 The extent of the slide

The full extent of the slide became known the next day at dawn.

A total area of 27 hectares (270,000 m^2) was involved in the slide which makes it somewhat larger than the slide at Surte. The final shape of the slide area was governed by areas of solid ground on both sides of the valley and was triangular with a base of 600 m and a height of 800 m. The western part of the slide area did not reach firm ground but ended with back scarps of clay up to 10 m high. A few patches of clay remained in the southern part. The slide went deep and for the greater part of the slide area it largely followed the contour of the firm bottom. Well defined slip surfaces could be seen in the clay at several points along the back scarp. These exposed slip surfaces also followed the contour of the firm bottom and here the remaining clay had in general a thickness of only a metre or two.

The back scarps were often vertical or almost vertical and a few superficial slides occurred afterwards.

The ground surface fell by up to 10 m in the upper half of the slide area. Soil displacements of nearly 200 m were measured here. The average rise of the surface in the lower part was 5 m.

As a rough estimate about 6 million cubic metres or 10 million tons of clay were involved in the landslide.

5.3 Consequences of the slide

The slide took place within a suburban area. Buildings, roads, cables, pipes and other constructions were deformed, displaced and broken up in pieces,

Appendix C. During the event 8 people were killed and afterwards about 40 injured were taken to hospital. Only 8 of these had to remain for care in hospital but one man later died from severe injuries received during the slide.

65 one-family houses - detached houses, linked houses and terraced houses - were carried away with the soil masses. Another 84 houses were located in the danger zone for further slides. About thirty of these were standing right on the back scarp of the slide in various states of demolition. A total of 436 persons became homeless; 230 from the actual slide area and 206 from the danger zone. Of the roads in the area only New Tuve Road provided a through route. The regional traffic could therefore be diverted around the slide area without great inconvenience. The railway which runs along the eastern side of the Kville valley was not affected by the slide. Apart from some local disturbances the landslide did not cause any serious communication problems.

The breaking of cables and pipes stopped the supply of power and water to surrounding areas for a short while but temporary supplies were soon connected.

The Kville brook was dammed by the slide masses but the high point and natural water divider in the valley was just a few hundred metres to the north and insignificantly higher than the lower part of the slide area. The water divider therefore moved a few hundred metres to the south and no flooding resulted from the slide.

6. REMEDIAL MEASURES

6.1 The rescue operations

The first alarm from the catastrophe went to the firebrigade which had people on the spot within ten minutes. The first task was to rescue people who were trapped in the slide area and in the first actions some 60 persons were brought to safety. The operations were very difficult due to the fog, the dark and the lack of electrical power. When the extent of the catastrophe became clear the head of the firebrigade asked the County Authorities to take charge of the rescue operations, which they did the same night. At the same time the head of the firebrigade was commissioned to lead the relief work at the site. During the following days about 70 men from the firebrigade were continously at work in the slide area. A large number of people from the homeguard and other military units were also put into the operations. The County Authorities remained in charge until December 5, one week after the slide. A detailed report on the rescue operations has been written (Länsstyrelsen, 1977).

The engineers from the Municipal Authority of Gothenburg were in charge of the geotechnical activities from the start. Close cooperation was established between the geotechnical staffs at the Highway and Street Department and the Department of Planning and Building Control. The former team led the activities at the site and the latter handled the administration. This organization remained for the whole period of reconstruction work in the area which lasted for two years.

Two consulting firms, Jacobson & Widmark AB and VIAK AB, were retained to take part in the geotechnical investigations and estimations. The Authorities also requested an advisory group of three specialists from outside to take part in the geotechnical estimations.

Members of this group were Dr Leif Andréasson, Swedish Geotechnical Institute, Mr Gösta Berg, Ludvigssons Consulting Engineers and Prof Allan Bergfeldt, Chalmers University of Technology. The consulting firms and the advisory group assisted during the whole period of reconstruction.

The geotechnical activities were started on the night of the slide by collecting and studying previous investigations in the area. The next day temporary roads were laid into the slide area. Investigations for evaluation of the risk of further slides were also started.

The most important task for the geotechnical engineers directly after the slide was to define what area was in danger of further slides or movements. Apart from the 65 houses which were carried away in the slide another 130 houses were considered in danger just after the slide. For a short while yet another 100 houses were considered unsafe and an evacuation of these was started. However, after preliminary geotechnical investigations this evacuation was stopped. The investigations required to establish the limits of the danger zone were carried out during the first week after the slide. A boundary for the danger area was drawn on December 9, Fig 13. This permitted the inhabitants to move back into 46 houses while 84 houses were considered unsafe. This boundary was kept in the first stage of the reconstruction work.

6.2 Reconstruction of the area

After the first week the geotechnical activities were directed towards saving what could and ought to be saved and to producing the necessary basis for a new plan of the area. The engineers made cost calculations for different solutions to the reconstruction problems and a large number of alternatives were studied. These calculations were considered when decisions were taken on which buildings should be saved and which should be demolished.

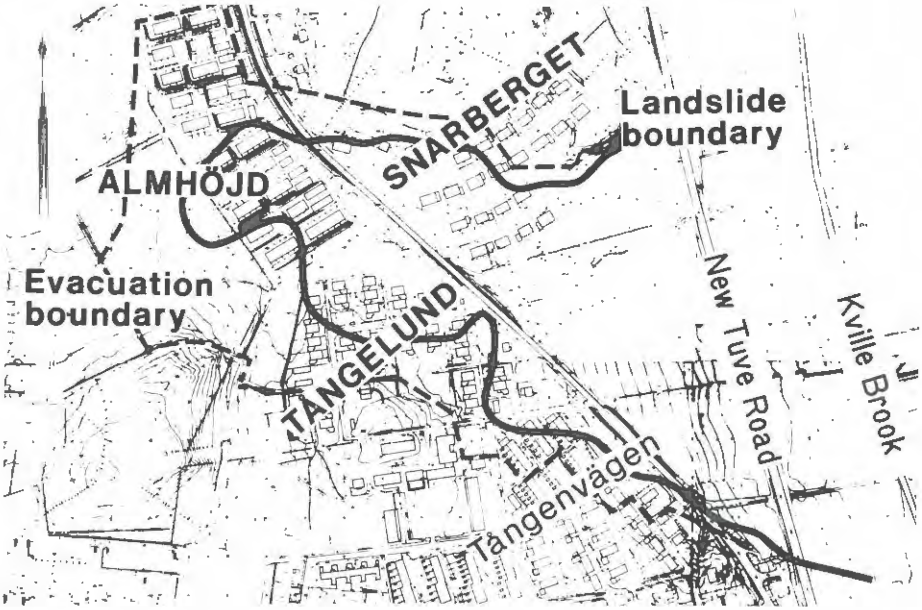


FIG 13. Boundary of the evacuation zone.

Directly after the slide a number of temporary stabilizing measures were taken. At an early stage fibre-concrete was sprayed at parts of the steep back scarp of the slide area to prevent superficial slides in the clay. This measure proved to be successful. Some houses were supported by beams and struts. Others were secured by temporary lightweight fills. These meausres were aimed at preventing further destruction of the houses before they could be restored or in some cases at enabling safe removal of personal property before demolition of the buildings, Fig 14.

The reconstruction work was carried out with the intention of giving remaining houses "quite sufficient" protection against further landslides. This meant that extensive stabilizing measures had to be taken for the Almhöjdarea in the northwest and for all buildings along the southern boundary of the slide area. No extensive measures had to be taken to ensure stability for the buildings at Snarberget as these were founded on bedrock or firm ground. Only one of the remaining houses here, in the southwestern part of the group, had to be stabilized as one corner of it was hanging free in the air. The greatest problem at Snarberget was to construct an access road to the area.

The stabilizing method that was used most extensively was to excavate the clay and replace it with rockfill. From a technical point of view there were more elegant and somewhat cheaper solutions but from a psychological view the rockfill "felt" safer. The economic gains of using other methods were insignificant and as the rockfill solution also had some practical advantages it was used in almost all locations.

South of the Almhöjdarea, which was the largest housing area in the danger zone, two back tied sheet pile walls were put into place before the soil between them was excavated and the rockfill was tipped in, Fig 14.

Back tied sheet pile walls were also used in some places north of Tångelund. When making calculations the sheet pile walls were only supposed to ensure stability during the excavation period. For long-term stability calculations only the effect of the rockfill was taken into consideration.

In one small area the clay was too deep for the excavation-refill method and other methods had to be used. Here the clay was stabilized with both circular sheet walls and lime columns.

Temporary support for houses.

Placing of rockfill between back tied sheet walls

Lightweight fill to improve stability.

FIG 14. Reconstruction work in the slide area.

The new roads in the slide area were, whereever possible, constructed on top of the rockfill. In the central part of the area, however, they had to be built floating on the clay. Settlements are expected on these roads. This is especially valid for New Tuve Road which was reopened for traffic as early as January 1978.

Light fill (expanded clay pellets) was used in the Almhöjdarea and at Tångenvägen to form pleasant slopes without overloading the ground.

In other places a suitable slope was obtained by flattening out the steep back scarp. This was the case at the "tongue" in the western part of the slide area. Here the two remaining rows of houses which were damaged were torn down and the ground was evened out.

Several buildings had their foundations improved by piling to prevent settling as the groundwater level was lowered close to the slide area. Most of the houses had to be repaired to various degrees.

The houses in the Almhöjdarea were the first to be restored and the inhabitants could return to their homes on July 1, 1978. The stabilizing measures for the houses south of the slide area were finished in spring 1979 although some repair work remained.

The Municipal Authorities in Gothenburg together with the consulting firms retained have given a comprehensive account of all the measures taken in connection with the reconstruction work; stabilizing measures, geohydrological investigations, measurements of deformations, prognoses for future settlements and a report on what future checks should be made.

The people who became homeless and those who had to be evacuated from the danger zone were first given shelter by friends and relatives, accomodated by the local authorities or in military cantonments and then as soon as possible offered alternative accomodation.

After the landslide at Surte new residential areas were immediately constructed in the vicinity but this was not possible at Tuve. The authorities worked on the principle that nobody should be hurt further, either economically or emotionally. All the homeless people were therefore offered the full value of their old houses and were given priority in the queue for new houses and sites in the Gothenburg region if there was something in that housing program they preferred. This offer was also open to the evacuated people whose houses were later reconstructed as some of them could not face the prospect of going back.

Even some people outside the evacuation zone had their houses bought by the authorities.

From the insurance point of view the situation after the slide was not quite clear but the insurance companies shortly afterwards decided to pay for the demolished houses, lost personal property and repair work on the houses.

7. GEOTECHNICAL INVESTIGATIONS

7.1 Earlier investigations

One of the main problems in investigating a landslide is that all the original material has gone and only displaced, disturbed and sometimes mixed material remains for investigation. In most slides no investigation has been made prior to the slide.

In this respect the investigators of the landslide at

Tuve had some advantage as at least some investigations had been made before the slide. The development of the area was carried out in steps. Local geotechnical investigations were made in connection with planning the different parts. Each investigation normally comprised weight-sounding or penetration testing in a net pattern adapted to the terrain with a distance between the penetration points of about 40 m and some sampling with a piston sampler. The geotechnical reports concerning areas close to the small brook mentioned the risk of local slides in the gully slopes if the ground near the gully was loaded. For this reason zones without a load increase were recommended close to the brook. These recommendations were considered in the planning of the area. To eliminate the risk for local failures in the slope south of the Almhöjdarea the brook was put in a conduit and the gully filled in. An extra fill of 1-2 m was put on the lower parts of the ground.

Within the area affected by the slide there were about 60 penetration points and samples had been taken in 7 holes. The number of investigations is normal for a survey in the planning stage aimed at obtaining enough information about the soil to find suitable locations for buildings and to choose a method for foundation construction. No investigation had been made regarding overall stability of the area.

Most of the investigations had been made by weight sounding, which is a very coarse method for investigating clay. The results, however, show that there was soft clay all over the area and all the way down except for a metre of sandy material between clay and bedrock.

In the later investigations penetration testing with continuous registration of the total penetration force and a slip-coupling to separate rod friction and tip resistance was used. These investigations were made in the Almhöjdarea, in the area of Snarberget and Snarbergsstigen and further down to the small brook.

The latter area is of particular interest as the slide seems to have started here. A plan of the penetration tests, weight soundings and sampling in this area is given in Fig 15 and the results are gathered in Appendix A.

The penetration tests generally show that the tip resistance, which normally increases with depth, only increased during the first ten metres of penetration, then became constant for some metres and finally decreased towards the bottom. In many cases there was hardly any tip resistance at all for the last three metres before the tip reached the sand layer on top of the bedrock. The penetration results indicate that there was a layer of very loose silty clay/clayey silt or clay with loose silt layers between the clay and the sand.

The samples were usually taken with an older type of piston sampler and their degree of disturbance may be discussed. However, in all sampling points the liquid limit rapidly decreased towards the bottom indicating a higher silt content.

Typical values for the lowest samples which were taken at some metres from the bottom were liquid limits of 30-35%. The water contents were generally 10-20% higher indicating a high liquidity index and loose material.

The general pattern conforms to the geology of the area and to what has later been found in the investigations inside and outside the slide area. Unfortu-

FIG 15. Earlier investigations of the area between Snarberget and the small brook.

nately no vane tests were made to measure the undrained shear strength prior to the slide.

7.2 Investigations after the slide

The day after the landslide comprehensive geotechnical investigations were started. Samplings, penetrations, vane borings and pore pressure measurements were carried out by field crews from the Municipal Authority of Gothenburg, the consulting firms Jacobson & Widmark AB and VIAK AB, the Geotechnical department at Chalmers University of Technology and the Swedish Geotechnical Institute, and samples were examined in the laboratories. During the first week activities were concentrated on the housing areas adjacent to the slide to evaluate the stability close to the back scarp of the slide. The results from these investigations were used to establish the danger zone for further slides and movements. When this task was finished the investigations within the actual slide area started.

These investigations aimed at creating a basis for reconstruction planning and the evaluation of necessary stabilizing measures. Very high pore pressures were measured in the slide area and large future settlements were expected in the masses involved in the slide. Penetration tests with a pore pressure probe were carried out to investigate homogeneity and thereby to learn something about the consolidation characteristics of the soil. Small test fields were made to examine the effect of prefabricated drains for lowering the pore pressure and the behaviour of lime columns in the Tuve clay. The objectives of the different investigations varied and they have been reported in different ways and at different times, e.g. Torstensson, 1979.

Measurements of movements in the area were carried out by observing a number of houses close to the boundary of the slide. The first clear day after the slide the area was photographed from the air. To investigate eventual changes new aerial photographs were taken in the middle of January 1978.

Staff from the Geological and Geotechnical Departments at Chalmers University interviewed eye witnesses of the landslide and Lars-Gunnar Hellgren from the Department of Planning and Building Control tracked the original and final locations of buildings involved in the slide, Fig 12. They also investigated the topography of the area before and after the slide and the topography of the bedrock.

Photographic documentation of the slide area was also made, Appendix C.

A special investigation was made by the Department of Building Construction at Chalmers University which studied how the buildings involved in the slide had sustained the very severe strains imposed by the slide. Some houses had moved over a hundred metres practically without any damage while others were totally destroyed after a displacement of only a few metres (Johannesson & Johansson, 1979).

In spring 1978 VIAK AB received a grant from the Swedish Council of Building Research to investigate the hydrogeological conditions at Tuve prior to the slide. This investigation involved test pumpings in the area and collection of meteorological and hydrological data from surrounding areas.

The Swedish Geotechnical Institute was not officially commissioned to investigate the landslide until seven months after it occurred, although the work in fact started earlier. The investigation was performed in cooperation with the Geotechnical Department at Chalmers University and with the Geological Survey of Sweden and the Geological Department at Chalmers University. The two latter institutions investigated the geology of the area.

7.3 Field investigations made for the slide investigation

These field investigations were made in two stages. The first stage was carried out in the period January to March 1978 and comprised:

- Penetration tests and weight sounding tests to determine roughly the thickness and relative firmness of the soil layers.
- Field vane tests to determine the undrained shear strength. The tests were performed with standard vane size 65 x 130 mm and recording instruments. Equipment with and without outer casings was used.
- Penetration tests with pore pressure probes for continuous registration of the soil stratification and to find any permeable layers.
- Undisturbed sampling with the Swedish standard piston sampler.
- Taking continuous cores with the Swedish metal foil sampler.
- Measurements of pore pressure.
- Measurements of electrical conductivity in the field by "salt sounding" to detect a slip surface.

When the results from these tests had been analyzed and the laboratory results were available it became evident that the slide had gone deeper than previously assumed. Supplementary investigations were therefore made during February and March 1979. They were carried out down to firm bottom and comprised:

- o Field vane tests in four locations in the central slide area. The tests were made with standard vane size 65 x 130 mm outer casing and recording instruments.
- o Undisturbed sampling with standard piston sampler in two locations.

A plan of the investigations into the slide is shown in Fig 16. Two of the boreholes, 18 and 20, are located outside the slide area in places unaffected by the slide. They were taken to obtain an idea of what the soil properties in the slide area were before the slide and have also been used in the geological investigation. Results from the field and laboratory investigations are shown in Appendix B.

7.4 Laboratory investigations

The samples from the continous foil sampler were brought into the geotechnical laboratory at Chalmers University where they were investigated by staff from Chalmers and SGI. The samples were then photographed at various stages of drying to illustrate the stratification, Fig 17. The samples from the standard piston samplings were tested in the geotechnical laboratories at Chalmers University and SGI. The geologists at Chalmers University and the Geological Survey collected the specimens they required and made their tests in their respective laboratories.

The geotechnical laboratory tests comprised determination of natural water content, consistency limits, bulk density, resistivity, grain size distribution, undrained shear strength measured by fall cone and sensitivity. The preconsolidation pressure has been

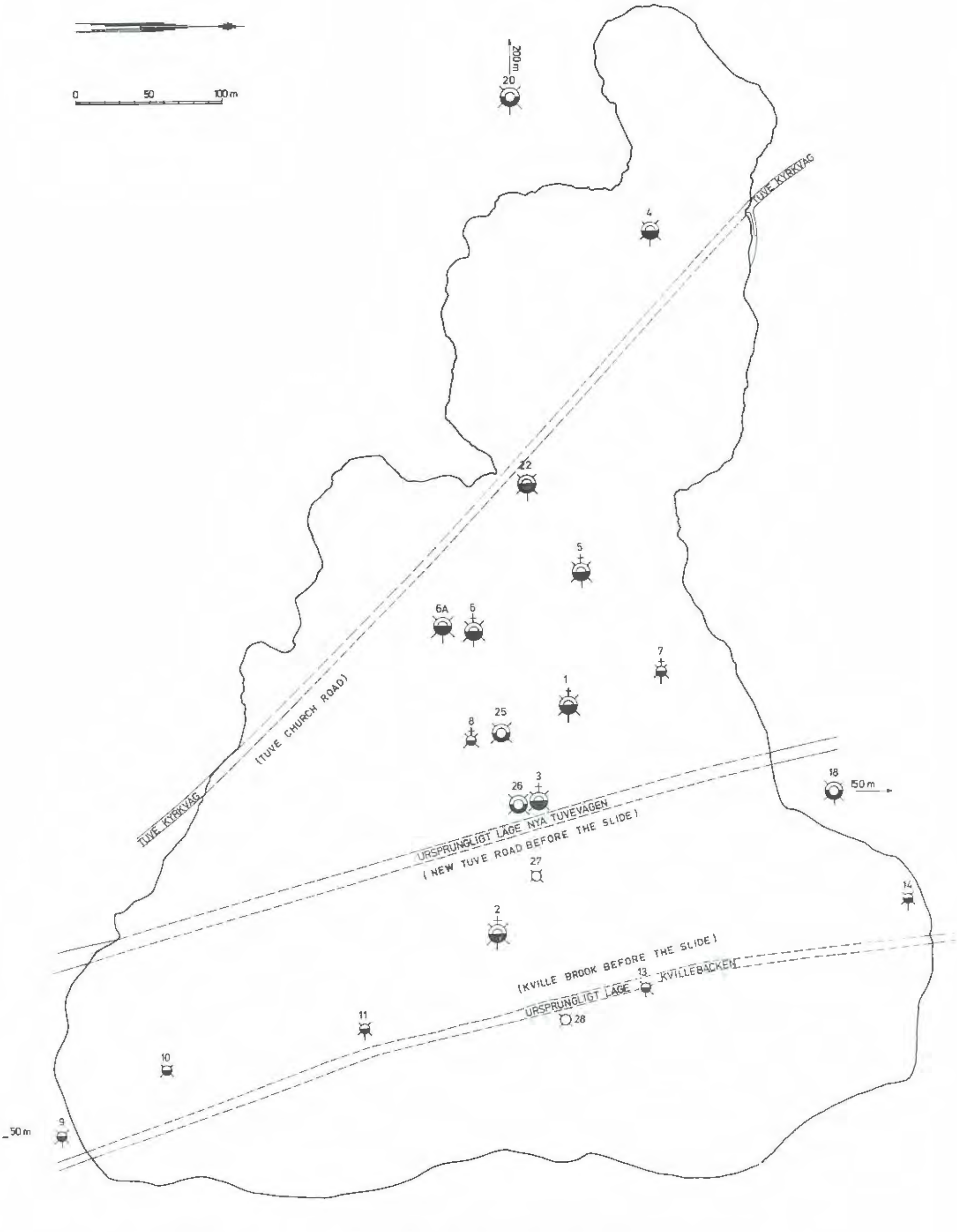


FIG 16. Plan showing the locations of SGI boreholes.

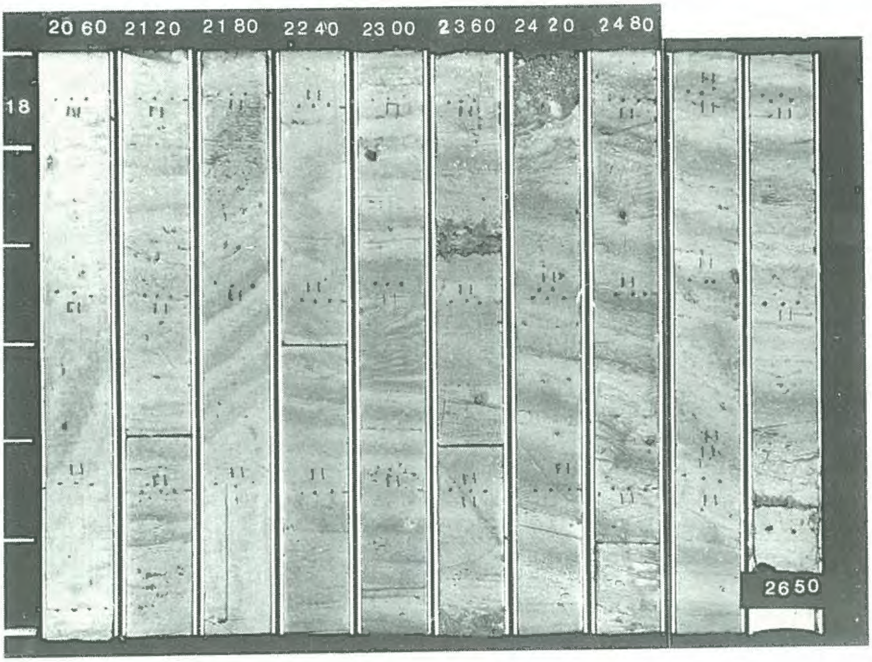


FIG 17a. Stratification in the lower part of core 18. Photograph taken at natural water content.

