STATENS GEOTEKNISKA INSTITUT
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Pore pressure distribution – Shear strength properties – Stability – Environment

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Preface

This report presents the results of a research project concerning the long-term effects of excavations performed at crests of clay slopes in order to increase the stability. Three slopes in which such excavations were made more than ten years ago have been investigated with respect to their present conditions. The studies have concerned the pore pressure distribution and the shear strength of the soil, whether the intended stabilising effect has been achieved and general aspects of the environment in the area.

The report is intended for those who plan and design measures to increase the stability of clay slopes and for those who plan the use of land in stabilised areas. It is thus intended for practising geotechnical engineers, the Swedish Rescue Services Agency, the Swedish Rail and Road Administrations, other similar private and national agencies, landowners and municipal and regional offices for physical and environmental planning.

Besides elucidating the questions already mentioned, the results of the project have led to new recommendations for shear strength testing by field vane tests. These have been presented separately in SGI Varia No. 509 and a paper in Väg- och Vattenbyggaren, No. 4, 2001. The results have also led to revised methods for evaluation of field vane tests, CPT tests and dilatometer tests in overconsolidated clay. Furthermore, the project has contributed to the initiation of a study concerning the possibility of improving slope stability investigations by implementing geophysical test methods among the ordinary geotechnical tests. The results of this study have been presented in SGI Report No. 62.

The project has been supported by grants from the Swedish Rescue Services Agency and the Swedish National Rail Administration and by internal research funds at the Swedish Geotechnical Institute.

A large number of colleagues and companies have contributed to the special studies mentioned above, whereas SGI personnel has performed all the work presented in this report. The implementation of the project has relied on good co-operation with the involved landowners. The authors wish to express their gratitude to all private landowners and municipalities who have put the land at our disposal and contributed different types of information.

Linköping, March 2003

The authors

Contents

Notations and symbols

a	constant
b	exponent
c'	cohesion – shear strength parameter in effective stress analysis
c_u	undrained shear strength
CPT	cone penetration test
CRS	constant rate of strain oedometer test
DMT	dilatometer test
DPPR	differential pore pressure ratio – pore pressure parameter from CPT test
DSS	direct simple shear test
E_D	dilatometer modulus – base parameter in dilatometer test evaluation
F	calculated safety factor
f_T	total sleeve friction in CPT test
HfA	dynamic probing test according to Swedish A-method
$I_{D(corr)}$	corrected material index – base parameter in dilatometer test evaluation
I_L	liquidity index
I_P	plasticity limit
K_D	horizontal stress index – base parameter in dilatometer test evaluation
K_0	coefficient of earth pressure – σ'_h/σ'_v
N_{KT}	cone factor used in evaluation of CPT test
OCR	overconsolidation ratio – σ'_c/σ'_v
p_0	lift-off pressure at start of expansion of the membrane in dilatometer test
p_1	pressure at full expansion of the dilatometer membrane
q_T	total cone resistance in CPT test
R_f	friction ratio in CPT test – f_T/ q_T
u	pore water pressure
u_0	in situ pore water pressure
Δu	generated excess pore water pressure in CPT test
w_N	natural water content
w_L	liquid limit
w_P	plastic limit
w_N/w_L	quasi liquidity index

ϕ'	friction angle – shear strength parameter in effective stress analysis
μ	correction factor for field vane test
μ_{OCR}	correction factor for field vane test with respect to overconsolidation ratio
μ_{WL}	correction factor for field vane test with respect to liquid limit
σ'_c	preconsolidation pressure
σ'_h	effective horizontal pressure
σ_{h0}	total horizontal pressure in situ
σ_{v0}	total vertical pressure in situ
σ'_{v0}	effective vertical pressure in situ
σ'_v alt σ'_0	effective vertical pressure
τ_{fu}	undrained shear strength
$\tau_{fu\ ACTIVE}$	undrained shear strength in active shear
$\tau_{fu\ PASSIVE}$	undrained shear strength in passive shear
τ_v	uncorrected shear strength value from field vane test

Notations and symbols on plan drawings

 Static sounding test

 Dynamic probing test

 Pore pressure measurement station with closed system

 Pore pressure measurement station with open system

 Undisturbed sampling

 Field vane test

 Dilatometer test

 CPT test

Summary

In this project, the result of stabilising measures in terms of excavations at the crest of slopes has been studied at three places in western Sweden. At one of these places, two separate areas have been investigated and the study thus includes four different slopes. The crests of all these slopes were excavated 10 – 15 years before the investigations in this project.

The study has comprised mapping of the areas and their geological history, an examination of previous geotechnical investigations and the basis for the design of the stabilising measures performed as well as possible observations or measures taken thereafter. New investigations have then been performed in order to establish the present conditions in the slopes regarding the pore pressure distribution, shear strength and stability situation. The investigations have been aimed at establishing the present stability conditions and both quantifying the improvement in stability and detecting changes in the soil properties because of the unloading. The areas have also been inspected with respect to the conditions of the present slopes and erosion protections, the condition of the ground surfaces within the excavated parts and the vegetation that has been established. The results of the investigations have been presented in detail in three separate chapters.

The results that appeared during the course of the investigations have initiated two new studies. The first of these studies concerned the influence of the type of equipment and the vane dimensions on the results from field vane tests, and the second the possibilities of rationalising slope stability investigations by using geophysical methods as supplements to the ordinary geotechnical methods. The detailed results of these studies have been reported separately in SGI Varia No. 509 and SGI Report No. 62 respectively.