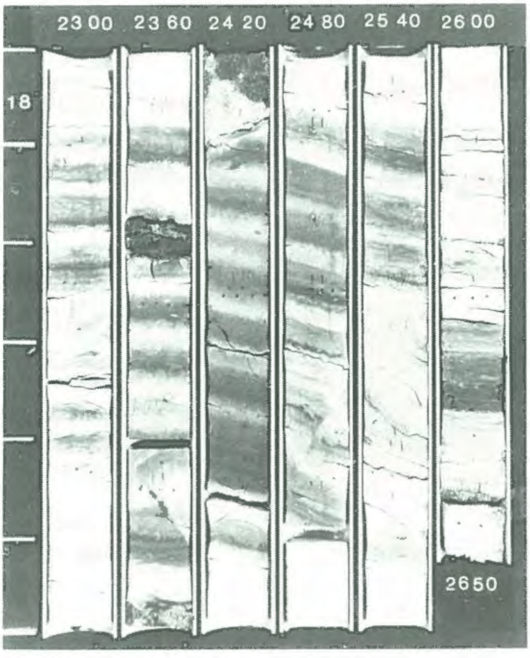


FIG 17b. Stratification in the lower part of core 18. Partly dried.

determined in oedometer tests with constant rate of strain. Drained and undrained shear strength has been tested in active and passive triaxial tests and direct simple shear tests. Special tests to illustrate the effect of increasing pore pressure and large shear strains on the shear strength have been performed. Some chemical and bacteriological investigations have also been made. The Norwegian Geotechnical Institute kindly offered to run a series of undrained tests according to their practice, which was gratefully accepted.

Results from the geotechnical laboratory investigations are shown in chapter 8 and Appendix B.

8. RESULTS FROM THE FIELD AND LABORATORY INVESTIGATIONS OF THE SLIDE

8.1 General limitations of the results

The results from all the investigations within the slide area are more or less affected by the slide. The soil has moved, broken up and in some parts become partly remoulded and mixed. The second stage of the investigation was not made until 15 months after the slide and reconsolidation and decrease in water content may have occurred in the deeper layers during this time.

However, the geological investigation indicates that the stratification in the area was very uniform and the picture emerging from the old investigations, the investigations within the slide area and the reference investigations outside the slide area is consistent. Within the slide masses there were also large blocks of soil up to 20 m thick which seemed to be relatively "unaffected" by the slide which helped to reconstruct the soil properties.

8.2 Topography

After the slide the upper part of the slide area sloped at an average of 1:40 while the lower part was almost horizontal. The topography of the firm ground and the present thickness of the soft soil have been determined mainly from penetration tests and weight soundings, old and new, Fig 18. The contour of the firm ground is steep in parts along Tuve Church Road and at Snarberget.

As a rule the upper part of the slide reached surrounding areas with firm ground. The thickness of the remaining clay in the western part of the slide area is 2-8 metres. The depth to firm bottom increases in an easterly direction towards New Tuve Road. In the central parts of the slide area the thickness of the soft soil is now about 20 metres and in the eastern parts it is about 40 metres.

The topography of the area before the slide has been determined from old maps and aerial photographs and is shown in Fig 19. Different cross sections before and after the slide are shown in Fig 20.

8.3 Ordinary soil properties

The investigations show that the clay in the upper part of the slide area and behind it is a varved clay which becomes more silty with depth and has silt and sand layers in the bottom. This type of material is found at the bottom of the soft soil in the whole area investigated. Further down the slope the varved clay is overlaid with a fairly homogeneous clay containing some shells. The upper metres contain some organic material and some infusion of clayey sand. Iron sulphides are found in different forms in the clay throughout the area.

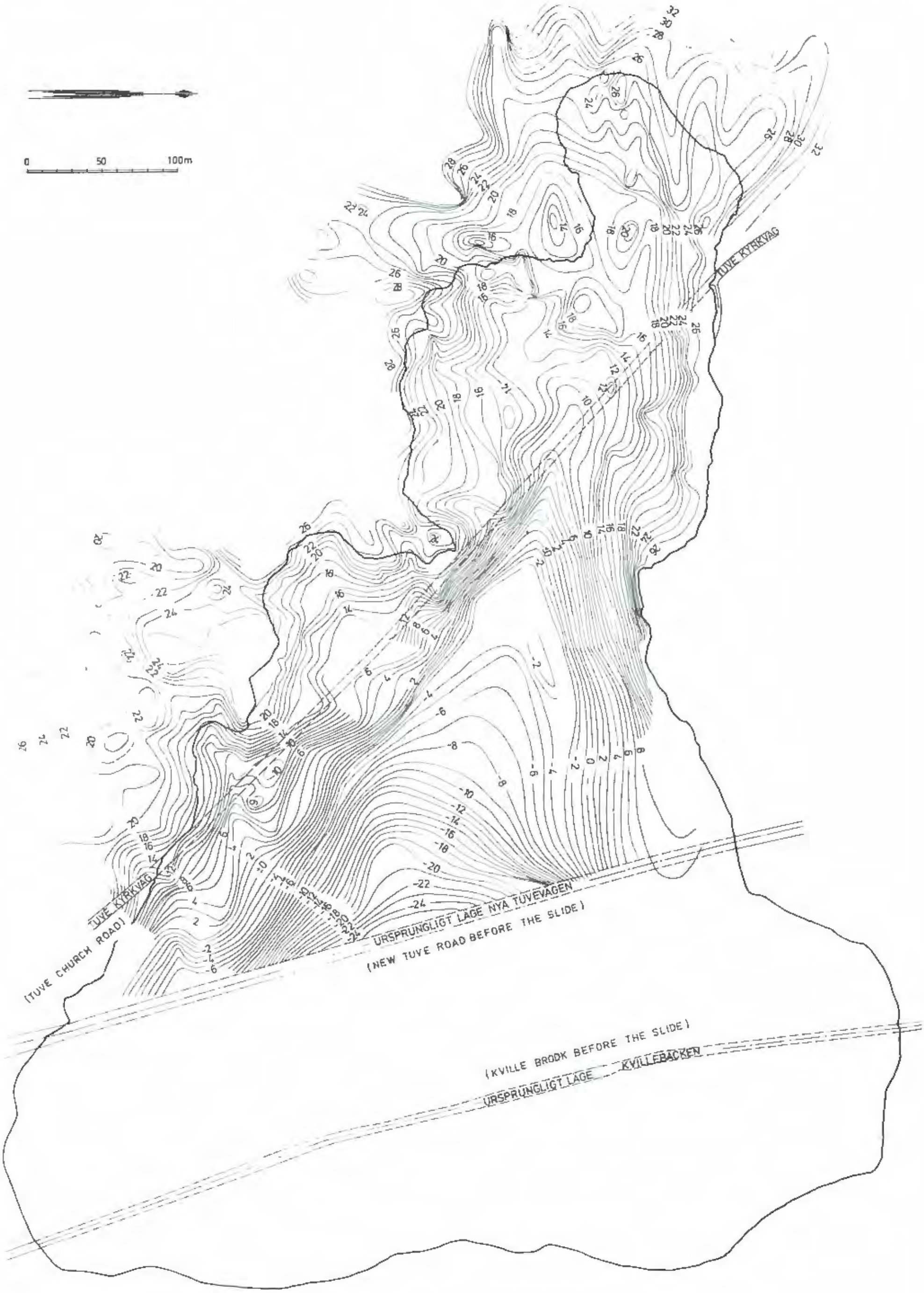


FIG 18. Topography of the firm ground at Tuve.

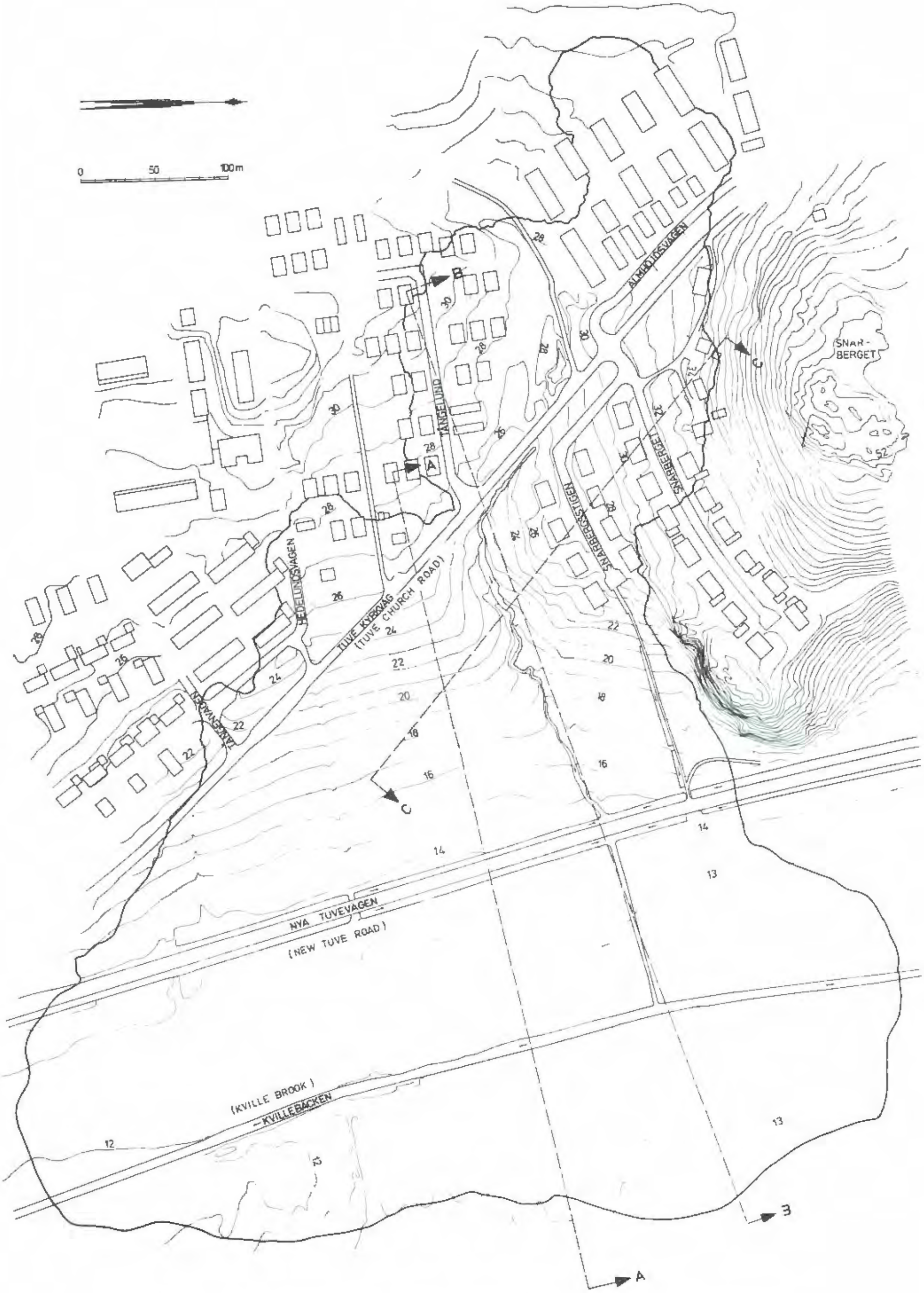


FIG 19. Topography at Tuve before the slide and location of cross sections.

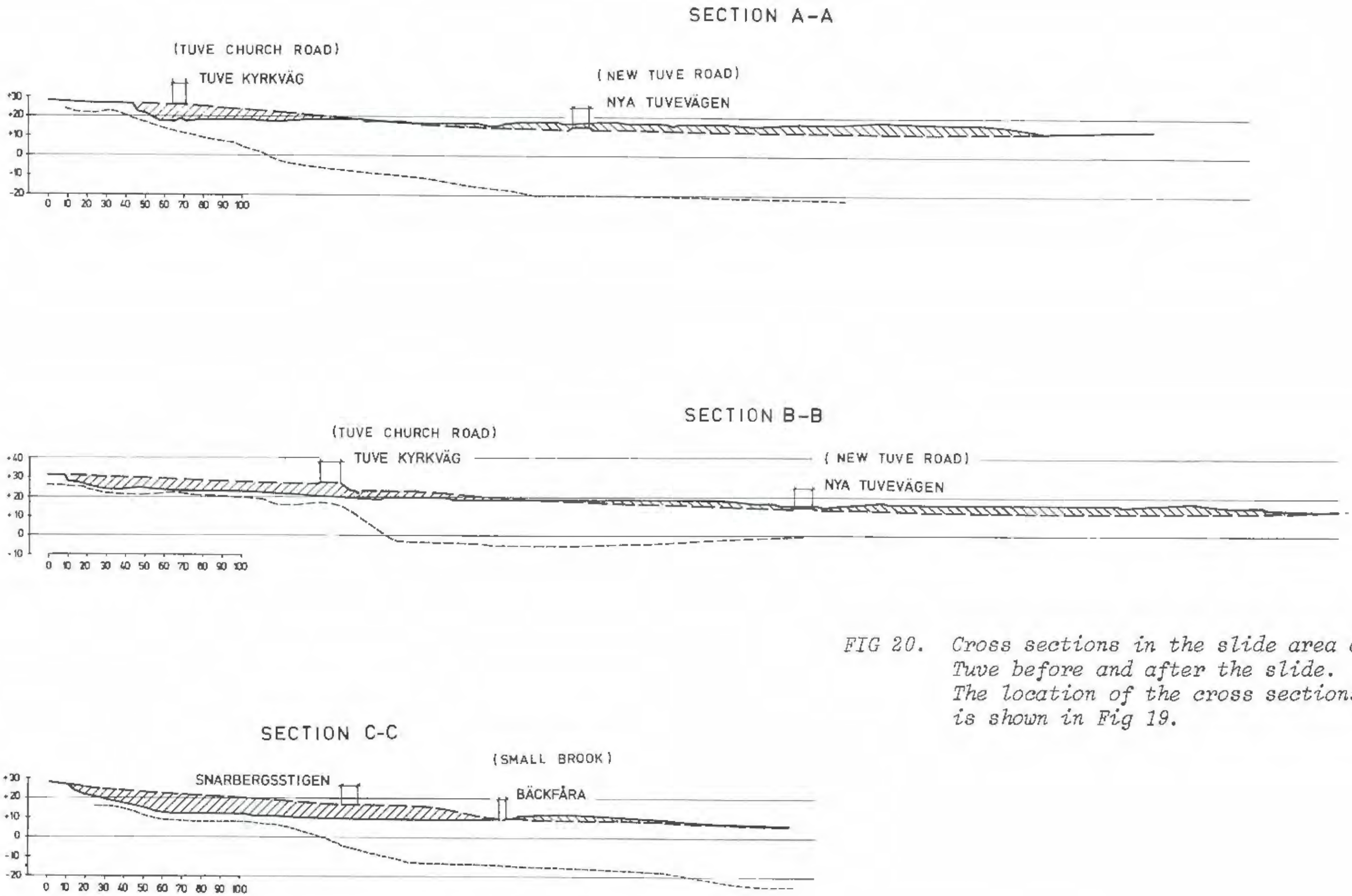


FIG 20. Cross sections in the slide area at Tuve before and after the slide. The location of the cross sections is shown in Fig 19.

The natural water content in the clay decreases with depth and normally varies between 70 and 40%. In the organic surfacial layers down in the Kville valley it reaches 120%. The liquid limit shows the same variation but is in general somewhat lower than the natural water content. At some points where the sensitivity has been high the liquid limit has been considerably lower than the natural water content. In the bottom layers of the clay the liquid limit drops to 30% or lower.

The plasticity limit of the clay varies between 40 and 20%.

The bulk density increases with depth from about 1.6 t/m^3 close to the ground surface to about 1.9 t/m^3 in the deepest layers.

The resistivity of the clay has been measured by inserted electrodes. There is some variation but nothing indicating layers with a pronounced lower salt content than the surrounding clay. Thus no indication of a slip surface has been found. The lowest resistivities indicating a high salt content have been measured on samples taken down in the Kville valley. Further up the slope the resistivity increases indicating a lower salt content. The measurements are consistent with the picture drawn by the geologists, chapter 3, and with the theory of leaching of the clay.

Furthermore the measurements of resistivity clearly indicate which borings have reached the coraser bottom layers.

The values of undrained shear strength obtained in the upper part of the slide area vary very much. Very low values alternate with quite normal values throughout the profile indicating that the clay is heavily broken up and kneaded by slide movements all the way down to firm bottom. (Borings 4, 5 and 22.)

In the lower parts of the slide area the situation is different. With the exception of the dry crust the values of undrained shear strength in this part increase almost linearly down to depths of 25 to 30 metres. Below this depth the shear strength radically decreases and stays on this low level down to firm bottom. The results indicate that the clay in the lower part of the slide area has been displaced in large blocks and that a zone of up to 10 m above firm ground has been subjected to heavy disturbance during the slide.

The sensitivity of the clay generally varies between 20 and 40 which is somewhat higher than normal for Swedish clays but nothing alarming.

There are some layers in borings 1, 5, 6 and 25 where the sensitivity varies between 50 and 220 but it should be observed that this soil had been displaced about one hundred metres and there is no indication of any greater disturbance in these layers. The highest sensitivities are found in borings 1 and 6. According to the measured displacements of the houses (Fig 12) the soil in borehole 1 originates from Snarbergsstigen and the soil in borehole 6 comes from Hedelundsvägen.

In the previous investigations from these areas sensitivites up to 50 and occasionally 80 have been found.

Some analyses to determine the grain size distribution and clay content have been made. They confirm the results obtained by the geologists, chapter 3.

8.4 Preconsolidation pressures

A number of oedometer tests with constant rate of strain have been run. This type of test has shown to give a reliable value of the preconsolidation pressure in soft clays (Sällfors, 1975). Tests have been made on samples from boreholes 3, 5, 6A, 22, 25 and 26.

The oedometer curves show that the clay in the upper part is so disturbed that the preconsolidation effect has disappeared and the earlier preconsolidation pressure cannot be evaluated. At a few levels in boreholes 6A and 22 reasonable preconsolidation pressures have been obtained, which shows that there are smaller blocks of moderately disturbed clay also in these parts of the slide area.

The conditions in the lower part of the slide area are different. Oedometer curves and preconsolidation pressures agree well with what could be expected from a fairly undisturbed normally consolidated clay. Tests on clay from this area have been run on samples from boreholes 3, 25 and 26. The tests on samples in borehole 26 taken below 26 metres gave oedometer curves typical for heavily disturbed clay and no preconsolidation pressures could be evaluated.

Preconsolidation pressures evaluated from tests on fairly undisturbed samples are shown in Fig 21. The results indicate that the clay below 7 metres was normally consolidated for a groundwater level 1 m below the ground surface. In wet seasons there was artesian water pressures in the bottom layers, which could reduce the effective stresses by 20% at the most. Observations of groundwater fluctuations, however, indicate that there was a large variation of groundwater level in these layers and the temporary high water pressures should not have affected the preconsolidation pressures significantly.

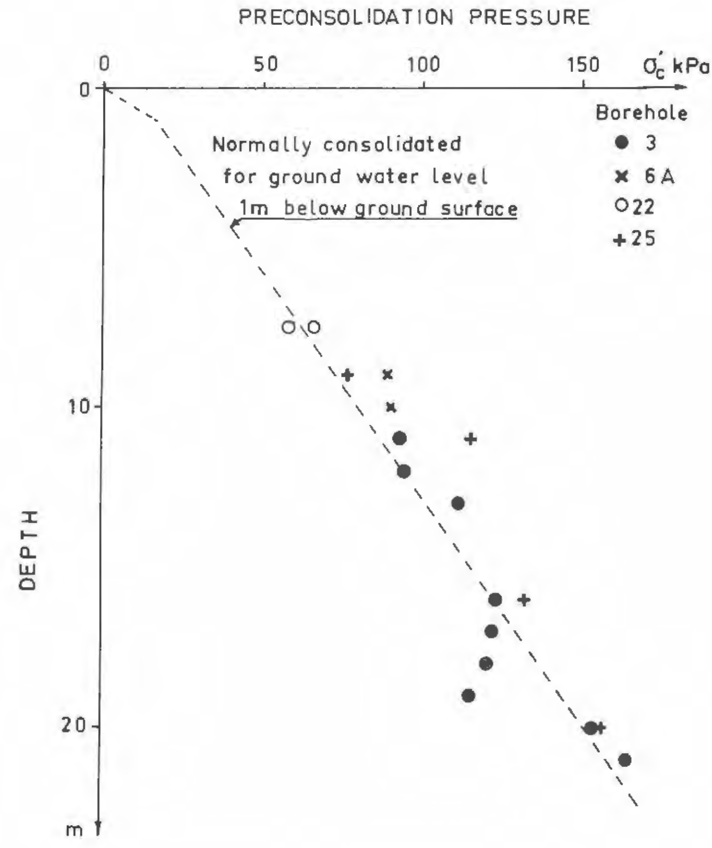


FIG 21. Preconsolidation pressures in the clay at Tuve.

8.5 Shear strength

The values of undrained shear strength measured within the slide area are more or less affected by the slide. Most determinations were made about 2 months after the slide. At that time the pore pressures in the soil were still very high and the effective stresses in the slide masses were low, at many points almost zero.

Investigations by Ladd and Foot (1974) have shown how the undrained shear strength decreases in clay when the pore pressure rises, Fig 22. This happens even if the clay remains undisturbed. Disturbance such as kneading and partial remoulding further decreases the undrained shear strength.

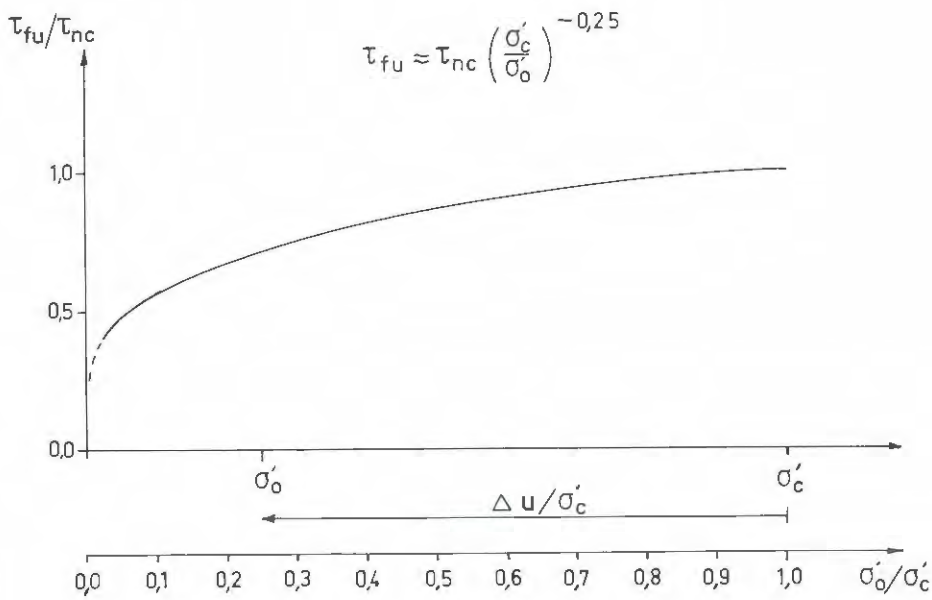


FIG 22. Decrease in undrained shear strength of clay after a rise in pore pressure. Replotted from Ladd and Foot, 1974.

There are, however, empirical relations for how the undrained shear strength measured by the field vane test or the fall cone test varies with the preconsolidation pressure and the liquid limit in Scandinavian clays. The relation proposed by Hansbo (1957), $\tau_{VANE} = \sigma'_c \cdot 0,45 \cdot w_L$, has been found to give good estimates of the undrained shear strength in normally consolidated or slightly overconsolidated marine clays in western Sweden.

Hansbo's relation has been checked against the undrained shear strength measured by field vane tests and fall cone tests in boreholse 18 and 20 which were located outside the slide area, Fig 23. The comparison shows that the empirical relation seems to be largely valid for the clays in Tuve and nothing unusual about the undrained shear strengths from that aspect was found.

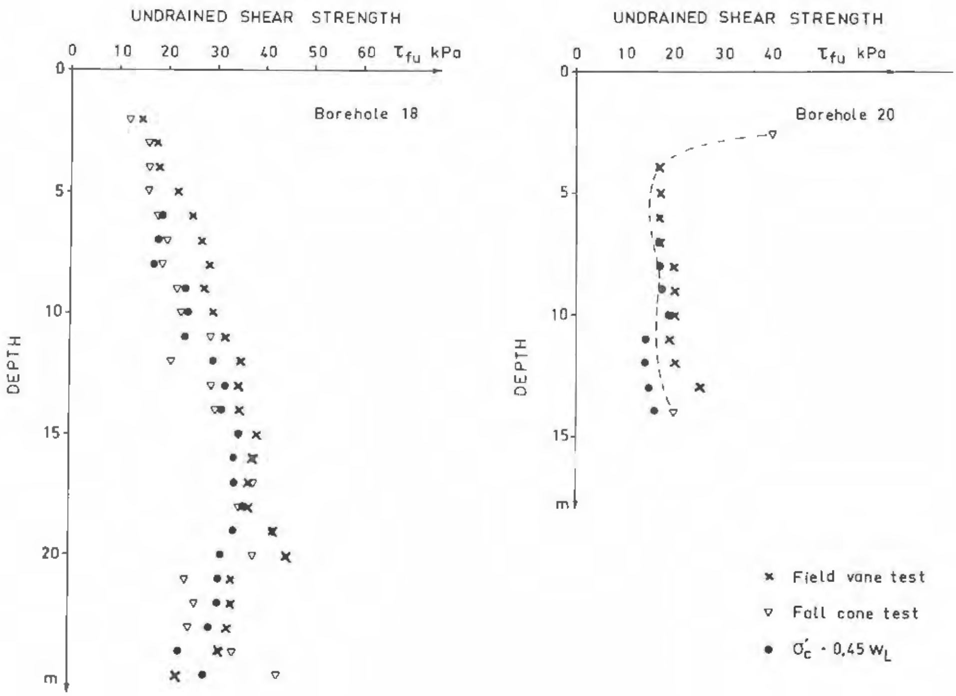


FIG 23. Undrained shear strength measured by field vane and fall cone in undisturbed clay from the Tuve area compared to Hansbo's (1957) empirical relation.

The field vane tests were performed at two different times, 2 to 6 months and 15 months after the slide. Many of the first borings were not carried out all the way to firm bottom and when it was later realized how deep the slide really went supplementary borings were made. In one of the first borings, number 3, the vane tests were continued all the way down but the sampling stopped at a higher level. In the supplementary boring number 26, close by boring number 3, new vane tests and sampling were carried out down to firm bottom. There are thus vane borings in two adjacent boreholes made 6 months and 15 months after the slide, Fig 24.

The undrained shear strength 6 months after the slide in most of the profile was about two thirds of what could have been expected from the empirical relation. In the lower layers it was less than half. In the new tests thirteen months later the undrained shear strength was unchanged in the upper part of the profile but had increased considerably in the lower layers. In this part the undrained shear strength approaches the values calculated with empirical relations and the new measured groundwater level in the sand layers at the bottom of the clay. 6 months after the slide some reconsolidation may be expected already to have occurred in these layers and the results of the vane tests 15 months after the slide certainly indicate reconsolidation. Concerning the other properties of the clay a reconsolidation would mainly have affected the water content, which may be expected to have been higher in these layers before the slide than was measured later.

Undrained triaxial and direct simple shear tests were carried out at the Norwegian Geotechnical Institute and at SGI.

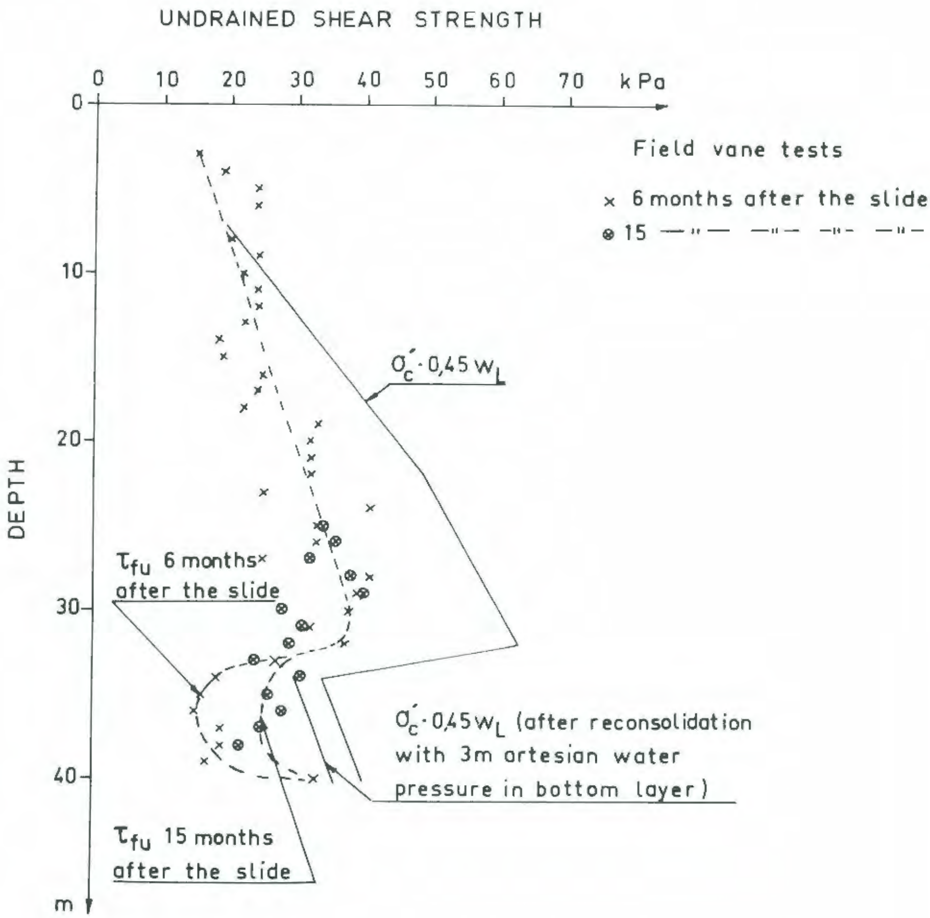


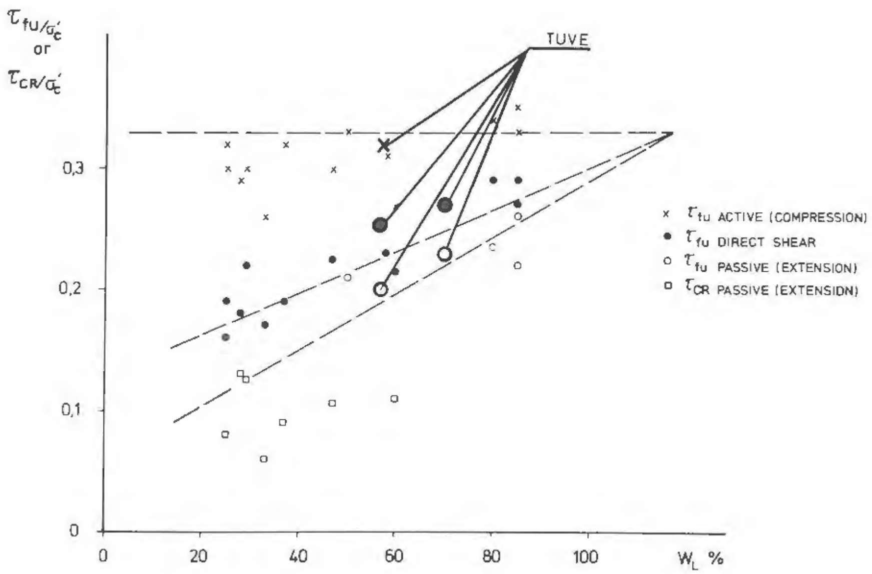
FIG 24. Field vane tests in boreholes 3 and 26 made 6 and 15 months after the slide compared to empirical relations.

Also for these types of tests there are empirical relations between the undrained shear strength measured in the different types of tests, the preconsolidation pressure and the plasticity of the soil (Larsson, 1977 and 1980). The results from tests on clay from Tuve conform to these empirical relations, Fig 25.

The undrained shear strength in Tuve thus seems to conform to what could be expected considering the preconsolidation effects and the plasticity of the soil. The main thing unusual about Tuve, although not exceptional, is the very rapid decrease in plasticity with depth and the loose layers close to the bottom. This condition makes the undrained shear strength profile deviate from the normal pattern where the undrained shear strength increases with depth. Instead the undrained shear strength at Tuve decreases below a certain level in the profiles and becomes a minimum in the low plastic silty layers close to firm bottom.

The rapid decrease in plasticity with depth in the lower parts of the profiles is probably partly due to leaching of salts caused by contact with fresh water in the permeable bottom layers and artesian water pressures in these. Leaching may, apart from decreasing the plasticity, also cause an increase in sensitivity and a decrease in undrained shear strength.

The undrained shear strengths in the deeper layers are in this case of particular interest as the slide surface or zone seems to have been located here. There are, however, particular problems in sampling and handling very soft silty clays/clayey silts and so far the number of cases where really undisturbed samples of this kind have been tested in the laboratory is limited.



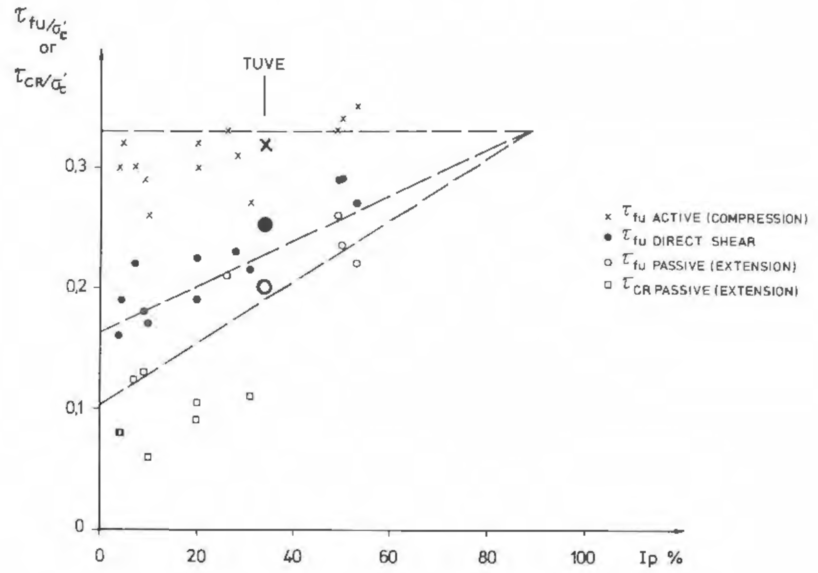


FIG 25. Undrained shear strengths from triaxial and direct simple shear tests on clay from Tuve compared to normal results from other Scandinavian clays. (τ_{CR} represents the shear stress mobilized at a deformation corresponding to the failure deformation in the compression test, which is why the peak undrained shear strength in extension is somewhat higher.)

A special effort to investigate this type of material was made by the Norwegian Geotechnical Institute after the slide at Furre 1959 (Bjerrum et al, 1960). From these investigations it emerged that the shear strength in low plastic soils could be as low as 0.12 times the preconsolidation pressure (Bjerrum, 1961). This low value is further supported by the investigation of the slide at Bekkelaget (Eide, 1955).

Börgesson (1981) in his investigation of properties of inorganic silts illustrated how the undrained shear strength in simple direct shear tests decreases with decreasing clay content. The undrained shear strength in rather loose clayey silts was found to be of the order of 0.13 σ_c' .

The empirical relation for undrained shear strength measured by the vane test also gives minimum values around 0.12 σ_c' and strength values of this order were measured in the bottom layers at Tuve.

Some drained triaxial tests and direct shear tests were run to investigate the shear strength at low effective stresses under drained conditions. Empirically the friction angle for Scandinavian clays and silts without dilating effects is about 30^0 (Börgesson, 1981, Kenney, 1967, Larsson 1977 and 1981). The tests on high plastic clays and on silty clays from Tuve also gave friction angles close to 30^0.

Some special tests to illustrate the effect of rising pore pressures in a soil subjected to shear stresses have been run. In a triaxial test a sample was reconsolidated to *in situ* stresses with a vertical stress of 150 kPa, a horizontal stress of 100 kPa and a pore pressure of 50 kPa. The pore pressure was then raised in small steps and the sample was allowed to absorb water. The result is shown in Fig 26. Very small deformations occurred until an effective angle of 30 was mobilized. Then the deformations increased and at a slightly higher effective stress relation the sample failed. Identical results have been obtained in direct simple shear tests in connection with other investigations.

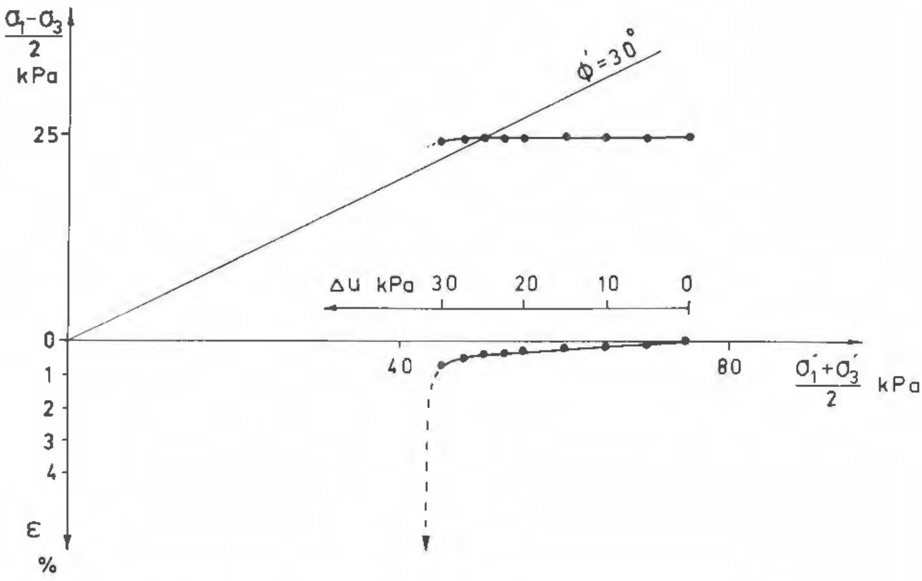


FIG 26. Result from a triaxial test where the sample has first been reconsolidated to in situ stresses and then subjected to a rise in pore pressure.

8.6 Sensitivity and rapidity

The sensitivity of a clay is generally assumed to play an important role in the evaluation of risks in connection with landslides. The risk of progression and retrogression of slides and the influence of vibration on the strength properties is often connected with sensitivity.

To become highly sensitive a soil has to have a natural water content higher than the liquid limit and thereby a liquidity index higher than one. This was the case for almost all soil strata in Tuve and the sensitivity generally varied between 20 and 40, which might seem high in an international perspective but is not at all unusual for Scandinavian clays. As mentioned earlier there were some layers in the middle of the clay profile in the area at Snarbergsstigen and Hedelundsvägen where the sensitivity varied between 50 and 220 but the clay in these layers remained largely intact during the slide.

In the two boreholes taken outside the slide area the sensitivity increased with depth and was found to vary between 50 and 70 in the low plastic bottom layers. This higher sensitivity in the bottom layers has not been found within the slide area but it should be considered that these layers lost most of their undrained shear strength during the slide and probably has become a reduced water content due to reconsolidation.

Clays with a high salt content and clays with high plasticity are normally not very sensitive. Leaching can decrease the plasticity of clays and thereby increase their liquidity index and sensitivity. There are also indications that some chemicals such as cleaning agents (tensides) in waste water might cause changes in the clay. In Tuve the highest sensitivities are generally found in the upper parts of the slope and in the bottom layers while the lowest are found in the high plastic clays in the lower parts of the slope, which is also consistent with the measured salt contents.

In later years investigators have tried to find a concept other than sensitivity to describe the breakdown of undrained shear strength during disturbance of the clay.

In 1955 Kjellman pointed out that it normally does not matter very much if a clay is quick and loses 98% of its original strength or if it is a medium to low sensitive clay and loses 90% of its strength at complete remoulding. The main questions usually are how much disturbance is required to break down the strength by a certain degree or how much the strength is reduced by a certain disturbance.

In 1974 Söderblom proposed a new concept termed rapidity to classify the breakdown of clays after a certain amount of pounding. The method is very little used as it is somewhat subjective and is not applicable for stiffer clays. Another method specially designed to determine the effect of piledriving on soil adjacent to the pile was suggested by Massarsch 1976. In 1982 Tavenas et al presented results from an investigation on Canadian clays aiming at determining what properties of a soil did increase the risk of retrogression of landslides. They proposed a method where the rate of decrease in undrained shear strength during cyclic direct shear tests was used to determine a remoulding index. The investigators concluded that low plasticity was a better indicator of rapidity than sensitivity or liquidity index.

A similar study has been made on Swedish soils at SGI. In this study samples were consolidated for the *in situ* effective vertical stress and then sheared back and forth under undrained conditions in a direct simple shear apparatus. The shear strain was an angular rotation starting at 0 and then going from +0.15 radians to -0.15 radians. A full stroke was completed in 10 seconds and the stress and strain relations were recorded continously. The highest shear stress in each cycle was then plotted against the number of strokes. The peak shear strength in clays is highly rate-dependent and to obtain a reference strength parallel direct simple shear tests were run at the standard testing rate. The peak strengths in the cyclic tests were 1.35 to 1.85 times the undrained shear strength in standard tests with an average of 1.55 times.

The tests were evaluated in terms of how many cycles were required to reduce the undrained shear strength to half the undrained shear strength measured in standard tests. The results were plotted against liquid limit, sensitivity and liquidity index. Relations could be obtained for all of them and rapiidity seems to be dependent on not one but all of them. In soft clays a decreasing plasticity is often accompanied by an increase in liquidity index and sensitivity and this was also generally true for the tested clays, Swedish as well as Canadian. The results from the Swedish and Canadian tests are plotted in Fig 27. The number of cycles for 50% strength reduction or remoulding index $I_R = (\tau_{fu}-\tau)/(\tau_{fu}-\tau_{REMOULDED})$, where I_R is evaluated after an energy input of 40 times the energy required to achieve initial failure, are plotted against liquid limit. There is a clear trend that clays become more sensitive to a limited disturbance the less plastic they are. There are some examples of medium plastic clays which are relatively easily broken down and they also have a very high sensitivity. Since, on the other hand, the sample with the highest sensitivity >600 followed the general trend there must be other factors than sensitivity playing a role. All the samples which were relatively easily broken down ($I_R > 70$ or $n < 15$) had a high sensitivity.
It thus seems that the materials most sensitive to a limited disturbance are also highly sensitive in the ordinary terminology although there is no direct correlation. Low plastic clays and silts are more likely to be sensitive to a limited disturbance than soils with a higher plasticity.

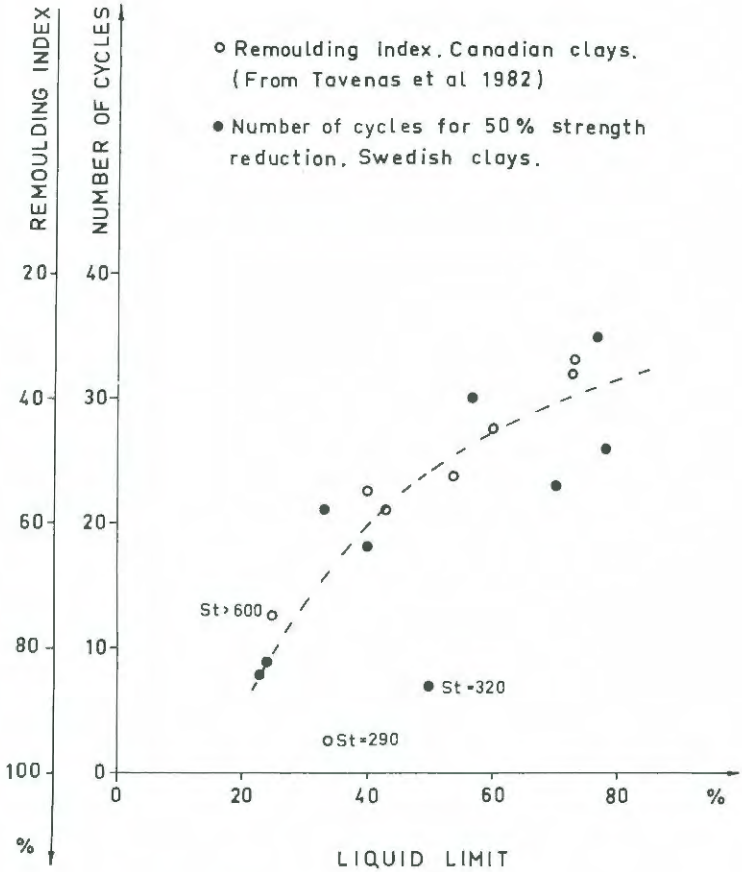


FIG 27. Results from Canadian (Tavenas et al, 1982) and Swedish tests to determine the sensitivity of different clays for limited deformations.