The investigations have led to a number of conclusions and recommendations regarding the design of excavations at crests of slopes and the concomitant stability calculations. They have also led to recommendations as to how slope stability investigations should be performed and what methods should be used. Methods for

prognostication of pore pressure distribution and changes in shear strength after an excavation have been tried out and as far as possible compared to the real outcome.

The results of the study have called into question the way in which the design of excavations at slope crests has hitherto normally been carried out. This concerns both the resulting stability conditions and the environment in the area. Furthermore, the results have raised serious questions regarding the methods normally used for evaluation of shear strength in overconsolidated soil in general and in slopes created by excavation and erosion in particular. This has led to recommendations for revised evaluation methods for field vane tests, CPT tests and dilatometer tests in overconsolidated soils.

The results of the investigations illustrate how the properties in different parts in a soil mass depend on the geological history of the site and how a detailed model can be built up with the aid of this.

The accumulated results have been synthesised and discussed in a separate chapter. Thereafter, more concise recommendations are given regarding investigation techniques, calculation methods and design of excavations at slope crests in order to increase the stability. Finally, comments are given on the need for further research in this area.

Chapter 1.
Introduction

Background to the project

Slips and slides are common occurrences in natural clay slopes in Sweden. They are usually results of the ongoing geological process with isostatic uplift of the land and erosion, which creates high and steep slopes, primarily towards watercourses. When this process proceeds, slides normally occur sooner or later when the available shear strength is exceeded. Different kinds of human activities can affect the process and speed it up or alternatively retard it. The slides can be small and superficial slips and then only constitute a minor adjustment, resulting in a new temporarily stable condition for the slope. They can also be dramatic landslides, particularly in quick clays, causing entire basins of fine-grained sediments to flow out and a radical change of the landscape. All forms in between these limits also exist.

On account of natural causes, there are thus a large number of slopes with unsatisfactory stability, which pose risks for existing or planned buildings and other constructions in the area. If there are such constructions of importance, stabilising measures for the slope have to be taken in order to achieve an acceptable risk level. The ongoing erosion process also has to be stopped in order to avoid a renewed worsening of the situation.

Stabilising measures can be of many different types, which in principle can be divided into geometrical flattening of the slope by earth-moving and soil-reinforcement methods. The latter involve different construction elements being installed into the soil mass, such as sheet pile walls, soil-nails, retaining walls, piles or columns with reinforced soil. They are all generally more expensive than flattening by earth moving and are therefore primarily used when the available space is limited or when valuable existing constructions have to be preserved.

Different types of drainage systems can also be regarded as soil-reinforcement methods. These are primarily effective in parts of the soil mass with overconsolidated material where the drained shear strength is governing the stability.

Flattening of slopes can be done by excavation at the slope crest, filling at the toe of the slope or a combination of both these actions. The intended result is that the average inclination of the slope is reduced and/or that the height of the steepest part is reduced and that the degree of mobilisation of the shear strength in the slope is thereby reduced. A larger fill at the toe of the slope often involves existing watercourses having to be moved or put in conduits. Besides costs and practical problems, this also brings drawbacks for water transport and any fish life and shipping. The hitherto most commonly used method has therefore been to excavate at the crest of the slope and thereby reduce the height of the steepest part and also the average inclination of the slope as a whole. For practical reasons, and also to limit the width of the excavated area and thereby the influence on existing gardens, buildings and other constructions, the excavation has mostly been performed as a single terrace with a horizontal surface and a steep slope at the back.

The fact that the measured increase in shear strength with depth has been very small or non-existent has often been a problem in the design of stabilising measures in areas with very thick deposits of "normally consolidated" clay. This is contrary to the established models for the general behaviour of clays, according to which the shear strength should increase more or less continuously with depth. To what extent this lack of shear strength increase can be attributed to the methods employed for measuring the shear strength, usually field vane tests, cannot be readily estimated. However, it has in many cases resulted in very extensive excavations.

Until the end of the 1980s, stabilising measures in terms of flattening of clay slopes were designed almost solely based on undrained total stress analyses, with the assumption that the shear strength remained unchanged after an unloading. According to this type of analysis, the normal type of excavation described above is very effective. The general validity of the method of analysis started to be seriously questioned in the middle of the 1980s, (e.g. Leroueil et al. 1983, Larsson 1983 and 1984). However, new rules for design only came into general use in 1995 when the guidelines and recommendations for slope stability investigations and stabilising measures were published by the Swedish Commission on Slope Stability, (Commission on Slope Stability 1995 and 1996, a and b). Since then, both drained and combined analyses are used together with undrained analyses in all clay slopes. According to these analyses, the use of excavations at the crest of the slope in the usual manner is not very effective in all circumstances. The result depends greatly, among other things, upon the resulting pore pressure distribution in the ground. This was not at all considered in the undrained analyses. Furthermore, it has to be expected that the undrained shear strength will also be changed as a result of the excavation. It is thus very uncertain whether the intended stabilisation really has been achieved by the measures taken.