It should be observed though that even the most sensitive soils require a considerable amount of disturbance to lose half their undrained shear strength. Normal testing rates in Scandinavia are chosen to eliminate peak effects due to the rate. For Canadian clays where this is hardly possible, the undrained shear strength to be used in calculation is evaluated from slow tests as post peak strength or strengt at large strains where the shear strength is only slowly decreasing with strain (Lefebvre, 1982, Trak et al, 1980). The deformation required in the most sensitive samples to reduce the undrained shear strength to half the value from standard testing was about one hundred times the deformation required to reach failure in a standard test.

8.7 Chemical and bacteriological investigations

Quantitative analyses have been made on ten samples from the bottom layers in boreholes 25 (depth 31-35 m) and 26 (depth 39-43 m) to find the amount of methylene-blue active substance. The contents were found to vary between 0.02 ppm to 0.18 ppm which means that the amount of surface active substances (e.g. tensides) in the samples was very small and thereby also an eventual content of active waste water. A bacteriological analysis of the groundwater in the slide area shows a total content of bacteria of less than 5 per ml and a content of coliform bacteria of less than 2 per 100 ml. Also this investigation indicates that there is no or very little waste water in the area.

The Geological Survey of Sweden has made comprehensive chemical analyses on soil from boreholes 18 and 20. The results are reported in chapter 3 and SGI report No 11b.

8.8 Pore pressures after the slide

Measurements of the pore pressure were commenced about a month after the slide. The pore pressures were then found to be very high and in the upper parts of the slide area they were almost equal to the new overburden pressure. The effective stresses in these parts were thus almost zero. This condition prevailed in the larger part of the profiles. Also in the lower parts of the area considerable overpressures in relation to the hydrostatic pore water pressure were measured but the pore pressures in these parts did not go as high as the total overburden pressure.

The pore pressures in the permeable bottom layers corresponded in January 1978 largely to a groundwater level in the new ground surface for large parts of the slide area. Below the Almhöjdarea pore water pressures corresponding to a groundwater level 2 metres above the new ground surface were measured in the bottom layers. Also in some of the lower parts of the slide area artesian water pressures were measured in the bottom layers. Here they corresponded to a groundwater level about 3 metres above the new ground surface.

The continued pore pressure measurements show that the high pore pressures in the slide area generally dissipate very slowly. In areas where vertical drains have been installed, however, the pore pressures have been considerably reduced. Measurements in the bottom layers show that the pore pressure levels measured a month after the slide were largely unchanged. The artesian water pressures in the lower part of the slide area were also the same. In general, there has only been a local reduction of the pore pressures in those areas where steps have been taken to lower it (e.g. by installation of drains or lime columns).

Water pressures measured in the bottom layers in December 1979 are shown in Fig 28.

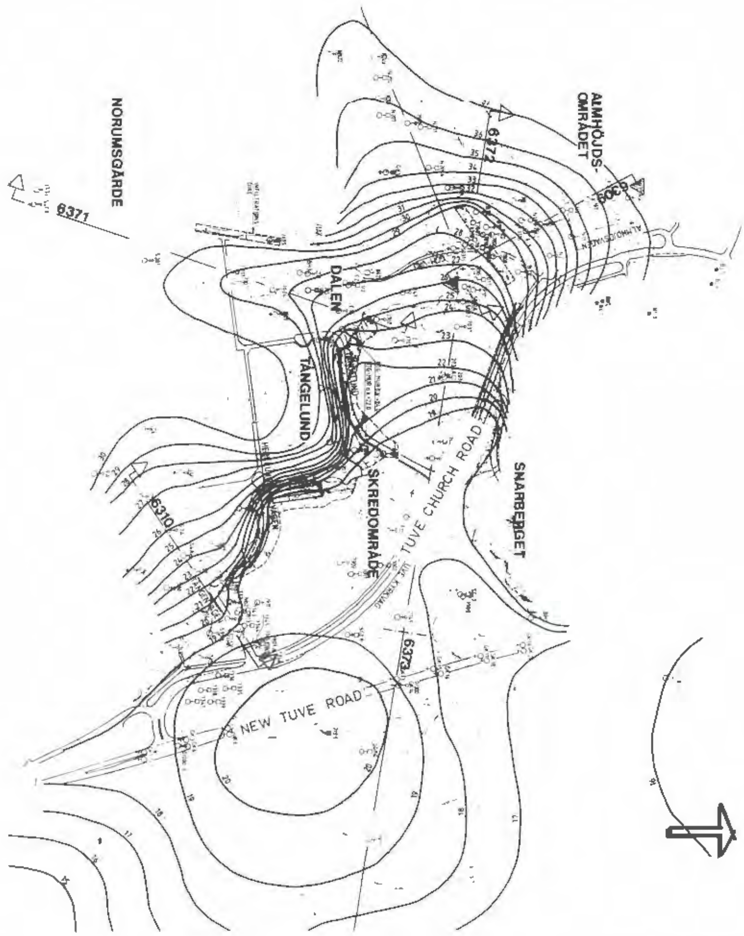


FIG 28. Groundwater levels in the bottom layers in December 1979. (Blomqvist and Gustavsson, 1981)

8.9 Soundings with pore pressure probe

Soundings with pore pressure probe are normally carried out to investigate the soil stratification and to find layers with higher permeability than the surrounding soil. Combined with measurements of tip resistance they can give a very good picture of the soil stratification.

Experience of soundings with pore pressure probes in slide masses of partly disturbed and remoulded soil is limited and it is therefore difficult to interpret the results. Within the masses large blocks of seemingly unaffected clay have been found. The pore pressure generated during the penetration of the probe has dropped here and there in the profiles but no continuous layers with abnormal properties or discontinuities indicating a slip surface passing through a larger area have been found.

The results indicate that the whole soil mass has been involved in the slide and disturbances are found all the way down to firm bottom.

8.10 Soundings with the salt probe

A special probe, the salt probe, which measures the resistivity of the soil has been used in previous investigations to find variations in salt content and slip surfaces. A slip surface can show up as a discontinuity in the resistivity of the soil (Söderblom, 1969). The salt probe was used at Tuve and generally confirmed the distribution of salt contents found in the laboratory investigations. No clear discontinuity or layers with differing electrical properties were found.

8.11 The slide surface

It was at first assumed that the slide surface or sliding zone was located somewhere in the clay layers and a number of the investigations were aimed at finding out exactly where.

The first samplings and also some of the other investigations were stopped at a depth of 25 m in the passive zone. As the investigation progressed it became evident that the whole soil mass all the way down to the coarser bottom layers had been involved in the slide. In the upper part of the slide aera there were numerous slip surfaces within the clay mass and all soil except a layer of a metre or two on top of the bedrock had been involved. In the middle part of the area the upper part of the soil was broken up in large glocks and below them the soil seems to have been remoulded in a large zone that reaches down to firm bottom. In the lowest part of the slide area there were very large blocks of relatively undisturbed soil, up to 25 m deep, on top of a partly remoulded zone of soil which was up to 10 m thick. In the passive zone a series of what seemed to be forward sloping slices could be observed from the surface.

The sliding zone seems to have followed the contour of the bedrock. It is impossible to say how thick the actual sliding zone was and how much of the final disturbance was an effect of kneading when large soil masses were transported long distances over uneven bedrock. The heavily disturbed zone comprises the layers of low plastic silty clays and the zone with silt and sand layers in the clay.

9. GEOTECHNICAL CONDITIONS AT TUVE BEFORE THE SLIDE

For evaluation of the geotechnical conditions at Tuve before the slide the information gathered from earlier investigations and geotechnical, geological and geohydrological investigations after the slide has been considered together with empirical relations and general experience from geotechnical conditions in western Sweden.

The following picture of the probable conditions before the slide has emerged:

The side valley of the Kville valley and the part of the Kville valley in which the slide took place were filled with a fairly homogeneous marine clay. The upper metres had some organic content and some infusion of sand and silt. The clay content decreased with depth and so did the plasticity. Below the homogeneous clay there was a thick layer of soft silty clay and clay with loose silt and sand layers. The thickness of this layer varied between 3 and 10 metres and is estimated to have been 4 to 5 m in most of the area. The total thickness of the soft layers increased from 14 m in the upper part of the side valley to about 40 m at the middle of the Kville valley. On top of the bedrock there was a permeable layer of sand.

Along the sides of the valley the bedrock was exposed. The covering clay reached some way up the sides and enabled a limited artesian water pressure to develop in the permeable sand layer.

In the middle of the side valley there was a small brook which had eroded a small gully with locally steep banks.

The clay was normally consolidated for a groundwater level about a metre below the ground surface except for the dry crust and a limited area around the small brook where it was slightly overconsolidated.

The water content in the upper organic clay could be as high as 120% but mostly decreased from about 70% in the upper clay layers to about 40% in the bottom layers of silty clay. The liquid limit was in general about 10% lower than the natural water content. The plastic limit varied between 40 and 20%. Large liquidity indices existed in layers with high sensitivity.

The bulk density increased from 1.6 t/m^3 in the upper layers to 1.9 t/m^3 at the bottom.

The undrained shear strength of the clay largely follows empirical relations for how undrained shear strength varies with preconsolidation pressure and plasticity. With the stratification and preconsolidation found in Tuve this resulted in a shear strength as measured by the field vane of about 15 kPa down to about 5 m depth. From this depth the undrained shear strength slowly increased with depth but from a distance from the firm bottom of about one third of the thickness of the clay layer it decreased again. The undrained shear strength had a minimum in the silty clay and the clay with layers of loose silt and sand at the bottom of the clay profile.

The sensitivity of the homogeneous clay generally ranged between 20 and 40. The least sensitive clay was down in the Kville valley and the most sensitive was in the upper part of the side valley. There were layers of quick clay in the homogeneous clay in areas close to Tuve Church Road.

The silty bottom layers in the whole aera had a high sensitivity between 50 and 70. These layers also had the best prerequisites for high rapidity which means that they probably required relatively limited deformations for their strengths to be considerably reduced.

The overall inclination of the ground surface was about 1:30. There were local areas below Snarbergsstigen where the inclination was 1:5 and locally in the gully banks the inclination rose to 1:3.

The bedrock below the clay and the overlying permeable layer had steep inclinations at Snarberget and below

Tuve Church Road. The groundwater basin formed by the permeable layer was concave and had a limited capacity. It could be filled after periods of heavy rain as in the period before the slide and then brimmed over at the sides where the bedrock and permeable layers came to surface. Further rainwater then had to be discharged at the ground surface. As the clay cover reached some way up the valley sides an artesian water pressure corresponding in general to a groundwater level from 2 m above the ground level to 4 m above at the lowest points could occur. The period before the slide occurred was the worst period from the precipitation point of view since the final development of the area seven years before.

Development of the area was carried out during 1957-1970 and comprised the building of about 300 one family houses of different kinds in the upper parts of the side valley. Roads were constructed and pipes laid. A sewer was constructed along the brook. The upper part of the brook was laid in a conduit and the gully was filled in along the highest parts of the valley. On top of this a fill was placed in the same area to ensure local stability for the houses in the Almhöjdarea. Behind the valley itself large development programs were carried out but it is uncertain whether these affected the situation in the side valley in any way other than the waste waters and storm waters from these areas were discharged in the sewer and brook respectively.

10. CONDITIONS IN AREAS WHERE LANDSLIDES IN SOFT CLAY FREQUENTLY OCCUR

10.1 General

Numerous landslides have taken place in Norway and in one region over 500 scars from slides have been identified (Löken et al, 1970). Large landslides seem to occur at intervals of about 3 years (Helland, 1896, Jörstad, 1968). In Sweden the reoccurrence of landslides in soft clay with an area of more than one hectare is one every two or three years and the rate has been increasing during the last century (Viberg, 1982). Landslides in the Champlain clay in Canada are common. More than 700 slides are known in the St Lawrence river valley alone (Eden & Mitchell, 1973).

The geological conditions during and just after the last glaciation about 6,000-14,000 years ago have created very large and often very deep deposits of soft clays in the Scandinavian countries and in Canada. Scandinavia and large parts of Canada and northern USA were pressed down below sea level under the weight of the glaciers. After the deglaciation the clays deposited in the sea gradually emersed due to the isostatic uplift.

In the southwestern part of Sweden the bottom layers below the clay are often permeable layers of silt and sand and such layers sometimes exist inserted in the clay profile. Due to the topography the water pressures in these permeable layers are often artesian.

In Scandinavia three factors have essentially been the main cause of the creation of soil conditions where landslides occur, namely, the isostatic uplift, erosion and the reduction of shear strength and increase of sensitivity due to leaching of salts from the clay. The effective weight of the soil has been increased by the lowered groundwater level. Due to the settlements the ground surface has formed an increased slope in areas with variations in the thickness of the clay layer.

In valleys there is usually a brook or a river at the toe of the slope slowly eroding itself down into the slope.

The soft clays in Scandinavia are generally normally consolidated or slightly overconsolidated while the Leda or Champlain clays in Canada are generally overconsolidated. These Canadian clays therefore normally have a considerably higher strength than the Scandinavian soft clays.

Most of the landslides in the Champlain or Leda clays start as rather superficial slides in steep river banks due to a combination of erosion and high pore pressures following melting snow or heavy rainfalls. These initial slides may spread over large areas due to retrogression.

This type of slide is also common in Scandinavia but due to the more complex stratification, the generally lower strength and a different variation of strength with depth there are many other types of slides, such as flake-type slides, slides progressing forward and often slides caused by minor man-made changes in the stress situation.

10.2 Topography

The first prerequisite for a landslide to occur is generally that the ground surface is inclined. There are cases in Sweden where large volumes of soil have been displaced sideways even when the ground was rather flat but these displacements have been limited to 0.2-0.3 m and are not quite landslides. One exception is the landslide at Fröland 1973 (Bjurström and Broms, 1982). This slide is assumed to have started due to liquefaction of loose silt.

Attempts are being made to classify the stability conditions of slopes where the inclination of the ground surface is a major factor. However, the failing slopes in Canada are usually rather steep river banks, the stability of natural slopes in Norway is considered if the inclination is more than 1:15 while the limit in Sweden is proposed to be as low as 1:50. Inclination of the ground surface must therefore be seen in relation to the overconsolidation ratio, existence of permeable layers with high pore pressures and other factors to give an indication of the risk of initial slides. During recent years the topography of the bedrock and of permeable layers has been taken into account. The inclination of the bedrock in western Sweden is often very steep and the surface of the bedrock and the closest overlying layer is often pervious. If the water pressure in this layer is high enough the soil mass overlying the steep section is not stable in itself but leans on the soil masses beyond the toe of this slope in the bedrock for support. Models for calculation of stability and leaning forces from such soil masses have been presented by Broms 1978 and Sällfors 1979. Similar studies have been made on how high the pore pressures in inclined permeable layers within the clay may be before they start to affect the stability of the ground (Massarsch, 1979).

The topography of the surrounding areas is also important as it determines the catchment area for the rainwater that will de discharged through the slope area and the prerequisites for artesian water pressures to develop in permeable layers (Viberg, 1982).

10.3 Factors initiating landslides

The main natural factors initiating landslides are rainfall, snow-melting and erosion. Rainfalls and snow-melting result in high pore pressures in permeable layers, increase of the bulk density of the dry crust, waterfilling of cracks and with time a decrease of the shear strength in low permeable layers. In overconsolidated clay many cycles of high pore pressures may be required to reduce the strength. During this process the clay gradually absorbs water and creep movements, cracks and fissures occur.

In normally consolidated clays the rise in pore pressure in surrounding layers has to be of a certain duration to penetrate into the low permeable clay layers. The effects do not accumulate for many years as they will be counteracted in dryer periods. The effects of high water pressures in normally consolidated clays are thus confined to zones relatively close to draining layers or cracks but these zones increase in size with duration of the high water pressures. Considering the anisotropy in Scandinavian clays the increase in pore pressure in a normally consolidated clay has to be considerable to have an effect on the shear strength anywhere other than in the active zone (Larsson, 1982).

Statistics have shown that the frequency of landslides is highest in the snow-melting period in spring in the Champlain clay area in Quebec and northern Scandinavia and in the rainy autumn months in southern Scandinavia, Fig 29.

When trying to analyze the influence of rainfall a number of factors have to be taken into account, such as the amount of rainfall during the previous day, week and half year, the evaporation and discharge during that period and possibly very dry periods when the ground surface might have cracked up before the wet season started.

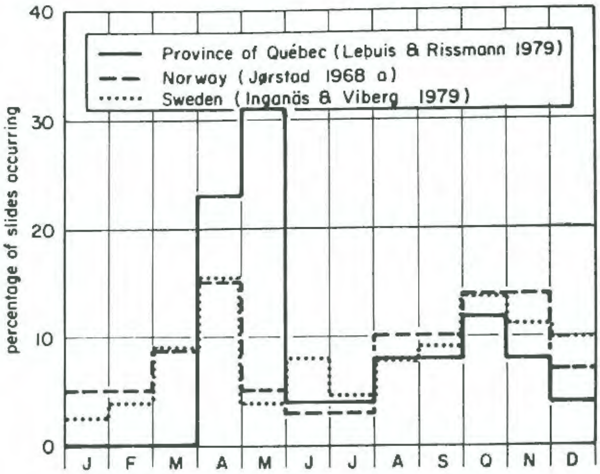


FIG 29. Slide frequency versus month of the year in Quebec, Norway and Sweden (from Tavenas and Leroueil, 1981).

In analyzing the effect of snow-melting not only the amount of snow but also factors such as the time for melting, whether the ground below the snow is frozen or not and the groundwater level before the snow-melting have to be taken into account (Rosén, 1982).

Erosion increases the inclination of a slope and thereby the shear stresses. It may also erode stiff layers of soil exposing weaker layers which may start a type of retrogressive slide or only reduce the average strength of the soil. In areas with high artesian pressure in lower permeable layers the erosion might with time cause hydraulic bottom heave at the toe of the slope and thereby cause the whole slope to fail.

The most common human activities causing landslides are excavation at the toe of a slope or loading at the top of a slope and piledriving.

Excavation at the toe has the same effect as erosion but the process is much faster. The landslide at Rissa in Norway 1978 is the worst landslide known to be caused by human activities at the toe of a slope (Gregersen, 1981). An excavation was made in quick clay. The excavated masses caused an initial failure at the toe of the slope which developed into a retrogressive slide comprising 5-6 million m^3.

Loadings on the top of slopes are common in urban areas where houses are built on high levels. The houses themselves may have foundations that do not affect the slope stability but very often patios and terraces are placed in front of the houses. A good example is reported by Karlsrud 1982. Also in rural areas this type of loading occurs in connection with movements of soil during landscaping. Construction of road and railway embankments also belongs to this category.

Piledriving and installation of sand drains have caused a number of slides and soil movementes even in areas which were considered very stable. Piledriving causes some increase in horizontal stresses and a decrease in the shear strength but mainly very high pore pressures in soft soils. Pore pressures higher than the total overburden pressure have been measured close to driven piles. In clay these high pore pressures are confined to a small area adjacent to the pile but when permeable layers are present high pressures can spread over large areas.

Piledriving in clay has caused failure in some slopes where the safety factor before piling was low but sufficient. For example Aas 1975 has reported a case where piledriving decreased the factor of safety from 1.3 to 1.0.

Piledriving through silt layers has caused soil movements even when the ground surface was almost flat but the silt layers were inclined.

Vibrations of different kinds, for example from traffic, are often mentioned as factors contributing to landslides. Large vibrations from blastings have started landslides. An example of this is the landslide at Fröland in 1973 where a large area of silty clay with silt and sand layers was set in motion by blasting in an adjacent quarry. Earthquakes can start landslides and are supposed to have done so in the 18th century in the Göta river area but Scandinavia has in general been spared from earthquakes of any significant magnitude, which is probably a major reason why there are so many natural slopes with low safety against failure.

11. MECHANISMS OF LANDSLIDES IN SOFT CLAY

11.1 Initial slides

Many landslides develop in stages where there is first an initial slide which afterwards spreads backwards, retrogressing, or forwards, progressing, or both. The most interesting from the engineering point of view is the initial slide because if this is prevented neither of the other types of slide will occur.

The most common type of initial slide is the failure of river banks and other steep slopes due to erosion and high pore pressures. These slides are often relatively superficial and can be calculated with the use of effective stress parameters and maximum pore water pressures. Even if the pore pressures vary with season and climatic circumstances the most dangerous pore pressure situation during snow-melting and rainy seasons can be estimated. More comprehensive slides can occur if the clay under the dry crust is normally consolidated or only slightly overconsolidated and especially if a load is added at the top of the slope. In these cases the load and the increase in shear stress due to water filling of cracks and softening of the surface layers will further increase the pore pressures in the softer clay. A prediction of the magnitude and distribution of these pore pressures is very difficult to make. A relatively simple way of avoiding this problem in calculating the stability of the slope is to compare the undrained and the drained shear strength for each element of the assumed sliding zone and to use the lowest value to calculate for the most dangerous case (Larsson, 1982). In this case the undrained shear strength should preferably be determined by tests simulating the actual loading case for the different elements. In most cases undrained shear strength from field vane tests corrected from experience for plasticity is used in the calculations. The correction factors, however, are heavily influenced by experience from full scale failures of embankments and footings and loading cases where the sliding zones include equal parts of active zones, passive zones and zones with direct shear (Bjerrum, 1973, Larsson, 1980). Considering the anisotropy the corrected field vane often gives too low values for undrained shear strength in steep slopes where the sliding zone is dominated by the active and direct shear zones. This is particularly valid for low plastic clays, Fig 30.

Excavations cause similar slides. Stability is often calculated on the basis of undrained shear strength but the possibility of softening of the soil due to access to free water and the consequences thereof must be taken into account.

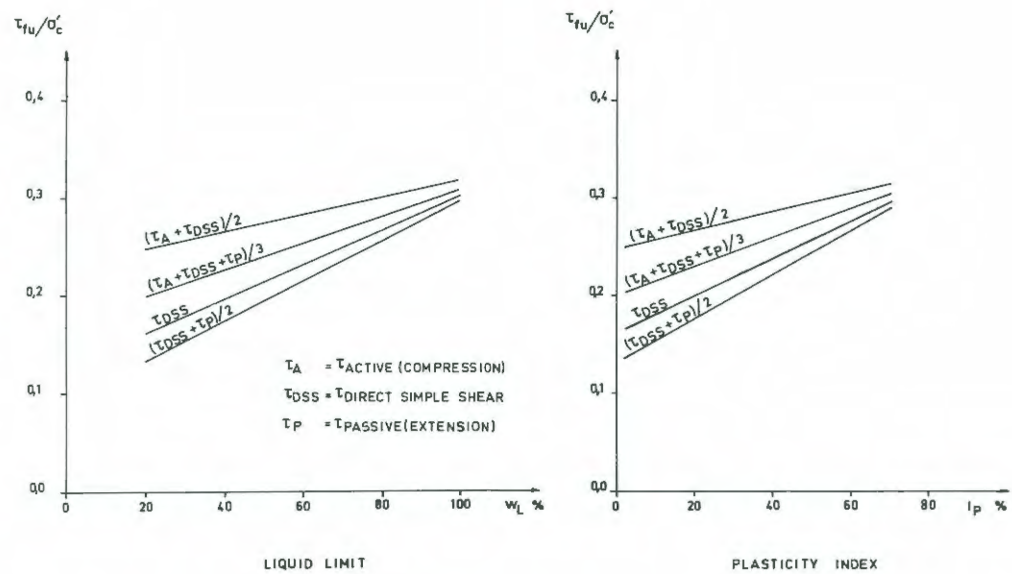


FIG 30. Empirical values of undrained shear strength for Scandinavian clays versus plasticity.

Flake-type slides often occur in areas where a stiff surface clay overlies a weaker soil. If the conditions are such that a locally instable soil mass is leaning on the soil in a lower area and the passive resistance further away in this lower area is either exceeded by an increase in downslope forces or reduced by erosion or excavation a flake-type slide can occur, Fig 31.

The increase in downslope forces may be due to loading, softening of the active zone in the stiffer clay due to waterfilled cracks and increased pore pressures or high water pressures in an inclined permeable layer. The weaker layer may be normally consolidated clay under a thick dry crust, a clay that has been weakened by leaching or other chemical changes, a thick layer of silty clay or clay with loose sand and silt layers or a permeable layer with high pore pressures.

When calculating the stability of these slopes both the drained and undrained shear strength in all elements of a sliding zone and in the different soil layers must be taken into account. If the undrained shear strength from corrected field vane tests is used the stability may be overpredicted in cases where the active zone is small or fully softened and the passive zone is relatively large in comparison to the direct shear zone or the drained shear strength is dimensioning in the direct shear zone.

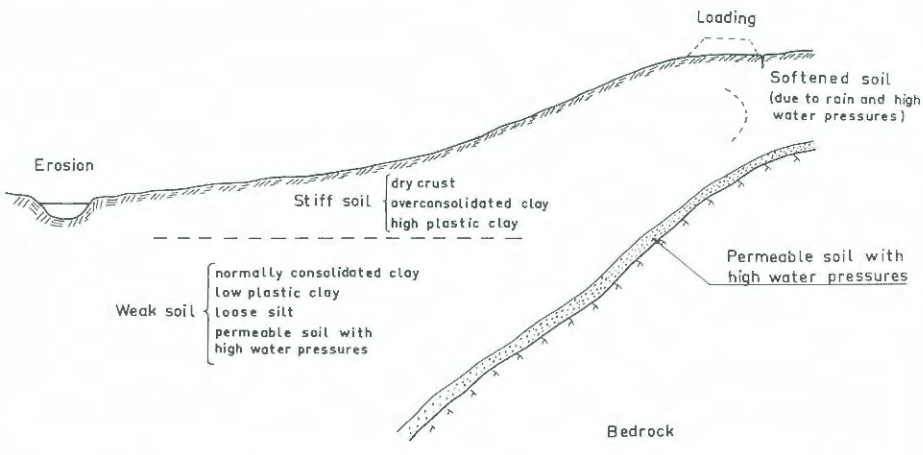


FIG 31. Typical conditions in which flake-type slides occur.

The use of partly drained and partly undrained analyses in clay is used in a very simple way by Swedish engineers in that a water-filled crack is usually assumed at the top of the sliding surface and the strength increase in the dry crust is usually neglected. In the analyses of slopes at the Norwegian Geotechnical Institute the active zone is considered softened and a water filled crack is assumed.

Instead of varying the correction factor for undrained shear strength different "required calculated factors of safety" against failure are used in Sweden. It should then be remembered that these "safety factors" are the relations between resisting and driving forces in the actual cases calculated with standard procedure that experience has shown to be required to ensure stability.

Recent research has shown that undrained shear strength is the end result at failure of the change in stresses and pore pressure and thereby effective stresses. Anisotropy of undrained shear strength is due to the different response in pore pressures at different types of loading. It would therefore seem scientifically more correct to use effective stress analyses in all cases. However, in normally consolidated clays every increase in shear stress brings an increase in pore pressure which becomes larger with shear stress level and in the case of direct simple shear becomes many times the increase in shear stress as failure approaches. This means that in this case the available resisting forces decrease at the same time as the shear forces increase. Safety factors for slopes are given as the relation between available resisting forces and driving forces. A safety factor calculated with existing effective stresses in a slope may thus well be, for instance, 2.0 when an increase in shear stress of 20% is enough for failure in the slope. Such safety factors are meaningless.

In cases where the pore pressures rise due to the stress change the available resisting forces and the safety factor should be calculated on the basis of undrained shear strength for the particular loading case. Bearing in mind that the undrained shear strength is also the strength obtained with effective strength parameters at the effective stresses prevailing at failure the use of undrained shear strength is an effective stress analysis taking the rising pore pressures into account. In more overconsolidated clays where the stress changes result in decreasing pore pressures that will rise with time the drained effective stress parameters should be used in calculation of slope stability. In all types of soil where the strength is due to effective stresses the case with the highest pore pressures and lowest effective stresses should be considered.

The pore pressure rises in the most common types of loading can be predicted from the standard types of testing (active and passive triaxial tests and direct simple shear tests) but in some complex loading situations special tests simulating the special case might have to be run (as demonstrated by Bernander et al, 1979).

11.2 Retrogressive slides

Initial slides often leave high and steep back scarps. When the passive support from the soil in the initial slide is removed the stability of the soil behind the back scarp is decreased. Very often the slide retrogresses and the final slide area can be enormous compared to the initial slide. How far a slide can retrogress seems to depend on the sensitivity and the rapidity of the soil. If a low sensitive clay fails the soil masses will not move very far and the passive resistance partly remains, Fig 32.

If the surface of the slope above the slide is only slightly inclined the average angle of the irregular slope formed after a few retrogressive slides will be small enough to prevent further slides (Kjellman, 1954).

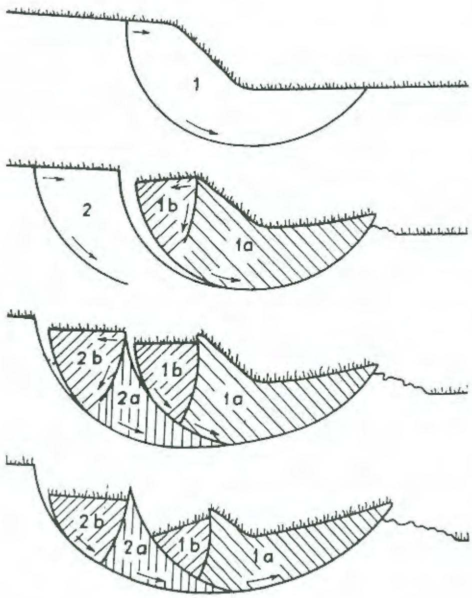


FIG 32. Mechanics of retrogressive slide according to the Geotechnical Commission of Swedish State Railways.

If, on the other hand, the clay is highly sensitive and has a high rapidity each failing slice of the soil will disintegrate and flow away like a fluid. There are many examples of this kind of behaviour from slides in Quebec and from the Trondheim area in Norway, for instance, the slide at Saint-Jean-Vianney (Tavenas et al, 1971) and the landslide at Rissa (Gregersen, 1981). The retrogression has in many cases been more than 500 metres. Tavenas et al (1982) have shown that the really large retrogressions occur in low plastic clays ($w_L < 40$) with a high sensitivity, which are also the types of clay most likely to have a high rapidity.

If there is a layer with low shear strength in the clay behind an initial slide the retrogressive slide is likely to follow this layer. The slide then retrogresses as blocks start to slide along the weak layer and are then broken up (Odenstad, 1951), Fig 33.

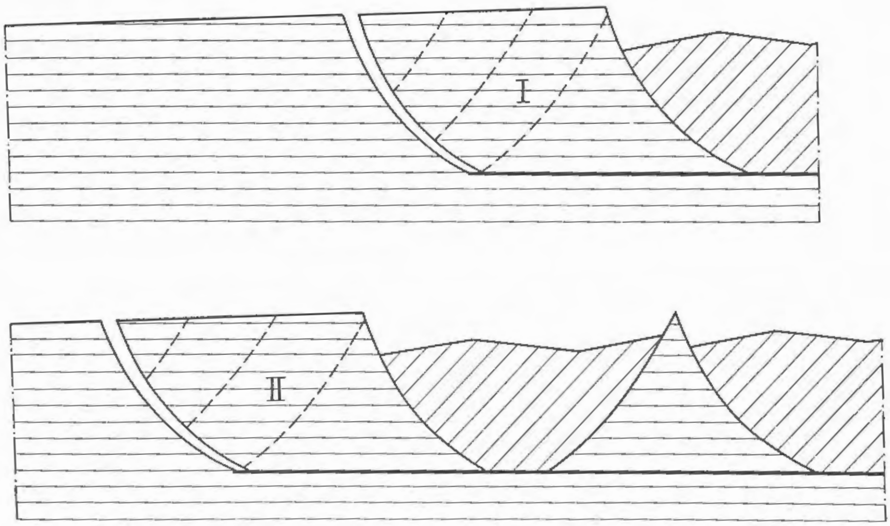


FIG 33. Mechanism of a retrogressive slide along a weak layer according to Odenstad 1951. The blocks slip and break down. A deep cleft is assumed to open behind each slipping block and to remain during the first moments of the slide.

The clay in the weaker layer is often squeezed out by the overburden pressure (Bjerrum, 1955 and discussion by Meyerhof, 1957). Mitchell and Markell 1974 showed that these slides continue as long as $H \cdot \rho \cdot g \geq 6 \cdot \tau_{fu}$ where H is the height of the unsupported back scarp after the previous slip.

The retrogressive slides can be identified by the typical steep ridges sticking up in the slide debris.

11.3 Progressive slides

After an initial slide in a steeper area the slide has often progressed very far down a gentler slope which in itself would have been stable. Kjellman in 1954 discussed how the shear strength in the initial slip surface decreases due to the large shear strains. Because of this reduction the active pressure exerted on the masses below increases and causes a new failure in the adjacent area below. There again the shear strength becomes reduced and a new failure develops further down. Examples of large progressive slides are the landslides at Surte and Svärta (Jacobsson, 1952). Lundström (1956) pointed out that the dynamic forces exerted by the moving masses in the initial slide and the additional dynamic force from masses in retrogressive slides could not be neglected but were a main contributing factor to the progression of the slide. Bernander and Olofsson (1981a and b) have elaborated on these concepts and suggest that no fully developed initial slide is necessary to explain the progressing failure.

It can be questioned how large soil masses can be expected to interact as an elastic-plastic body, but in practice the present methods have been satisfactory to predict the stability even for very long slopes (e.g.) Aas, 1982).

If an initial slide of significant size develops in a sensitive clay the prerequisites for progressive failures are certainly fulfilled. To predict the extent of this the velocity of the slide masses has to be known together with the reduction of the shear strength caused by the displacement of the soil masses over an often uneven firm bottom. These factors can hardly be predicted accurately beforehand. At best, some approximate assumptions can be made about them to study possible consequences of a progressive slide.

12. STABILITY ANALYSES AND PROBABLE SLIDE MECHANISM AT TUVE

In the first stage of the investigation a number of conventional stability calculations were carried out on the assumption that the undrained shear strength followed the normal pattern in the Gothenburg region with an average undrained shear strength of 15 kPa down to a depth of 5 m before increasing by 1.3 kPa per metre. In these calculations certain areas were found to have a locally unsatisfactory stability, $F \leq 1.5$. The lowest factor of safety was obtained for local slip surfaces adjacent to and involving the embankment on Tuve Church Road where the calculated factor of safety was about 1.0. The uncertainty of the material in the embankment, the thickness of the embankment, the strength below the embankment after more than 40 years of consolidation and the depth of the gully makes these calculations questionable. For relatively superficial slip surfaces in the area between Snarbergsstigen and the brook the calculated factor of safety was 1.3. Analyses of more extensive, deep slip surfaces gave factors of safety in excess of 1.7.

Analyses where the permeable layers in the bottom were assumed to be drained using the empirical effective strength parameters $c' = 0$ and $\phi' = 30^0$ were carried out. The water pressure in the friction soil below the clay was assumed to be equal to a groundwater level in the ground surface in the areas above Tuve Church Road. Artesian conditions with a head above ground level of 4 m were assumed at New Tuve Road and adjacent to the gully. These calculations gave a considerably higher factor of safety than those using undrained shear strength. Even if the water pressure level in the bottom layers was to be raised by 5 m within the entire area the factors of safety would still be in the range 1.8-2.2, Fig 34.

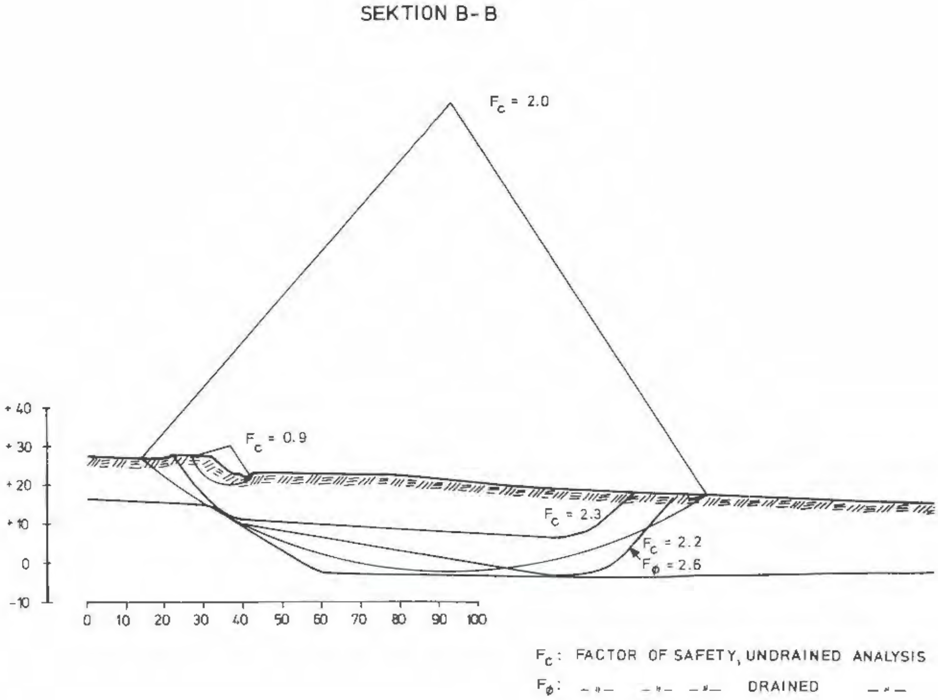


FIG 34. Results from stability calculations through section B-B (see Fig 19) assuming a continuous increase of undrained shear strength with depth. Increase of shear strength under the embankment due to consolidation has been disregarded.

When a more complete picture of the most probable geotechnical conditions before the slide could be drawn new stability calculations were made. In these calculations the water pressure in the bottom layers was assumed to be artesian with a water level 2 m above the ground surface in accordance with the geohydrological prediction. A one metre deep water filled crack was assumed in the active zone and the active shear strength was assumed to be fully softened for the high water pressures. The undrained shear strength in direct shear zones and passive zones was calculated from empirical relations (Fig 25) and the passive shear strength in the dry crusts was assumed to be 12 kPa. The bottom layers of silty clay and clay with loose silt and sand layers were assumed to have an undrained shear strength of 0.12 σ_c'.

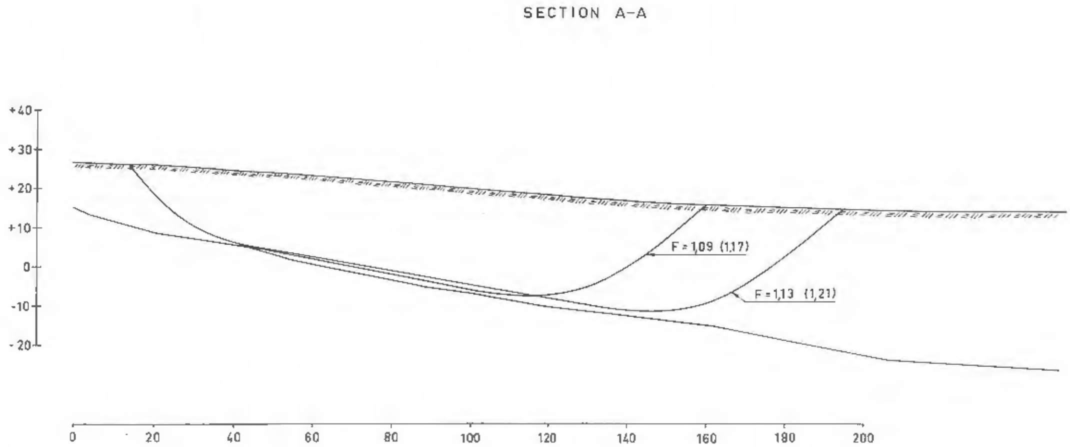
These calculations gave safety factors between 1.0 and 1.1 in areas between the small brook and Snarberget and below Tuve Church Road. The slip surfaces with the lowest factors of safety were deep and followed the weak silty layer. They had a length of 120 to 150 metres, Fig 35.

In the calculations the shape and depth of the brook gully as determined from aerial photographs in 1970 has been used. Until November 1977, further erosion had occurred but to what extent is unknown.

Some fills had been placed in front of the houses at Snarbergsstigen and the hydrogeological prediction of the artesian pressure in the bottom layers allows a somewhat higher pressure than the pressure assumed in the calculations.

If these factors are taken into account the undrained shear strength in the bottom layers could have been higher than assumed and the calculated factor of safety would still be about 1.0.

Additional calculations were made using the undrained shear strength that empirically should have been measured by the field vane test in the particular conditions. The conventional assumptions with a water filled crack and no strength increase in the dry crust were made. These calculations gave factors of safety between 1.1 and 1.3 for the slip surfaces which gave factors of safety between 1.0 and 1.1 when softening, pore pressures and anisotropy had been taken into account.



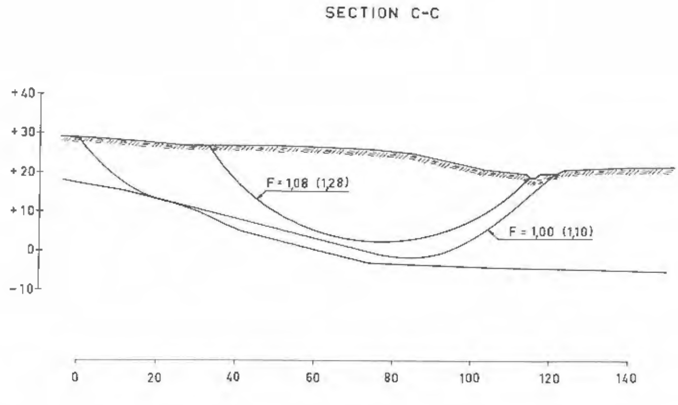


FIG 35. Results from stability calculations assuming empirical shear strengths and using the lowest alternative of drained and undrained shear strength in active and direct shear zones. The passive shear strength in the dry crust is assumed to be 12 kPa. Safety factors in brackets refer to conventional analyses using empirical undrained shear strengths as obtained in field vane tests.

A deep slip surface including the houses at Snarbergsstigen and extending to the brook gully had a calculated factor of safety of 1.0. This slip surface is within the area first seen to move and passes through the bottom layers where the sliding zone was later found.

A process leading to failure in this area would be that first the water pressures in the permeable bottom layers were rising due to the rainfall. When the pressures rose the shear stresses in the middle and lower parts of the area increased, the available shear strength in the steeper parts of the permeable layers decreased and the soil above had to lean on soil further down for support.

At the same time the cracks at the surface started to be filled with water and the softening process of the clay in the active zone started. This softening process went relatively fast in the dry crust and in areas where the thickness of the clay layer was limited as the permeable bottom layer gave access to free water from two sides. This softening process further increased the shear stress in the lower parts of the area. When the groundwater basin became filled and all further precipitation had to be discharged at the surface the surface cracks were completely filled. The down slope forces then increased further and the final softening which caused the shear stress to exceed the available shear strength occurred. The whole process was so fast that no significant dissipation of the pore pressures created in the clay down the slope due to the increased stresses in these parts could occur.

The conditions in the soil had probably been similar for short periods many times before but since the last time the area had been developed. The houses at Snarberget and connected roads had been built, the gully above Tuve Church Road had been filled in and an additional fill had been placed on top of the former gully. Some erosion had taken place in the gully below the road. The development caused only minor changes in the stress situation but this was all that was needed as the stability had been very poor in previous wet seasons.

Smaller initial slides can have occurred along the brook gully due to erosion. They can have taken place in the gully banks or in the road embankment. If so, they occurred within a larger area for which the overall stability was already critical.

The masses from the first slide area exerted an increase in driving force on the soil below Tuve Church Road and south of the small brook where the stability was already low (about 1.1, section A-A, Fig 35) starting a second large slide.

First the masses from the first slide and then the masses from the first and second slide jointly exerted increasing pressures on the soil further down the large slope as the shear strength in the sliding zone decreased and the slide progressed.

Due to the weaker zone in the bottom layers the progressing slide followed the bottom contour.

The first and second slides had left high and steep back scarps. The value of $H \cdot \rho \cdot g$ far exceeded 6 times the undrained shear strength in the weak zone and retrogressive slides mainly in the pattern described by Odenstad 1946, Fig 33, started. The retrogressive slides continued in most places until they reached firm ground. Only in a small part of the western slide area did they end in clay.

The dynamic forces from the ongoing progressive slide aided by forces from the masses involved in the retrogressive slide created the final extent of the passive zone.

13. SUMMARY AND CONCLUSIONS

A landslide occurred at Tuve in November 1977. Due to the slide 9 people were killed and 436 became homeless. The slide finally involved an area of 27 hectares and the direct costs resulting from the slide amounted to 150 million Swedish crowns (≈30 million US $).

The slide took place mainly in a weak silty bottom layer about 5 m thick. It was initiated by heavy rainfalls resulting in artesian water pressures in permeable bottom layers, softening of the clay and possibly erosion. The stability of large parts of the slide area had been poor for a long time and had lately been somewhat lowered due to development in the area. The main rule that in all earthwork in sensitive slopes measures should be taken to improve the stability of the slope at all stages had been followed locally. The stability of a slope above Tuve Church Road had been improved by a fill at the toe. This improvement however might have worsened the situation in the rest of the valley. For the slopes below Tuve Church Road where the slide started the effect of the development was a slight decrease in the stability.