The recommendations of the Commission of Slope Stability were partly based on theoretical calculations and apprehended reductions in shear strength, whereas there was a general lack of well-documented observations from real cases. The recommendations also hinted that the limitations for the common method of excavation mainly referred to overconsolidated soil. However, all soils become overconsolidated after removal of overburden load by excavation or natural erosion. The limitations may therefore be considered to be generally applicable.

Purpose of the study

The purpose of the project was to obtain answers to the uncertainties regarding the effectiveness of the method of excavation at the crest of clay slopes. One goal was thus to study how the pore pressure distribution in slopes adapts with time to the new situation after an excavation and how it then varies seasonally. Another goal was to find out to what extent the shear strength in the soil has changed when it has adapted after a long time to the new stress situation after an excavation. A further goal has been to investigate in more detail the increase in shear strength with depth in deep clay profiles.

These results of these investigations should provide a better basis for new stability analyses using all types of analyses and thereby a calculation of the increase in stability that has in reality been achieved by the measures taken. A more detailed determination of the shear strength properties in the soil should also show to what extent effects of shear strength anisotropy in the clays compensate for a lower stability according to these calculations. Such effects were not included in the previous analyses either.

Beside this specific check of what stabilising effects had really been achieved, the project was also intended to provide better knowledge about the general behaviour and properties of clay soils under different conditions.

The project did not aim at providing a basis for assessments of what use of the land and what activities could be allowed and performed in the specific investigated stabilised areas in the future. However, several general conclusions regarding the design of excavations at slope crests with regard to environmental aspects have also emerged during inspection of the areas and the course of the investigations.

Furthermore, new aspects have emerged on the determination of the geometrical extent of the soft soil layers and on the execution and evaluation of different methods of determination of shear strength in situ. This was not part of the original purpose either, but became essential for the implementation of the project and for the conclusions and assessments that could be made.

Scope of the investigations

The investigations have been performed at three places in western Sweden with different geometrical conditions. The first area, Torp, is located at the southern end of the municipality of Munkedal and has a slope about 20 m high down to the river Örekilsälven caused by erosion. The crest of this slope has been excavated over a stretch of several hundred metres. The depth of the excavation varies between 4 and 9 metres and its width varies between 30 and 70 metres. The deepest excavation here has been performed as two terraces. In this area, two sections about 200 metres apart and with very different geometries have been studied.

The second area has a slope about 10 m high down to the river Göta älv. It is located at Strandbacken in the municipality of Lilla Edet. Here, an excavation about 200 m long and about 50 m wide with a depth of about 4 m has been performed at the crest. A section located approximately at the centre of this area has been studied.

The third area is located in the village of Sundholmen and has a slope about 5 m high down to the small river Viskan. In this area too, an excavation about 200 m long has been performed at the crest of the slope. Its depth is only about 2 m and its width about 20 m. The size of the excavation was limited by the risk of flooding at extreme highwaters and the requirement that existing buildings should remain. The excavated area is mainly used as gardens, and certain measures have been taken to provide surface drainage.

All of the areas are located in valleys with thick deposits of clay in which a larger watercourse has eroded its channel down through the loose upper soil layers. The slopes are thus caused by erosion and end in watercourses at their toes. This is the normal case for slopes in which excavations have been performed at the crests in order to increase the stability. In all of the investigated areas, the geological process has also involved the clay layers in the central parts of the valleys being overlaid by coarser lateral fluvial sediments and delta sediments. These sediments have later been eroded away in the channels for the watercourses. The superficial layers at the crests and in the vicinity behind these thus consist of coarser and more permeable soil. The excavations performed have mainly been made in this type of soil.

For all cases, the excavations have been supplemented with a construction of an erosion protection at the toes of the slopes. The measures were taken at the end of the 1980s and it may be assumed that a sufficiently long time has now elapsed for the soils to adapt to the new stress conditions.

In the different areas, an inventory has been made of existing investigations, tests and calculations before the excavations and the results have been put together. The areas have been inspected and the ground conditions and environments have been documented. New geotechnical investigations have then been performed including penetration tests, other in situ tests, pore pressure measurements, samplings and laboratory tests. These investigations have been made comprehensive enough to clarify the conditions in all parts of the slopes. At location of the investigated sections and boreholes, attempts have been made to place them close to previous investigation points to enable a direct comparison between the conditions before and after the excavations.

The investigations have involved undisturbed sampling and field vane tests to at least the same depths as in the previous investigations. This has enabled comparison of results obtained by the same test methods before and after the excavations. The penetration test methods used in the previous investigations were too coarse to make a comparison between penetration resistances meaningful. In the new investigations, CPT tests of the highest class of accuracy have been used, i.e. Class CPT-3 according to the designation by the Swedish Geotechnical Society. These tests have provided both a more detailed picture of the stratifications and supplementary measures of the undrained shear strength and its variation with depth. Dilatometer tests have also been performed at some investigation points. The results of these tests have often been found to be less sensitive to disturbances when penetrating layered soils, and they also provide a measure of the horizontal stress conditions in the soil. All field tests have been performed in accordance with the recommended standards of the Swedish Geotechnical Society, SGF (1993–1996).