The previous geotechnical investigations had been aimed at finding suitable locations and foundation methods for the buildings. The reports had advised against placing larger fills in parts of the area because of the stability of the ground sloping towards the small brook. No investigation of the stability of the whole area had been made.

To investigate the stability of clay slopes it is not sufficient to continue the borings until a steady increase in shear strength is found. The borings must be continued to firm bottom to investigate the eventual existence of weaker or permeable layers and their topography. There is of course a depth limit beyond which it can be mathematically shown that not even the weakest layer could affect the stability.

If an investigation of slope stability using the field vane test and conventional stability analyses had been properly made, factors of safety in the order of 1.1 and 1.3 would probably have been calculated in large areas and it would have been obvious that the stability of the area was insufficient. If the safety factor from a conventional stability analyses is that low for a built-up area either stabilizing measures should be taken or more extensive investigations should be made.

More extensive investigations and more advanced methods of analyses would probably have shown that the factor of safety for the slope was close to 1.0.

The final extent of the slide was due to retrogressive failure in the areas behind and progressive failure in the sloping ground in front of the initial slide. The sliding zone followed the weak silty bottom layer.

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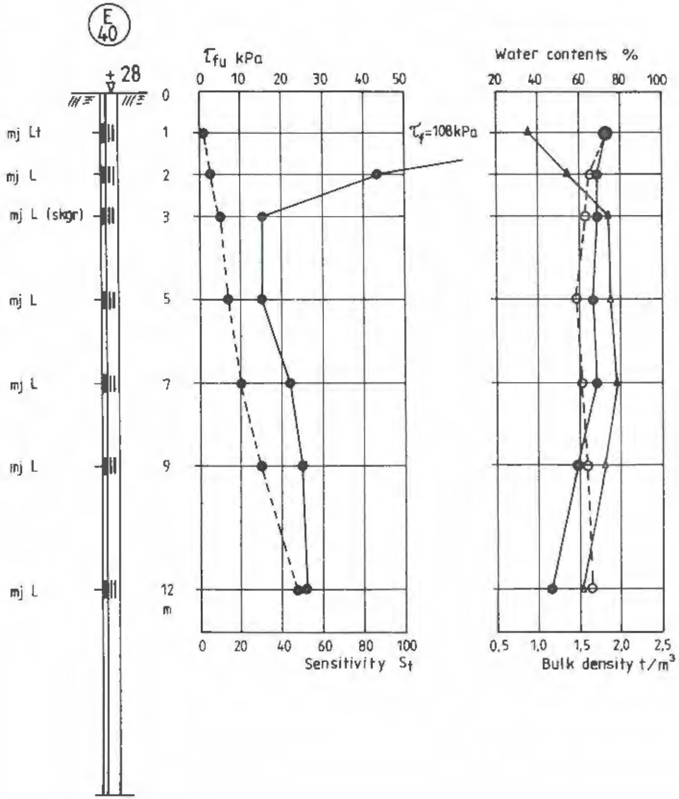
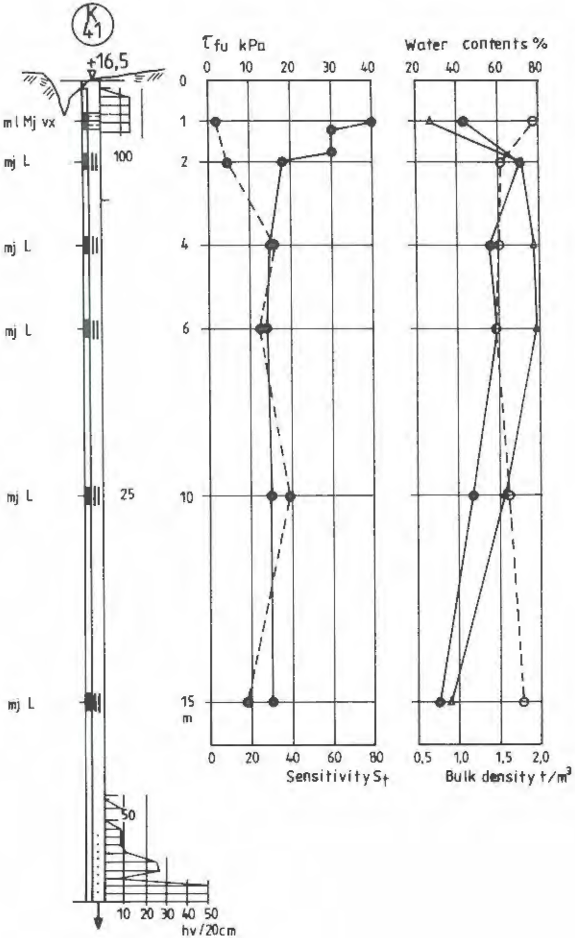
APPENDIX A

Results from geotechnical investigations before the slide.

The investigations in this appendix were made in 1964-1965 by Brodefors & Mattson AB for the development of the area between Snarberget and the small brook.

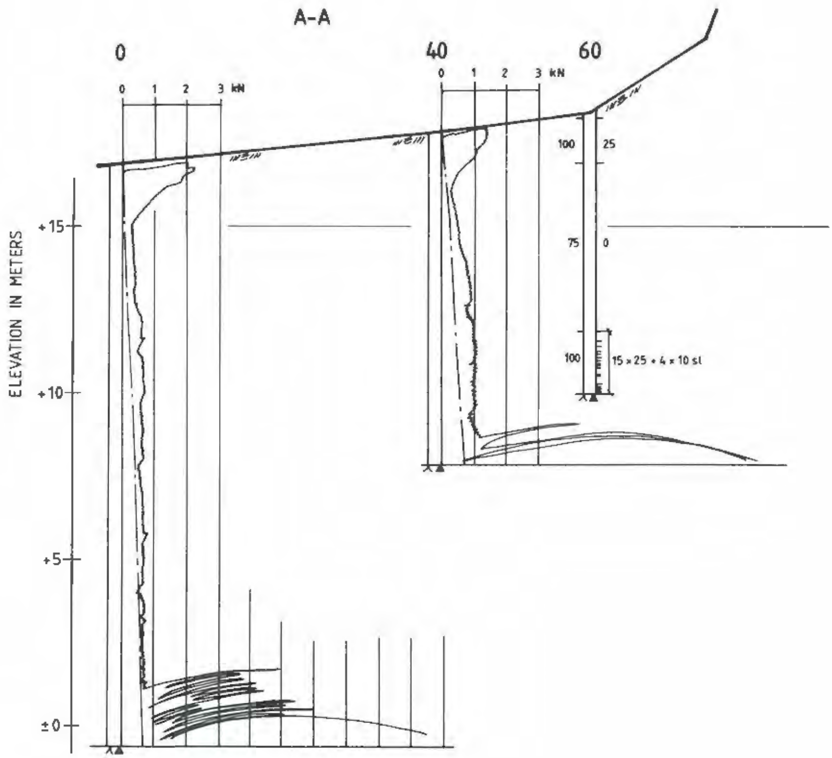
Location of borings in 1964-65.

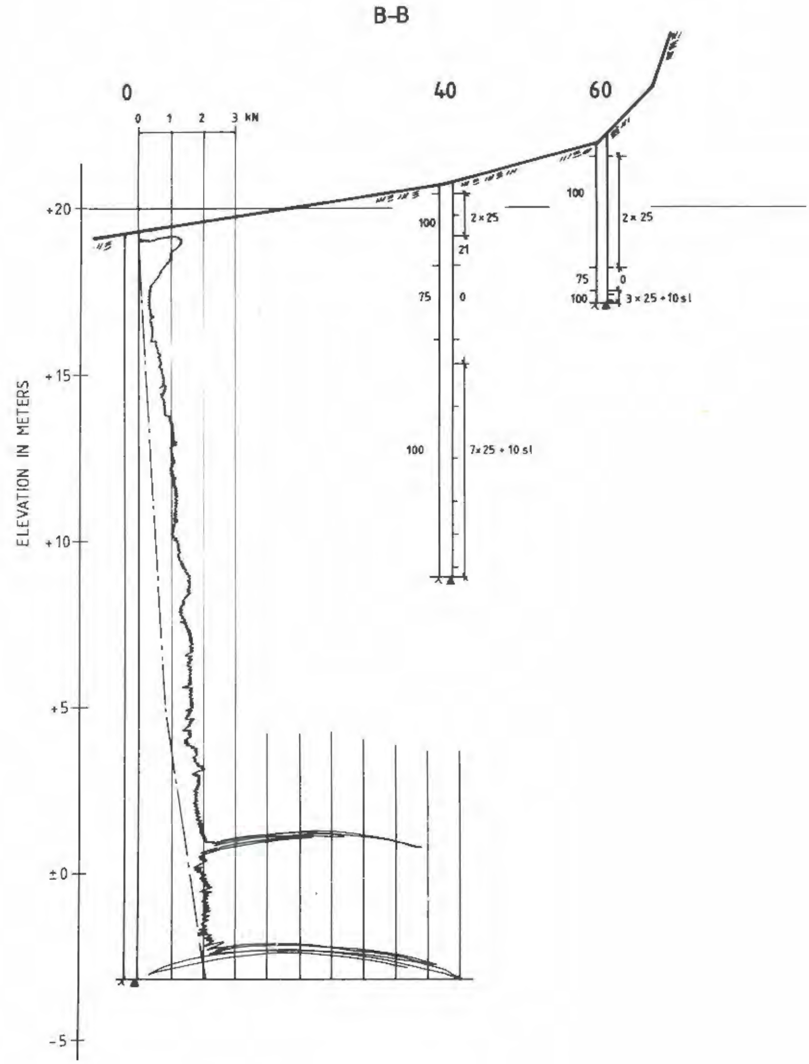


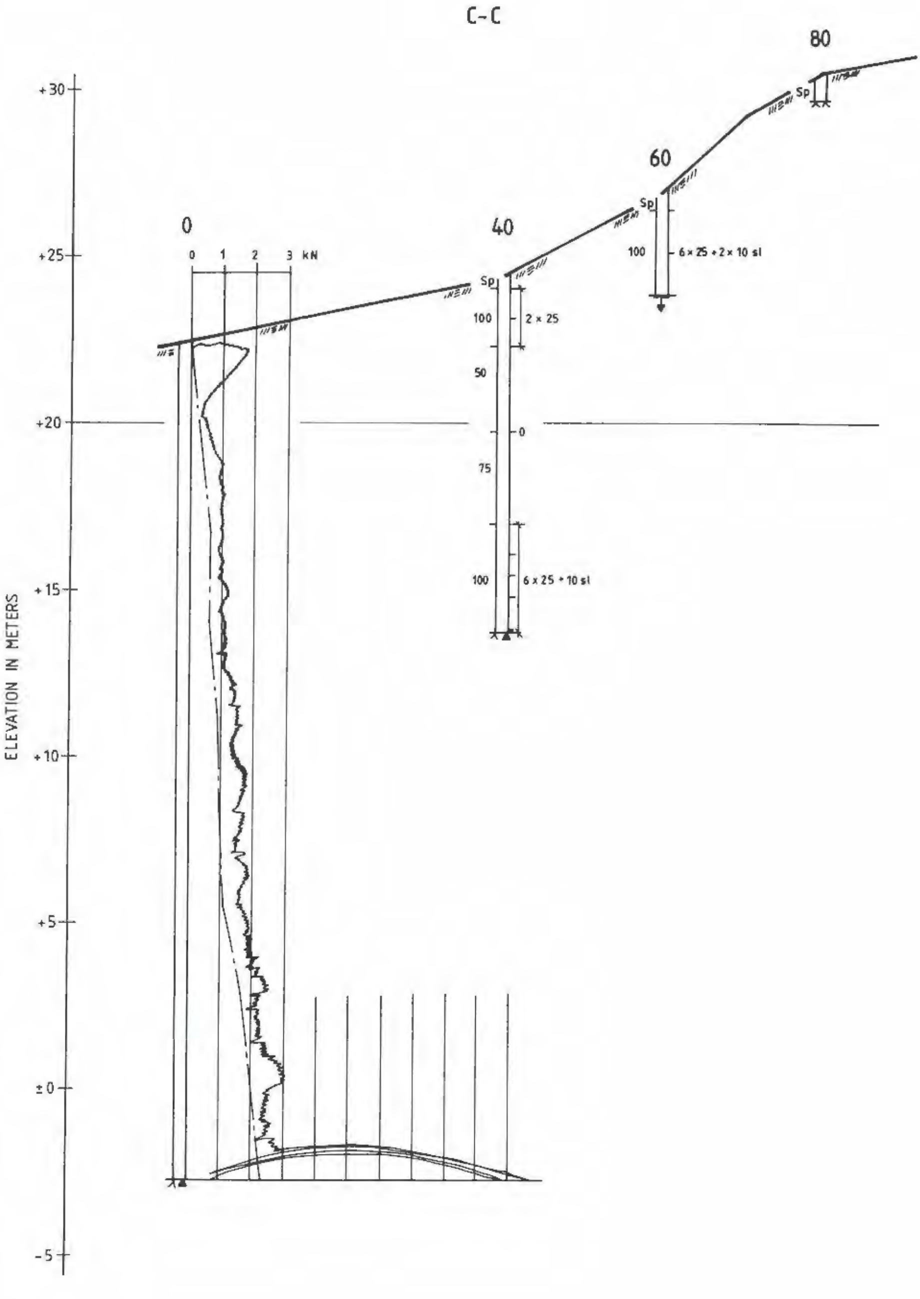


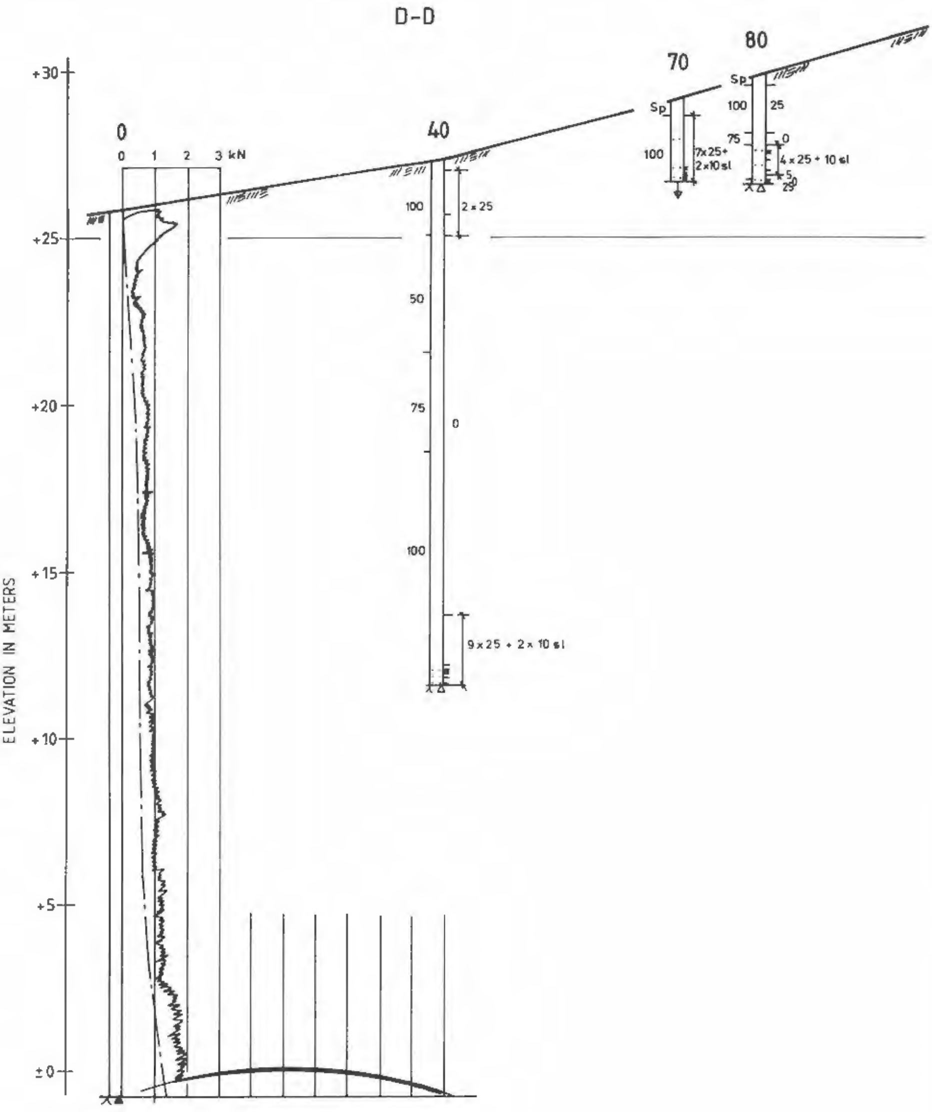
Anmärkning

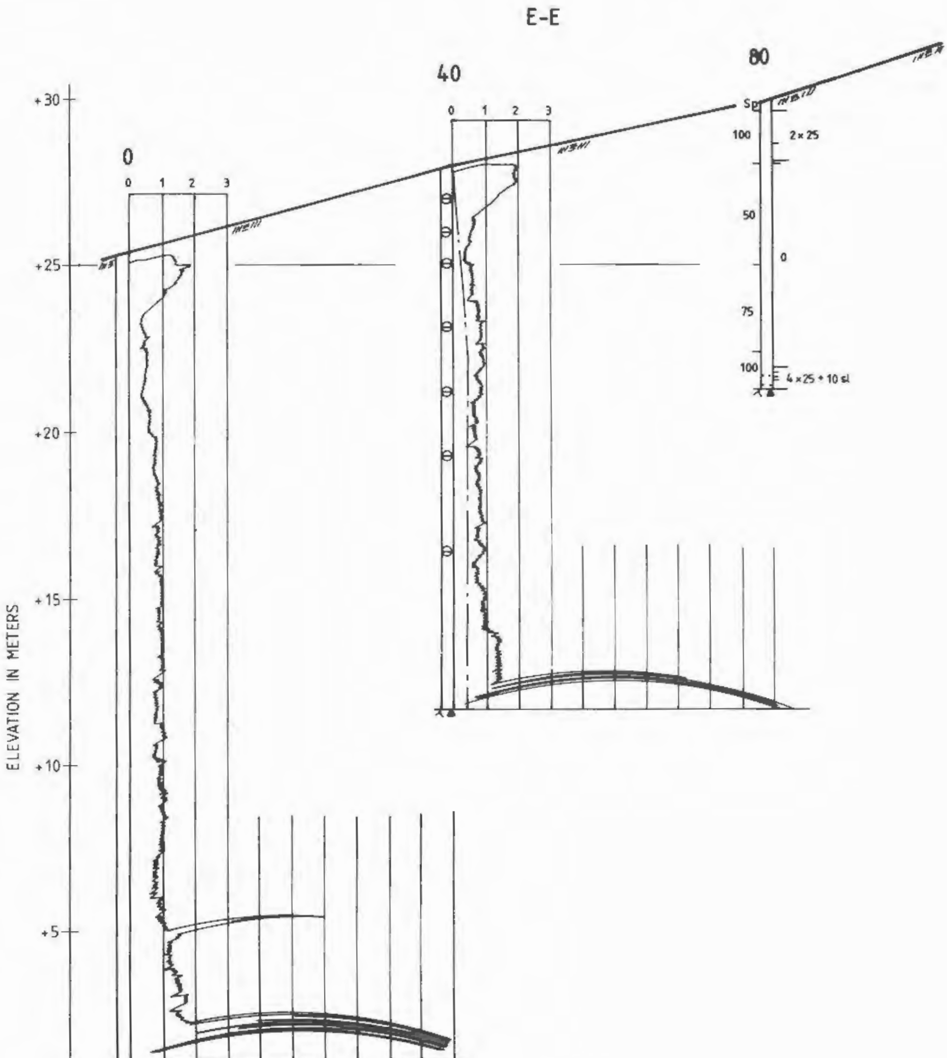
Borrpunkt E40 är hämtad från Brodefors & Mattsons
ritning 13-20-G5 (dat. 65-05-19)


A-A
0
40
60
0 1 2 3 kN
0 1 2 3 kN
ELEVATION IN METERS
+15
+10
+5
±0
100
25
75
0
100
15 × 25 + 4 × 10 st

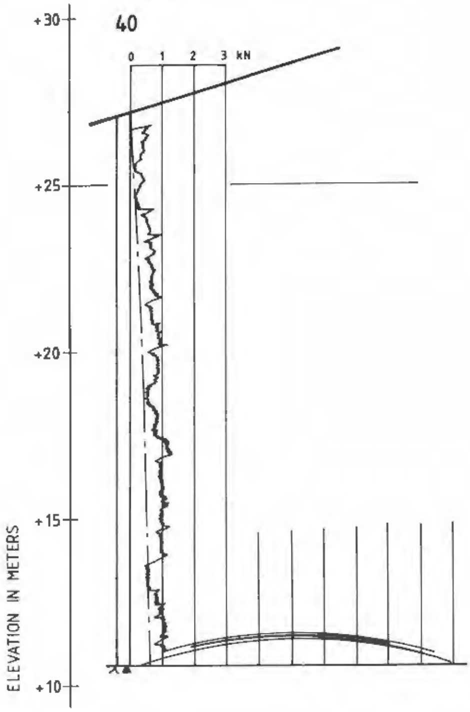

B-B
0
40
60
0 1 2 3 kN
+20
+15
+10
+5
±0
-5
ELEVATION IN METERS
100
2×25
21
75
0
100
7×25 + 10 s l
100
2×25
75
0
100
3×25 + 10 s l


C-C
0
40
60
80
0 1 2 3 kN
Sp
100
2 × 25
50
0
75
100
6 x 25 + 10 sl
6 × 25 + 2 × 10 sl
+30
+25
+20
+15
+10
+5
±0
-5
ELEVATION IN METERS


D-D
+30
+25
+20
+15
+10
+5
±0
ELEVATION IN METERS
0
0 1 2 3 kN
40
100
2 x 25
50
75
0
100
9 x 25 + 2 x 10 sl
70
Sp
100
7x25+
2x10sl
80
Sp
100
25
75
0
4 x 25 + 10 sl
5
0
25

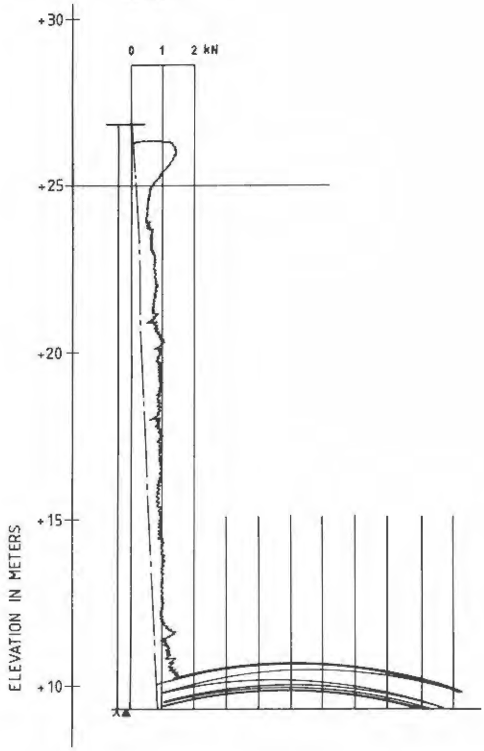

E-E
0
40
80
+30
+25
+20
+15
+10
+5
±0
ELEVATION IN METERS
0 1 2 3
0 1 2 3
Sp
100
2 x 25
50
0
75
100
4 x 25 + 10 sl