The routine tests in the laboratory have been supplemented with a large number of direct simple shear tests, which have given an alternative measure of the undrained shear strength. The results of these tests provide a control and local calibration of the empirical factors that are used for correction of results from field vane tests and fall-cone tests, the empirical cone factors used in evaluation of CPT tests and the empirical relations used for evaluation of dilatometer tests. Series of direct simple shear tests have also been run for determination of the factors required to express the relation between undrained shear strength, preconsolidation pressure and current effective overburden pressure for the particular soils in the slopes. A number of active and passive triaxial tests have also been performed in order to check the applicability of the empirical relations that are normally used to estimate the undrained shear strength anisotropy and the effective strength parameters.

The stress history of the soils and its variation in different parts of the slopes has been investigated by a large number of oedometer tests in the laboratory. These results have also been linked to the results of the CPT and dilatometer tests, which enabled a control and revision of the methods for evaluation of preconsolidation pressure and overconsolidation ratio from these tests.

All laboratory tests have been performed in accordance with existing Swedish standards. A few test methods for which there is no such standard have been performed according to the guidelines of the Laboratory Committee of the Swedish Geotechnical Society or some other well-established procedure.

The permeability of the soils has been determined in the oedometer tests, which were run as "constant rate of strain" tests. In one of the slopes, this has been combined with permeability measurements in situ by "falling head" tests in open pore pressure measurement systems installed in the ground.

The current pore pressure distributions have been modelled using an advanced numerical calculation method with support from the pore pressure measurements performed. The possibility of making a reasonably accurate prognosis beforehand of the pore pressure distribution in a slope resulting from an excavation at the crest has also been assessed. The factors of safety for the slopes against failure before and after the excavations have been calculated using undrained, drained and combined analyses with and without taking effects of anisotropy into account. The improvement of the stability achieved because of the excavations has then been analysed.

Due to the results that have come forward during the course of the project, a number of extra investigations have been added. The results obtained from the field vane tests, particularly in Torp, led to a larger study regarding the influence of different factors on these results. That study eventually came to comprise results from many more places than those involved in this report, and its results have been presented separately (Åhnberg et al. 2001). Because of shortcomings in the traditional geotechnical investigation methods that were found in Torp, among other places, a project was started to elucidate the usefulness of geophysical methods as aids in slope stability investigations. The results of this project, together with guidelines for how such methods can be implemented at various stages of the investigations, have been presented by Dahlin et al. (2001). The results of the first pore pressure measurements led to a comparative study of the pore pressure variations measured by open systems with inner hoses and closed systems installed in a thick clay layer. The results of this study are shown in this report. Finally, the investigations performed have offered a unique opportunity to study the influence of overconsolidation ratio on the results of different test methods. Together with results obtained in a previous investigation in clay till (Larsson 2000), this has led to a revision of the evaluation of field vane tests, CPT tests and dilatometer tests in overconsolidated soil. The new interpretation methods are presented in this report.

Kapitel 2.

Torp, Munkedal

2.1 DESCRIPTION OF THE AREA

The investigated area with a slope with an excavated crest constitutes the southern end of the municipality of Munkedal. In different contexts, it has also been called Torp övre, Kviström södra and the Åtorp area. The area is located on the west bank of the river Örekilsälven and extends from the Kviström bridge in the north to the Åtorp manor house in the south, Fig. 1.

The area is located in the Örekilsälven river valley, about 2 kilometres north of the river mouth in the Saltkällefjorden fjord in the northern part of the province of Bohuslän. The area was originally a plateau of sediments between the surrounding mountain ridges, which rise to a height of about 70 m above sea level on both sides of the plateau. The plateau nowadays has a level about 20 m above the mean sea level. Through the years since the plateau rose above sea level, the river has eroded its winding channel down through the sediments and created steep slopes on the riverbanks. This has resulted in numerous slides in these slopes. Most of these have been relatively shallow slips extending only a few metres in from the crest. However, the topography of the area indicates that larger slides have occurred too, among them a large quick clay slide in the northern part of the area.

Just south of the Kviström bridge, the river bends from running in a south-easterly direction to flow towards the north-east. The topography of the ground south-west of this bend indicates that a quick clay slide has occurred with its outflow in the river bend. About a hundred metres further downstream, the river makes a sharp turn of about 120 degrees and starts running in a south-south-westerly direction. It then continues for about 700 metres in a smooth shallow bend towards a southerly direction past the Åtorp manor house, whereupon it turns towards south-west. The area of the investigations is thus a geographically restricted, slightly protruding peninsula into the river Örekilsälven. The seasonal variation in water transport in the river is large, and the maximum variation in water level is about 3 metres.

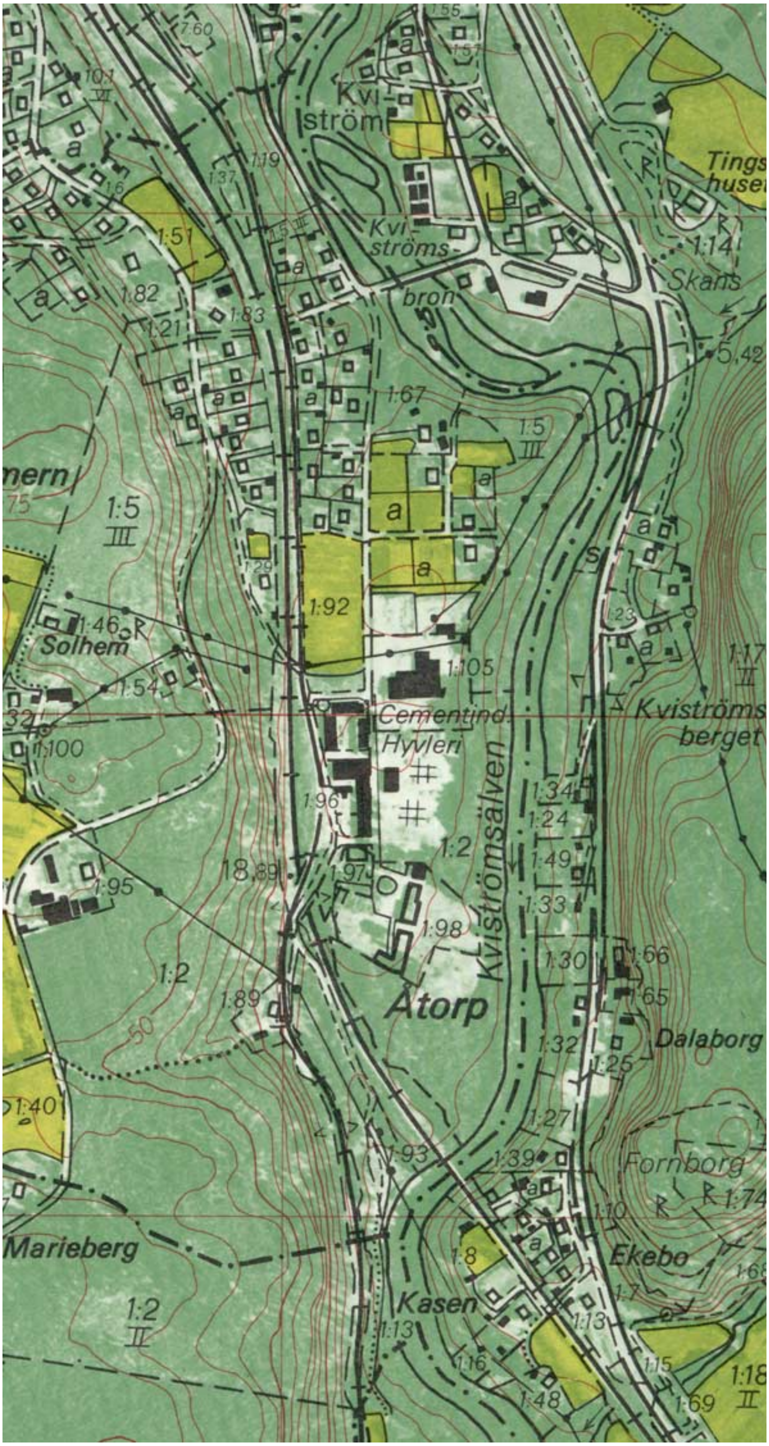


Fig. 1. Map of the investigated area in southern Munkedal from 1978. In this map, the river Örekilsälven is designated as Kviströmsälven.

The Bohusbanan railway line runs on the western side of the plateau, about 200–300 metres behind the location of the slope crest before the excavation. On the other side of the valley runs the main road between Göteborg and Oslo, the European highway E6. The buildings on the plateau today consist of the Åtorp manor house, which is a hotel and conference centre, a carpentry factory and a few dwelling houses. In earlier times, as shown in the map, there was also a cement works and a larger number of dwelling-houses, but the works has been closed since the excavation took place and the houses were demolished in connection with the stabilising measures.

The stability in the area has been investigated to a limited extent in connection with different extensions of primarily the industrial premises and Åtorp manor house. It has then been pointed out that the stability of the slopes was low and, already in the 1950s, an investigation by Caldenius suggested that no buildings should be placed closer than 100 metres to the crest for safety reasons. In that investigation, it was also proposed that the whole stretch along the river should be supplied with erosion protection to stop the ongoing erosion process. Later investigations cited and repeated these proposals, but nothing happened until a slide occurred in the slope directly beneath the main building of Åtorp manor house in 1980. This prompted an investigation that was carried out by Bohusgeo AB, which showed that the stability was clearly unsatisfactory in this part of the area. It was also pointed out that there were many sections in the area with more unfavourable geometrical conditions than in the investigated part. It was therefore apprehended that the stability was very low in other parts of the slope along the river as well.

This led to further investigations, which were carried out in co-operation between SGI and Bohusgeo AB (Swedish Geotechnical Institute 1985). Eventually, the results led to an investigation of the entire length of the slope towards the river in this area and also in a large area in the central parts of the municipality. The outcome of the investigations led to an intervention by the Swedish Rescue Service Agency in order to secure the safety in all of this extended area. The so-called "Munkedal works" comprised large excavations and earthmoving operations in the central part of Munkedal, where the whole river channel was moved (SVT 1985, NCC 1985). They also comprised large excavations and construction of erosion protection on the riverbanks in the area of the present investigation, Figs. 2 and 3. The slope directly beneath the Åtorp manor house was reshaped and flattened by a combination of excavation and filling. In connection with these works, a number of landed properties were bought up and the buildings on these were demolished. The size of the excavations was partly limited by the existing cement works, which at that time

Fig. 2. **Ongoing erosion and small slides before construction of the erosion protection.**

Fig. 3. **Carrying out of excavation works.**

was still in operation. This industry has later been closed and these buildings have also been demolished.