F-F



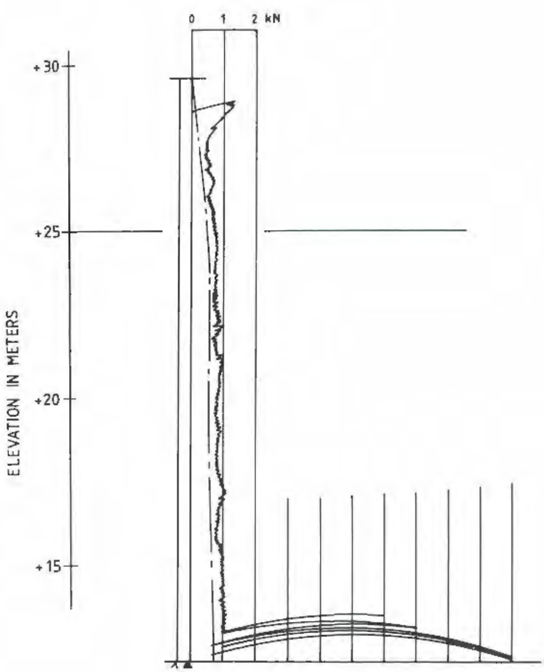
40
0 1 2 3 kN
+30
+25
+20
+15
+10
ELEVATION IN METERS

G-G



0 1 2 kN
+30
+25
+20
+15
+10
ELEVATION IN METERS

H-H

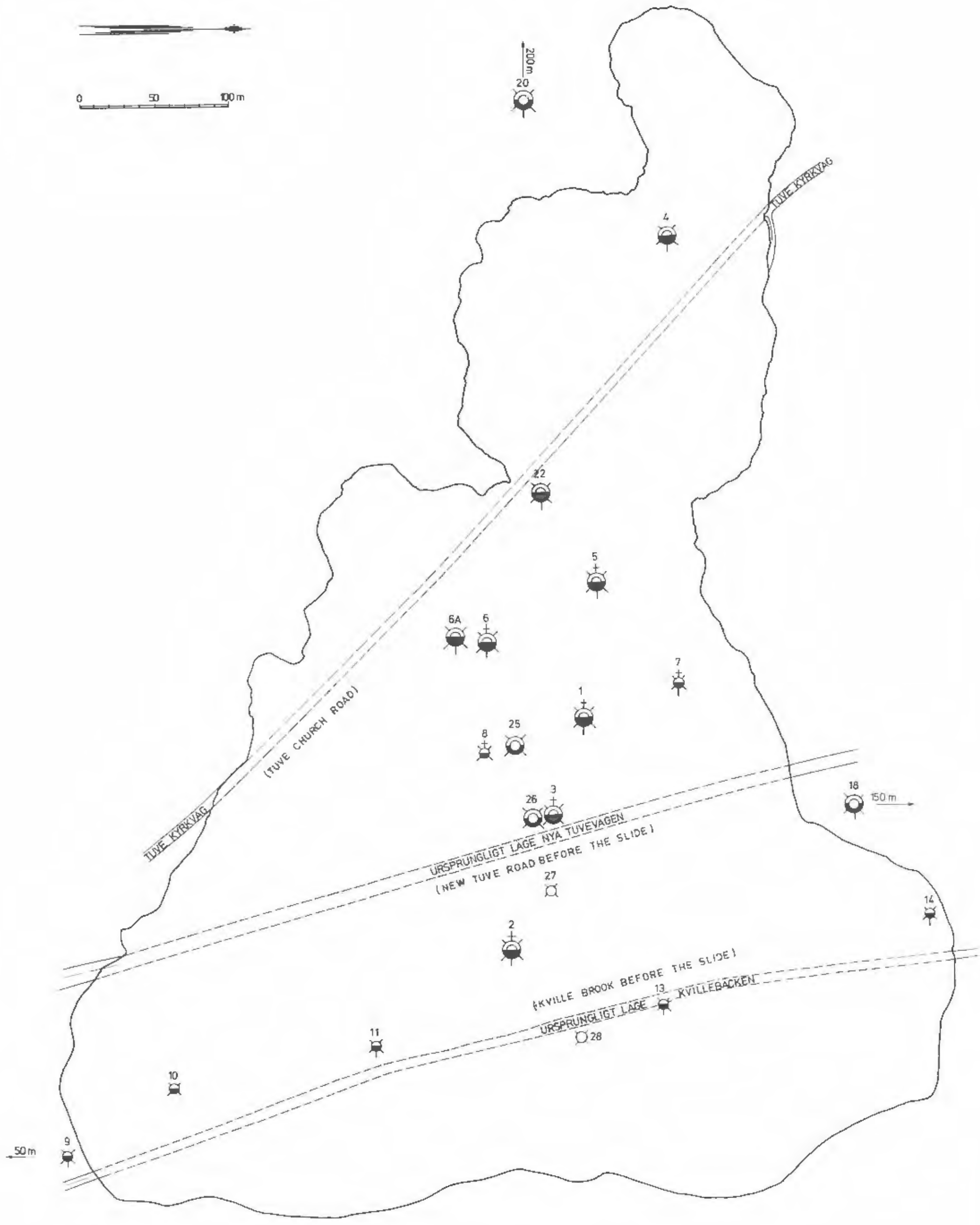


0 1 2 kN
+30
+25
+20
+15
ELEVATION IN METERS

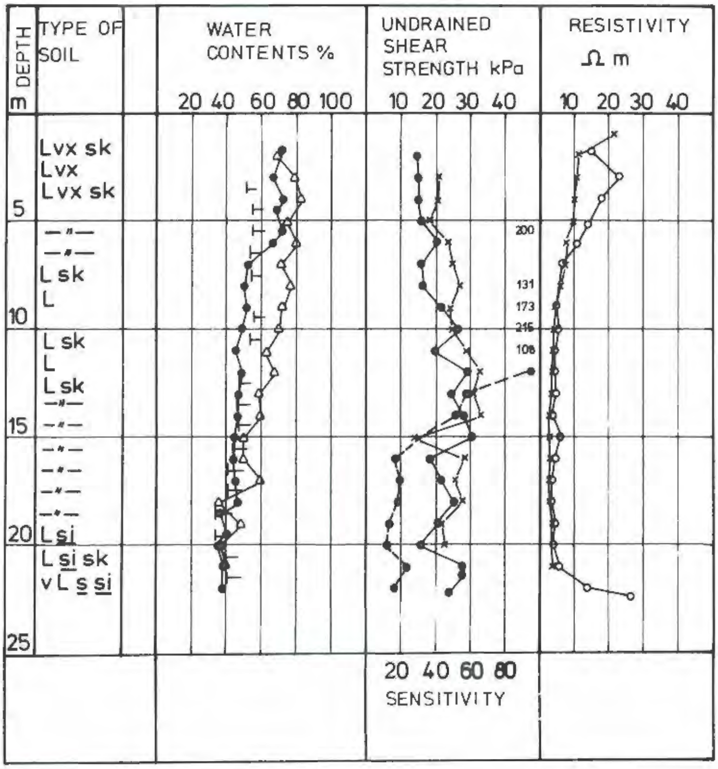
APPENDIX B

Results from geotechnical investigations after the slide.

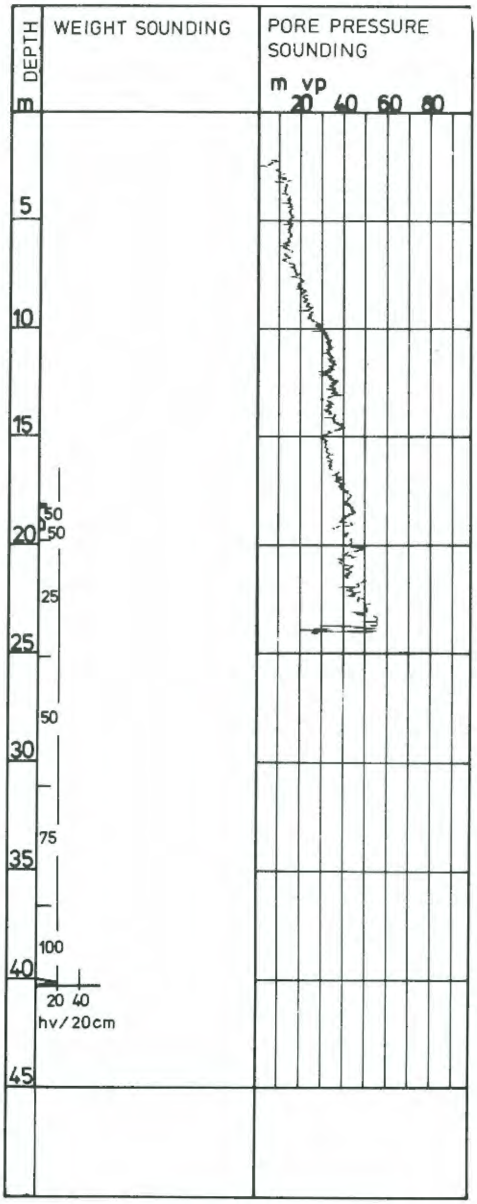
Borings made by SGI and the Geotechnical Department at Chalmers University of Technology for the slide investigations.



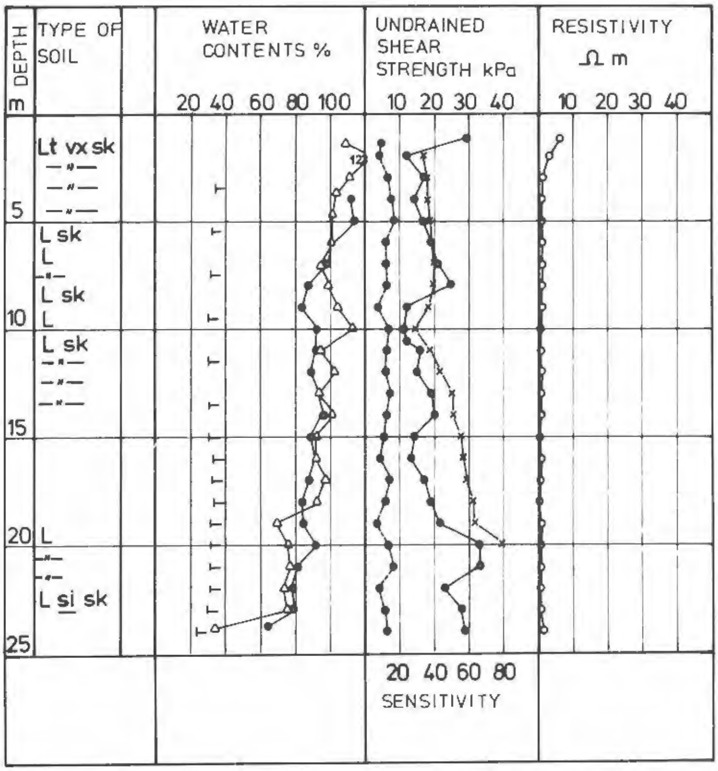
Location of borings made by SGI and CTH.



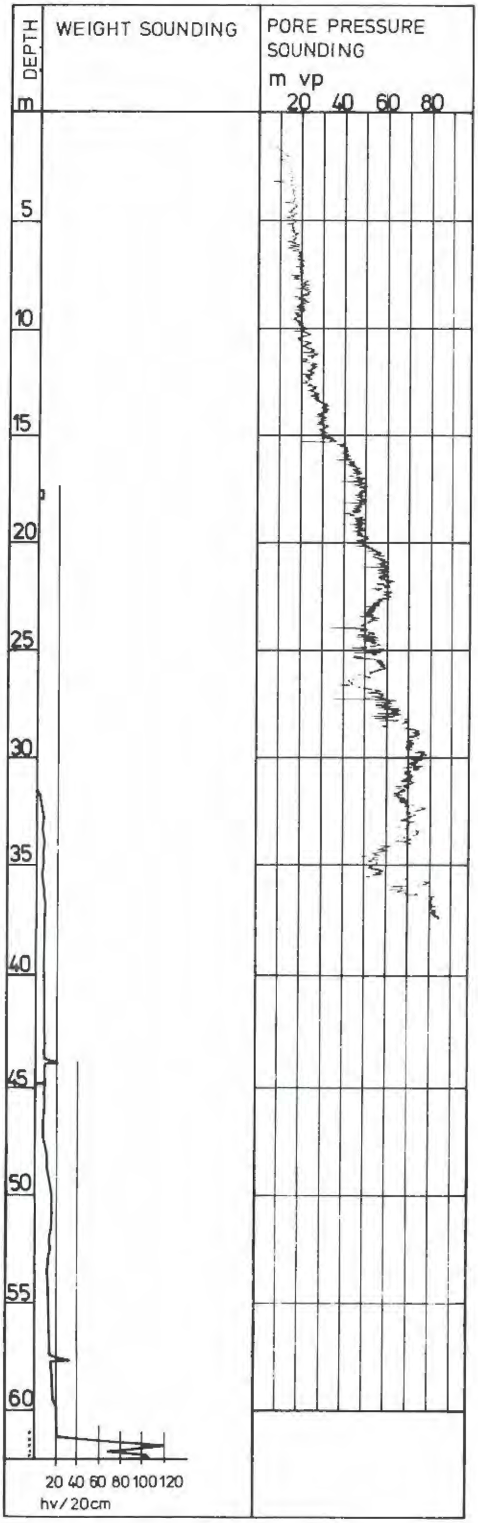
Borehole No 1.



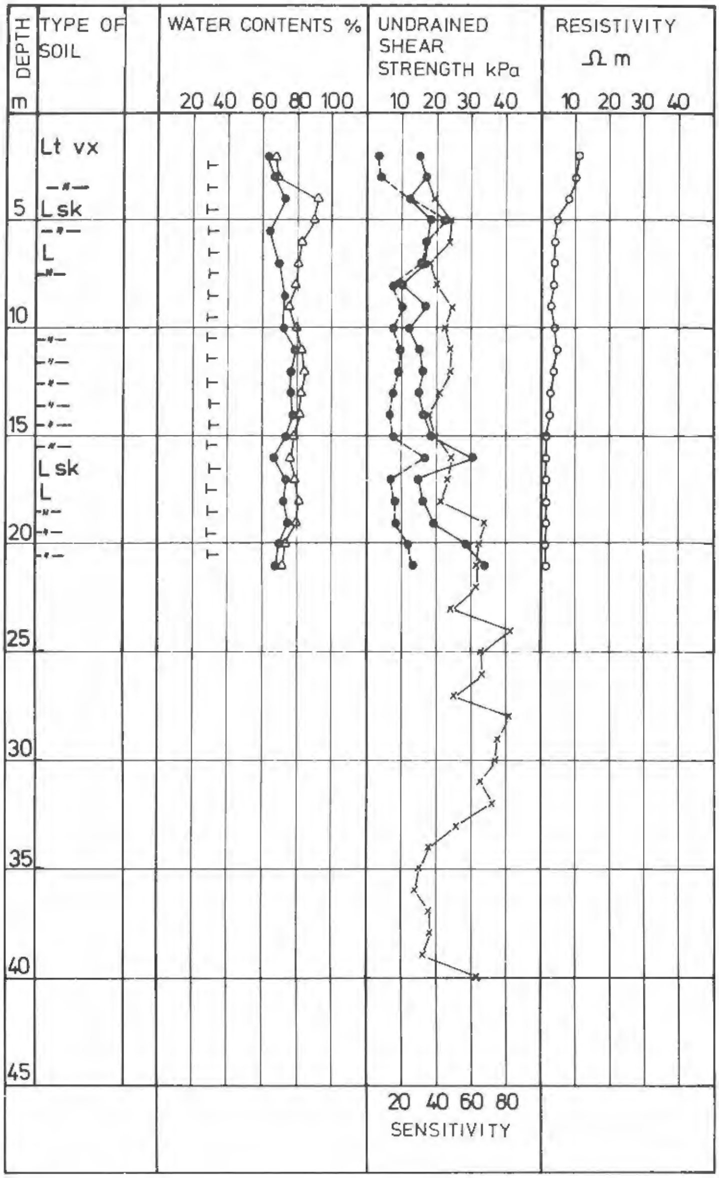
Borehole No 1.



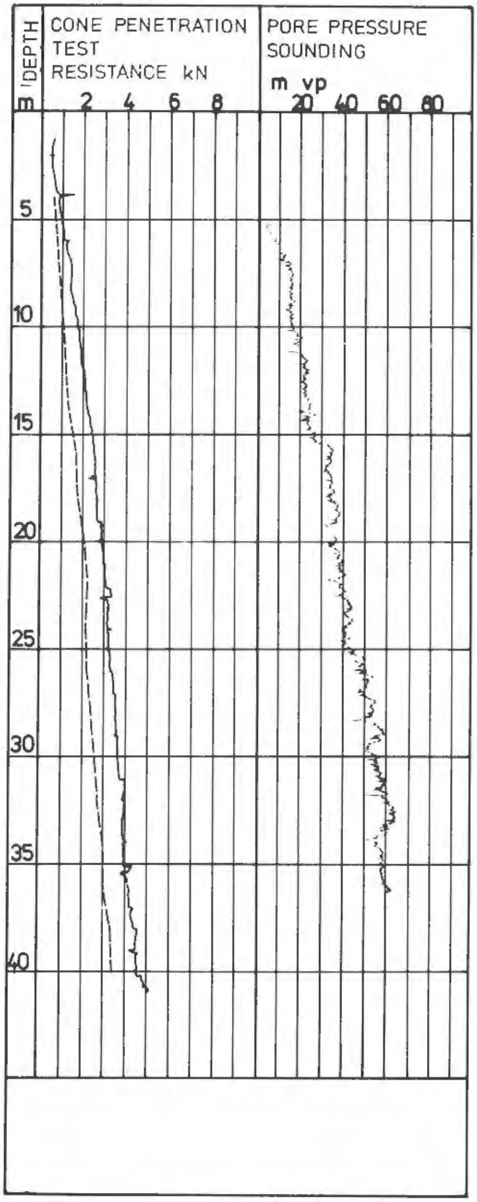
Borehole No 2.



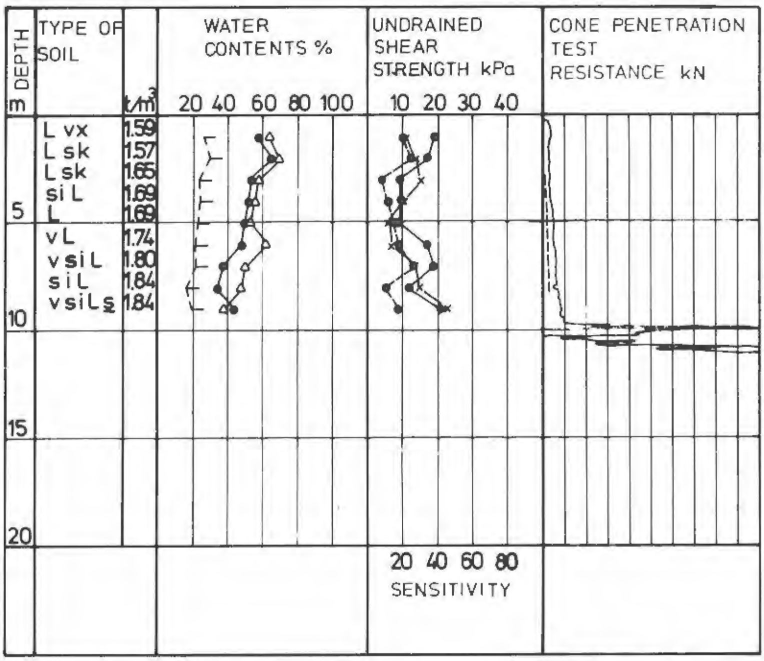
Borehole No 2.



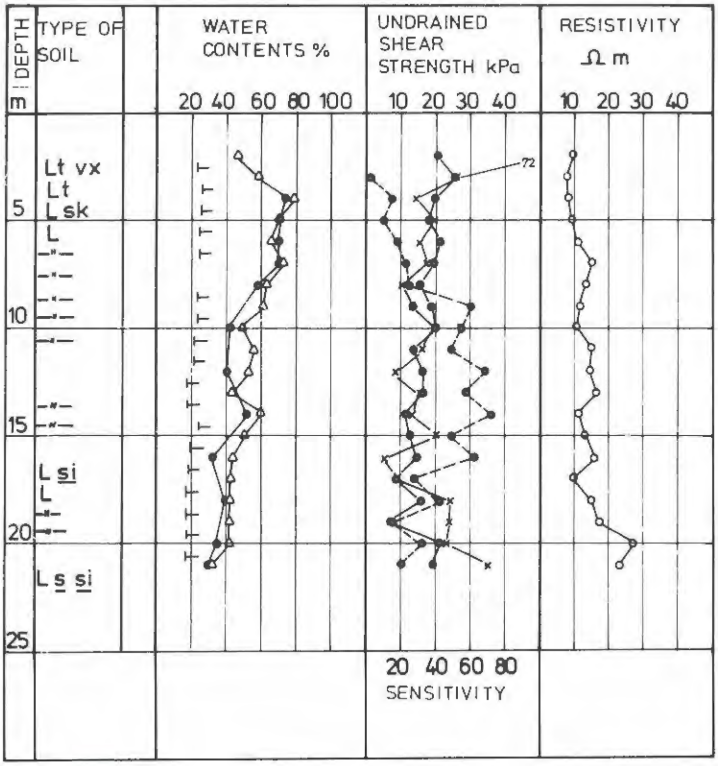
Borehole No 3.



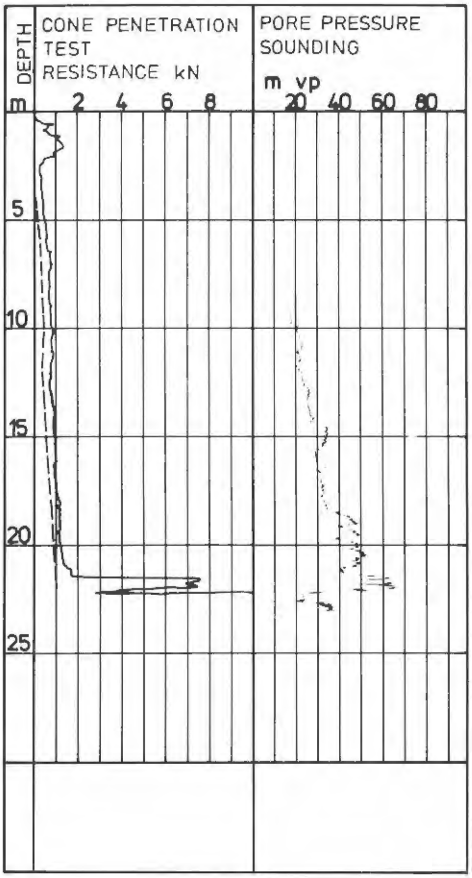
Borehole No 3.