In the southern part of the Torp area, there was a small wood of beech trees growing in the slope and the area just behind the crest. The wood was protected by an environmental decree but it became necessary to cut it down in order to enable the implementation of the stabilising measures. A plan for replanting was created in order to re-establish the character of the area with a leafy wood consisting primarily of beech trees. The plan sought to take into account both environmental interests of re-establishing the vegetation and geotechnical interests of avoiding trees catching a great deal of wind being placed at unsuitable locations in the area with regard to local stability.

2.2 GEOLOGY

No thorough geological investigation, like e.g. those in the Göta-älv valley and at Tuve (SGI Report No 11b), has been made in the Munkedal area. Nor is the area included in any recent geological mapping with an accompanying description of the area. It is only briefly mentioned in an old survey by Lindström (1902). However, the general outline of the geological history is the same as for the Göta-älv valley and other valleys with marine clay deposits in the Göteborg and Bohuslän area.

The area was covered by inland ice during the last glaciation and the deposition of sediments started when the retreating ice front passed the area. This occurred about 12,400 years ago. Fine-grained particles in the melt-water then started to accumulate by sedimentation on top of the bedrock or on top of a layer of till which had been deposited below the ice cover. The soundings in the Torp area that have reached firm bottom layers indicate a layer of till on top of the bedrock, and the pore pressure measurements in the same section indicate that the layer is continuous. This section, designated Section A, is located in the northern part of the area, see Fig. 4. The firm bottom in this section slopes down towards the location of the river channel, and the maximum thickness of the sediments is here about 60 metres. In a section located 200 metres further south, none of the penetration tests has reached firm bottom and the maximum thickness of the sediments here can only be estimated to more than 70 metres. Later geophysical investigations have confirmed that the bedrock surface slopes downwards towards the south and have indicated that the thickness of the till layer varies from a metre or so to about 10 metres.

The sediments were precipitated in sea water, whose salt content and temperature varied with depth and the distance to the ice front. Normally, the soils in the lowest layers are coarser with infusions of silt and sand layers, but the clay content then starts to increase continuously upwards in the profile. The lowest layers also normally contain sulphides in sufficient quantities for the clay to be significantly banded, striped or flamed by black sulphide colour. At the start of the deposition, the sea level was more than a hundred metres above its present level in relation to the bedrock, but the depths gradually became shallower as the ice front moved away and the isostatic uplift of the land progressed. Postglacial sediments then started to overlay the glacial deposits. When the area had become even shallower, the river had been created in the higher areas to the north and the eroded particles transported by this started to deposit at and some distance away from the river mouth. The very fine-grained sediments then started to be overlain by new sediments, which became coarser and coarser as the shoreline and river mouth approached the present area. When these passed the area, a delta was probably created, similar to that which can be seen today at the outlet of the river into the fjord about 2 kilometres downstream. Varying water transports and water levels resulted in different kinds of lateral fluvial sediments being deposited over the area around the river mouth. The thickness and grain sizes of these sediments were in principle largest around the main river channel and decreasing sideways, but large variations occurred due to the shape of the delta and its variation during the time of deposition of delta and lateral fluvial sediments in the area. From a certain level, the clay in the deposits thus started to become gradually coarser upwards. They were then overlain by silt and sand and in some spots even by gravel. These upper deposits are normally layered. Deposits of silt and sand at the top with a thickness of more than 10 metres have been found within the investigated area. The normal picture before the excavation was that these deposits had a thickness of about 6 metres at the crest of the slopes and then gradually became thinner towards the valley sides. However, large variations have been found between the two sections in this investigation and also in the different previous investigations. The delta and lateral fluvial sediments also contain varying amounts of organic material.

In this type of deposits, it is common that seams and layers of coarser wave-washed material are found embedded in the fine-grained material along the valley sides as a result of changes in climate and sea level during the period of deposition. This has not been reported in this particular area, which may be due to the fact that no investigations have been performed close to the valley side. However, in the southernmost section in this investigation, a continuous layer of coarser soil was found at a depth of 50–55 m below the original ground surface. This layer may be assumed to be a result of a temporary re-advance of the ice front during the protracted period of deglaciation, as described by Stevens (1987).

Since the area rose above sea level, the river has eroded its channel down through the sediments and the river bottom now lies more than 20 metres below the level of the plateau behind the slope crest. A number of slides have occurred in the slopes created during this process. The slide debris has mainly been eroded away by the river and has not significantly affected the soil stratification. During the same time, leaching has gradually reduced the salt content in the pore water in the soil. This process is most pronounced for clay layers in which the distance to a draining layer is relatively short. The process has thus advanced furthest at the valley sides whereas the original salt content is better preserved in the thicker clay layers. Leaching may entail that the shear strength becomes reduced, but primarily that its sensitivity to disturbance increases and that the remoulded shear strength decreases. Quick clay has thus been found in parts of the Torp area where the clay layers are thinner, which here occur only at a certain distance from the river. However, the river runs very close to the valley side in the northern part of the area, and this is where indications of an old quick clay slide are found.

2.3 PREVIOUS INVESTIGATIONS AND STABILITY ASSESSMENTS

The investigations concerning the slope stability that were performed during the early 1980s, and which are relevant for the excavated area, were primarily located in four sections designated as 22-23, A, B and C. The location of the sections can be seen in Fig. 4. (Section B is not shown in the figure, but lies midway between sections A and C). The investigations were primarily made by Bohusgeo AB by total pressure soundings and field vane tests, and samples were taken by screw auger in the upper silt and sand layers and with the Swedish standard piston sampler in the underlying clay. A few soundings were also made as weight sounding tests with machine-driven rotation. The total pressure soundings and field vane tests were performed using light-weight Geotech equipment. This type of total pressure sounding can apply a maximum push force of 10 kN. The soundings were stopped when this force was reached and the stop level was designated as “stone, boulder or rock”. This was done without any check whether the sounding could be driven further by use of rotation. However, the few weight sounding tests that were performed generally reached deeper levels than the total pressure soundings (Swedish Geotechnical Institute 1985).

The shear strength determinations by field vane tests generally showed a relatively small increase in shear strength with depth. A closer study of the results shows that when the measured shear strength value had reached 55–60 kPa, there was a sudden decrease in the measured shear strengths. This limiting value approximately corresponds to the maximum shear strength value that can be measured with the equipment used and the normal size of the vane. When this value is reached, the normal vane therefore has to be replaced with a vane of a smaller size. Normally, this change is not expected to cause any change in the measured shear strength, but a closer study of the results clearly indicates that this happened in this soil. However, this was not revealed until the present investigation and comparisons, which in turn led to a special study of the influence of the vane size and type of equipment on the results (Åhnberg et al. 2001).

The pore water pressures were measured at one point located some distance away from the crest in Section A using a piezometer installed in the upper part of the clay layer below the overlying silt and sand layers and an open pipe driven down to the assumed firm bottom. The results showed that the free groundwater level was located somewhere in the silt and sand layers and that there was a downward gradient towards considerably lower pressure heads in the bottom layers.

The soil samples were investigated in the laboratory concerning classification, density, water content, undrained shear strength determined by fall-cone tests and sensitivity. Oedometer tests were also performed for determination of preconsolidation pressures. The results showed a stratified composition of the upper soil layers with gravel, sand and silt with organic matter gradually changing to clayey silt and silty clay with depth. The clay below was first medium-plastic and then became high-plastic with black spots from sulphide colour. In a borehole taken relatively close to the crest in Section A, the water content was generally equal to or lower than the liquid limit and the clay was medium-sensitive. This is normal for clays in western Sweden. In borehole 22, which is located further away from the river channel and the thickest clay layers, the water content was generally equal to or higher than the liquid limit and the clay was highly sensitive at many levels at this point. That the clay was highly sensitive and sometimes quick at points located closer to the valley side had also been observed in earlier investigations for other purposes. The undrained shear strengths measured by fall-cone tests in samples taken from greater depths were generally lower than those measured by field vane tests, which is normal. The results of the oedometer tests showed that the clay in the area behind the crest was normally consolidated or only slightly overconsolidated in relation to the prevailing overburden pressure.

The stability calculations using total stress analyses and isotropic undrained shear strength generally showed safety factors between 0.8 and 0.9 for the most critical slip surfaces calculated in this way. These slip surfaces extended 10 to 20 metres behind the crest and deeper than the bottom of the river. In some sections, the assumed firm bottom restricted the depth of the critical slip surfaces. The lower ends of the critical slip surfaces coincided with the submerged toes of the slope at the river bottom. The discrepancy between the calculated safety factors and the theoretical minimum value of 1.0 was assumed to be due to the fact that the shear strength anisotropy had not been taken into account. However, no attempt was made to quantify this effect. This was not considered to be necessary since it was obvious that the safety factor was close to 1.0 and that the stability was highly unsatisfactory.

A calculated safety factor of at least 1.3 to 1.5 is normally desired in built-up areas, depending on what restrictions are applied for construction and use of the land in the area. In designing stabilising measures for slopes with a calculated safety factor of 1.0 or below, this means a required increase in calculated safety factor by 30 or 50 % respectively. Proposals for designs fulfilling the two different requirements were drawn up. The final extent of the stabilisation appears to vary between these proposals, probably as a compromise governed by small variations in the ground level, variations in the slope and crest, what landed property was decided to be bought up and the continued operation of the cement works. In principle, the demands for a calculated safety factor were set to 1.5 for remaining dwelling houses and to 1.3 for the other ground and the cement works (Bergdahl 2002).

2.4 STABILISING MEASURES

The extensive excavations at the slope crest in the Torp area in order to ensure satisfactory safety against slope failure were performed in 1985. At the same time, an erosion protection of blasted rock-fill was constructed on the riverbanks for the entire roughly 600 metre long distance between the first river bend just south of the Kviström bridge, around the protruding point of land and southwards down to Åtorp manor house. The major part of the excavation was made about 5.5 metres deep. The excavation depth varies somewhat and is smallest in the northern part of the area and then gradually increases towards the south. This roughly corresponds to the difference in slope height even though this, and thereby also the excavation depth, varied somewhat locally. The width of the excavation varies with the earlier topography and contour of the crest line. It is mainly between 25 and 50 metres. Within a small slightly elevated area located directly behind the new upper crest, the ground was also scraped off and lowered about a metre to become level with the surrounding ground.

Over a somewhat more than 100 metre long distance in the southern part of the area the excavation was increased and became a width of totally about 70 metres including the upper slopes. It was here performed in two terraces, an upper one with a width of about 40 metres and an excavation depth of about 5.5 metres and a lower one about 20 metres wide and an excavation about 4 metres deeper. The slopes from the new crests down to the terraces were given inclinations of about 1:2. The excavated terraces in the eastern part of the area were given inclinations of only 1:50 and thereby became almost horizontal planes. The terrace in the northern part of the excavated area was given a roughly twice as large inclination, but the ground surface still gives the impression of being almost horizontal.