Borehole No 4.



Borehole No 5.



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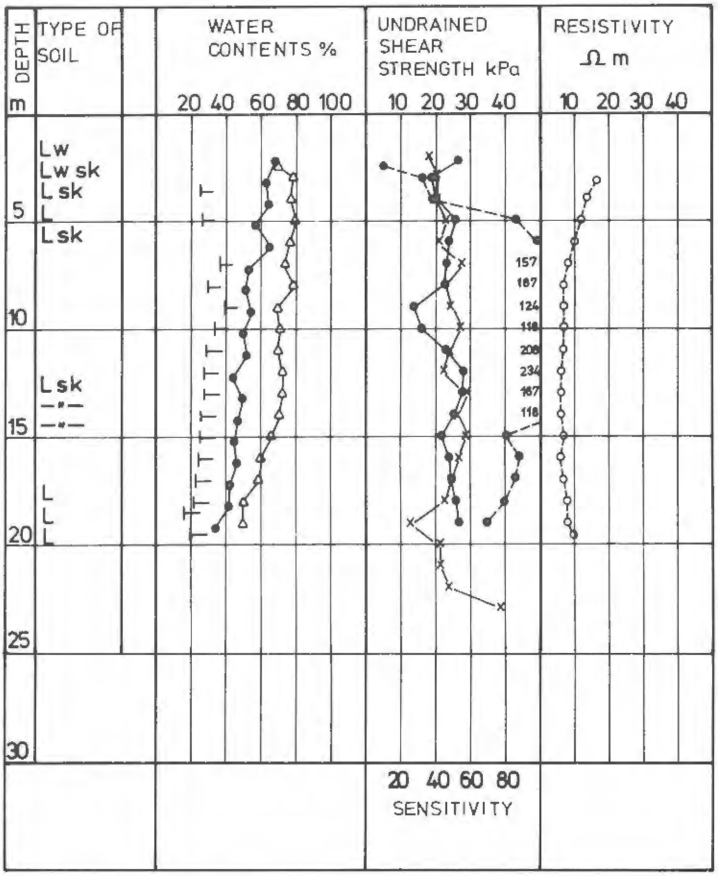
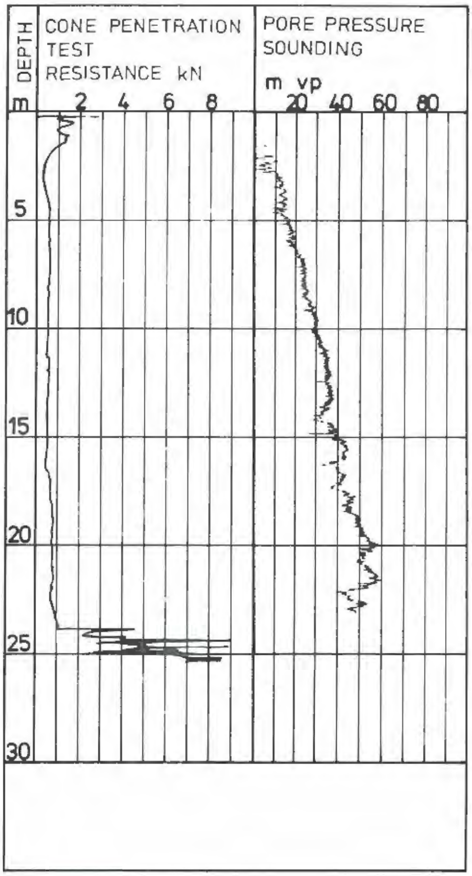
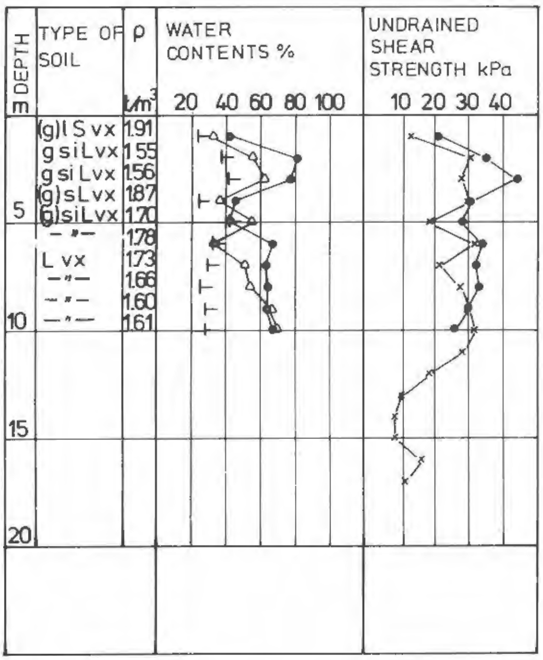


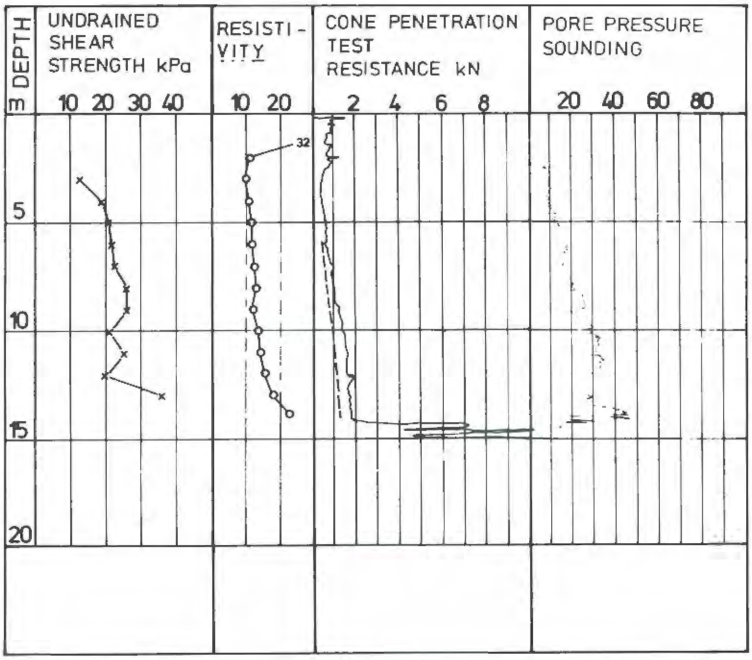
Borehole No 6.



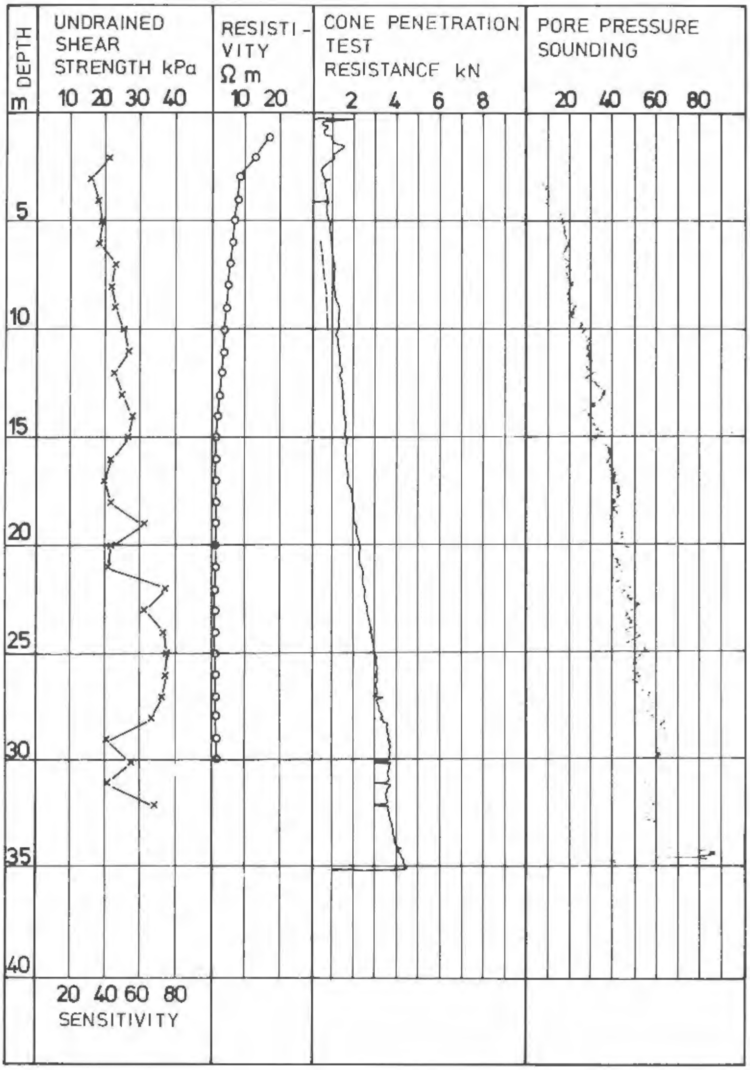
Borehole No 6.



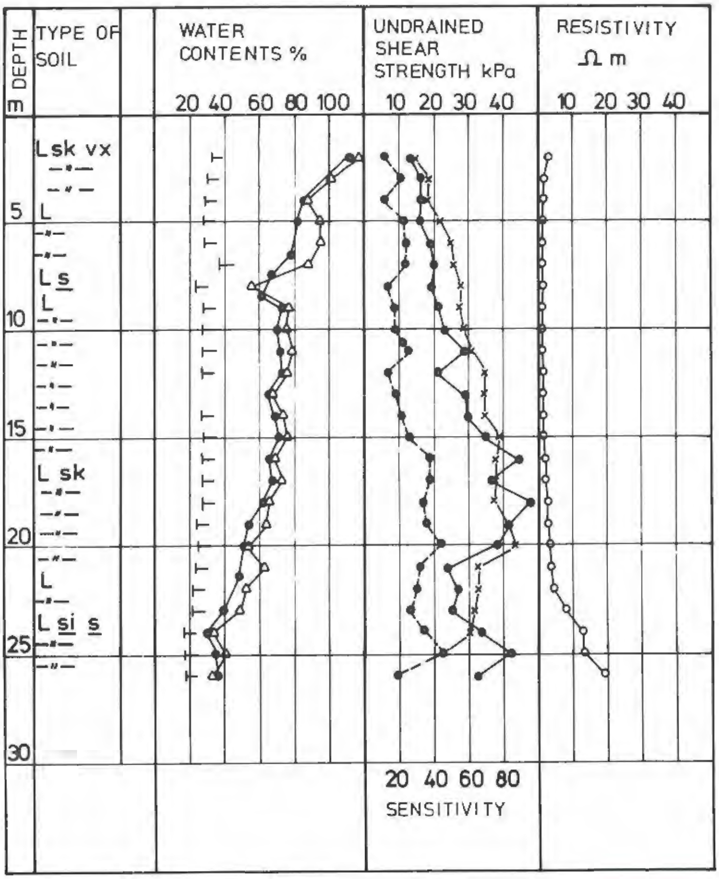
Borehole No 6A.



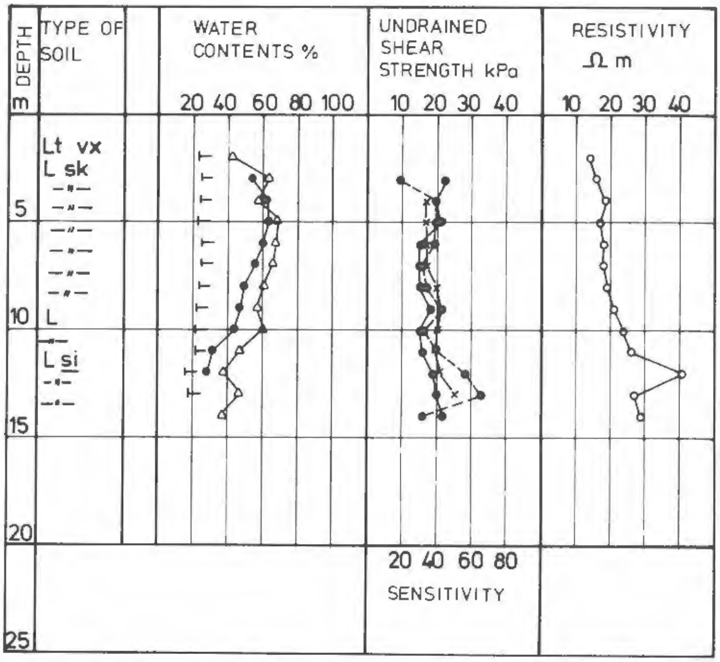
Borehole No 7.



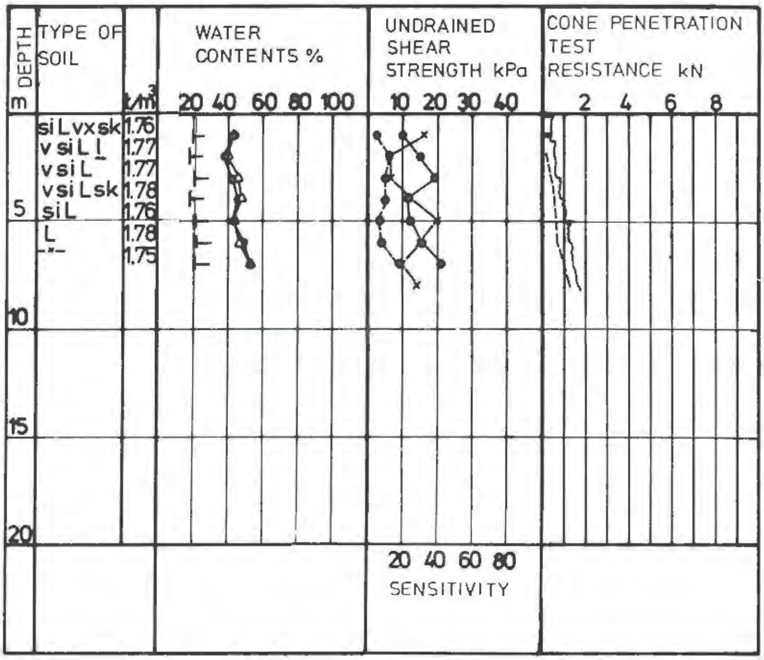
Borehole No 8.



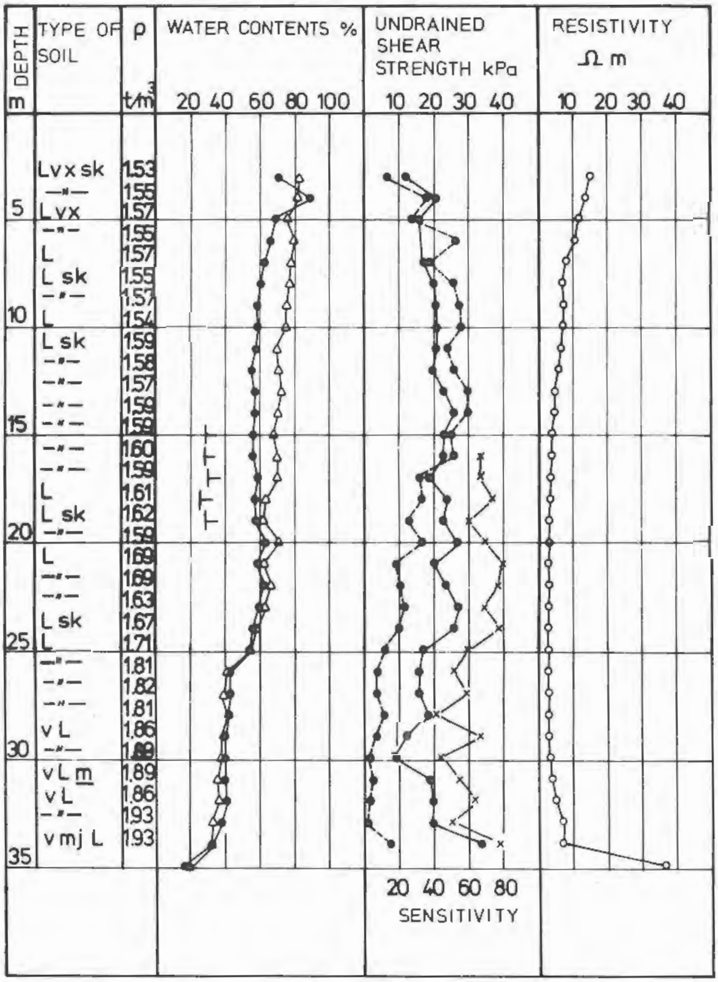
Borehole No 18.



Borehole No 20.



Borehole No 22.



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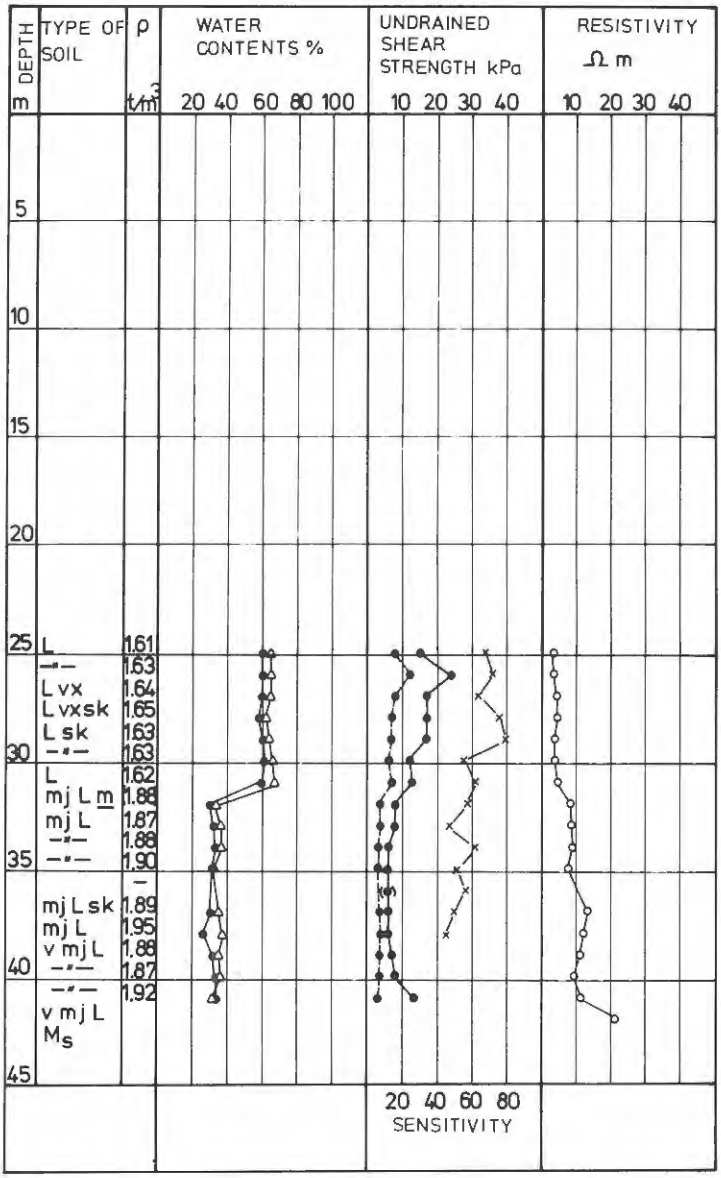

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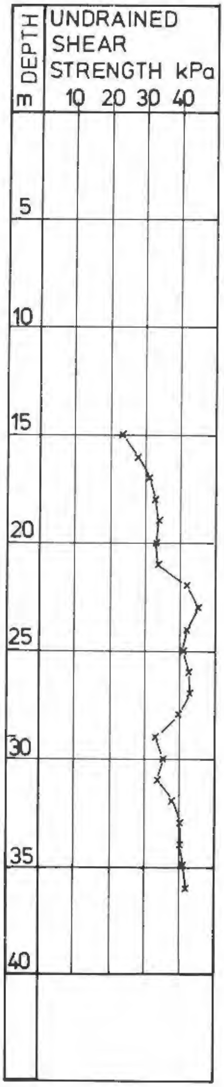
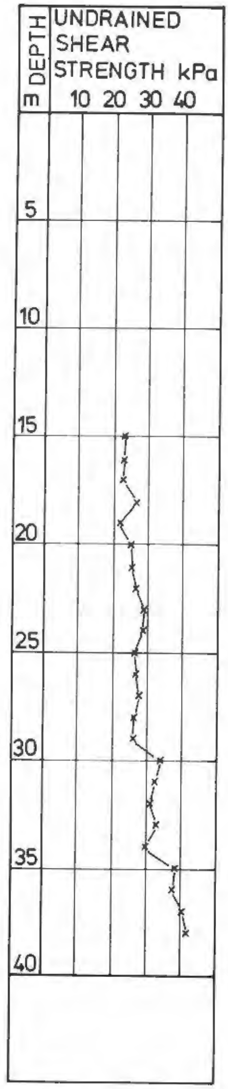




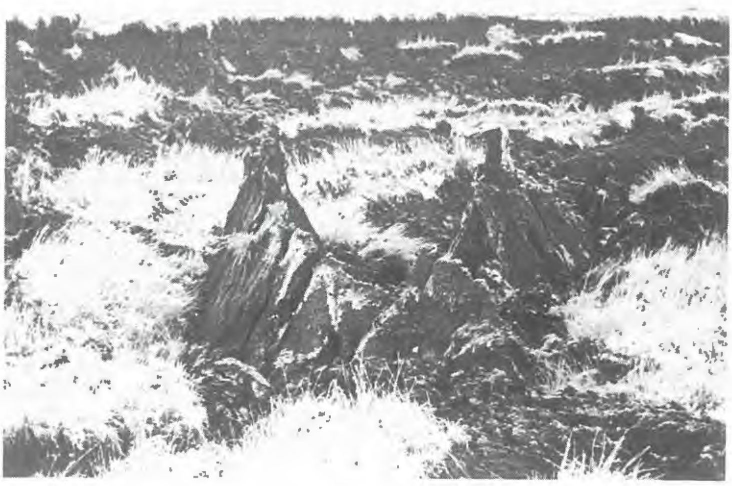
Boreholes No 27 and 28.

APPENDIX C

Photographs taken after the slide.

Collected by Lars Magnus Fält, Chalmers University of Technology, Department of Geology.

Slip surfaces close to firm bottom in the upper part of the slide area.



Steep ridges sticking up in the debris in the active zone (compare chapter 11.2).

The ground surface in the passive zone.

Demolished houses and broken up ground in the slide area.

Views of the slide area.

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