Directly in front of the Åtorp manor house, there was a steep, almost 10 metre high slope towards the south-east, which then flattened out in its further run towards the river. The stability problem here was limited to the upper slope. The stability of this slope was secured by an earth fill spread over a larger area at the toe of the upper slope. The average thickness of this fill was about 2 metres.

An erosion protection was constructed on the riverbanks. First a fill of stone and finer soils was laid out at the toe of the slope until the shore line was evened out and an inclination of 1:2 was obtained from the river bottom to a level about 3 metres above the mean water level. The outside of the fill was covered by a 0.7 meter thick shield of rock-fill and its top surface was covered by a 0.3 metres thick fill of clayey soil.

2.5 RESTORATION OF THE VEGETATION

Almost all trees in the area were cut down in connection with the stabilisation works. This was also done to a large extent in the remaining lower parts of the slopes since this was necessary to allow the construction of the erosion protection. Many large trees were also considered to constitute stability problems because of their large surfaces exposed to winds, which can be very strong seasonally along the west coast of Sweden.

For environmental reasons and to avoid surface erosion, it was considered important to rapidly re-establish a cover of vegetation. In the southern part, the protected wood of beech trees had been cut down and it was considered to be very important to replace this and to recreate the character of a leafy wood in the area.

A comprehensive plan for replanting of the area was therefore designed. The steep excavation slopes were sowed by spraying grass seed in the same way as is recommended by the Swedish Road Administration (1984) for excavated slopes along roads. The mixture of grass seed was supplemented with lupin seeds. Large parts of the terraces were sown in this way as well. On the terraces were also planted groups and groves of different kinds of bushes and leafy trees, which were intended to grow and through time reach different heights. A cover of topsoil was laid out on these special areas before planting. Osier bushes, which become about waist-high and grow even in very wet conditions, were planted on the small terraces caused by the fills on the riverbanks. In the remaining slope, the original vegetation consisting of beech trees and other plants was expected to re-establish itself through all sprouts that were left after the felling of the trees. Supplementary planting of beech trees was also to be undertaken in the southern part of the area.

Records from the time for the planting operations and minutes of a meeting concerning the vegetation a year later show that serious problems had been encountered. It had not been possible to carry out the planting in those areas that had the smallest inclinations because of marshy ground conditions, and a large number of plants among those that had been placed in other areas had died. Some plants had also been swept down with superficial slips in the slopes that had been laid bare. An unexpected amount of thicket and other unwanted growth had also been established on the excavated slopes. Clearing up, resowing and extensive re-planting was therefore planned.

The use and nature of the upper terrace in the southern part of the area had also been altered from intended woodland to a parking area during the stabilisation works. During the excavation work, a large flow of water started out of the upper excavated slope and the upper terrace became exceedingly wet. The upper excavation slope therefore had to be equipped with a material separation filter and a cover of rock-fill in order to prevent internal erosion. The upper terrace was first lowered about 0.9 metres further than originally intended and then supplied with a 0.6 metre thick base for the parking area. Rock-fill was used as base material because of the wet conditions. A thin cover of mixed coarse and medium-grained soil was then applied as pavement.

What has happened thereafter with the revegetation programme and any changes in use of the land cannot be seen in the available records.

2.6 NEW INVESTIGATIONS

2.6.1 Location

The new investigations have been made with the purpose of studying in detail the effects of the excavation in terms of changes in pore pressure conditions and shear strength and the related development of the stability situation. Other possible aspects of the design and the results of this type of stabilising measures should also be looked into. The investigations were therefore located in those sections and test points in the excavated area where the situation before the excavation had been investigated in most detail. The previous investigations in this area by Bohusgeo AB had been located in Sections A, B and C, where total pressure sounding, field vane tests by lightweight Geotech equipment and piston sampling had been carried out. In Section B, only total pressure sounding had been performed. The pore pressure had been measured at two levels in one point in Section A. Supplementary investigation had been performed by SGI in the northern part of the area after the planned excavation had been extended to include the slope towards the north as well. Both field vane tests using SGI-type equipment and piston sampling had been performed in point 22 within this part of the excavated area (Swedish Geotechnical Institute 1985).

The new investigations were therefore performed in Sections A and C with supplementation of new field vane tests at point 22. Section A is located approximately in the middle of the eastern part of the excavated area and Section C is located in the south-eastern part where the excavation was performed with two terraces at different levels. The distance between the two sections is about 200 metres. The points for the new investigations were as far as possible located close to the previous investigation points. The number of investigation points was also extended in order to allow comparisons between the conditions in natural ground behind the excavated area with those in the slope. Investigations were also performed below the river bottom to enable a similar comparison with the conditions in those parts of the area where the geological process had brought the largest unloading, Fig. 4. In order to differentiate between old and new investigation points, the letter S has been added to the designations of the latter.

The field investigations were performed in two stages. The first round of field investigations and sampling was performed in the autumn of 1996 and comprised investigations in Section A and point 7 in Section C. The latter point is located below the river and was investigated in this round because the rented raft was then in place.

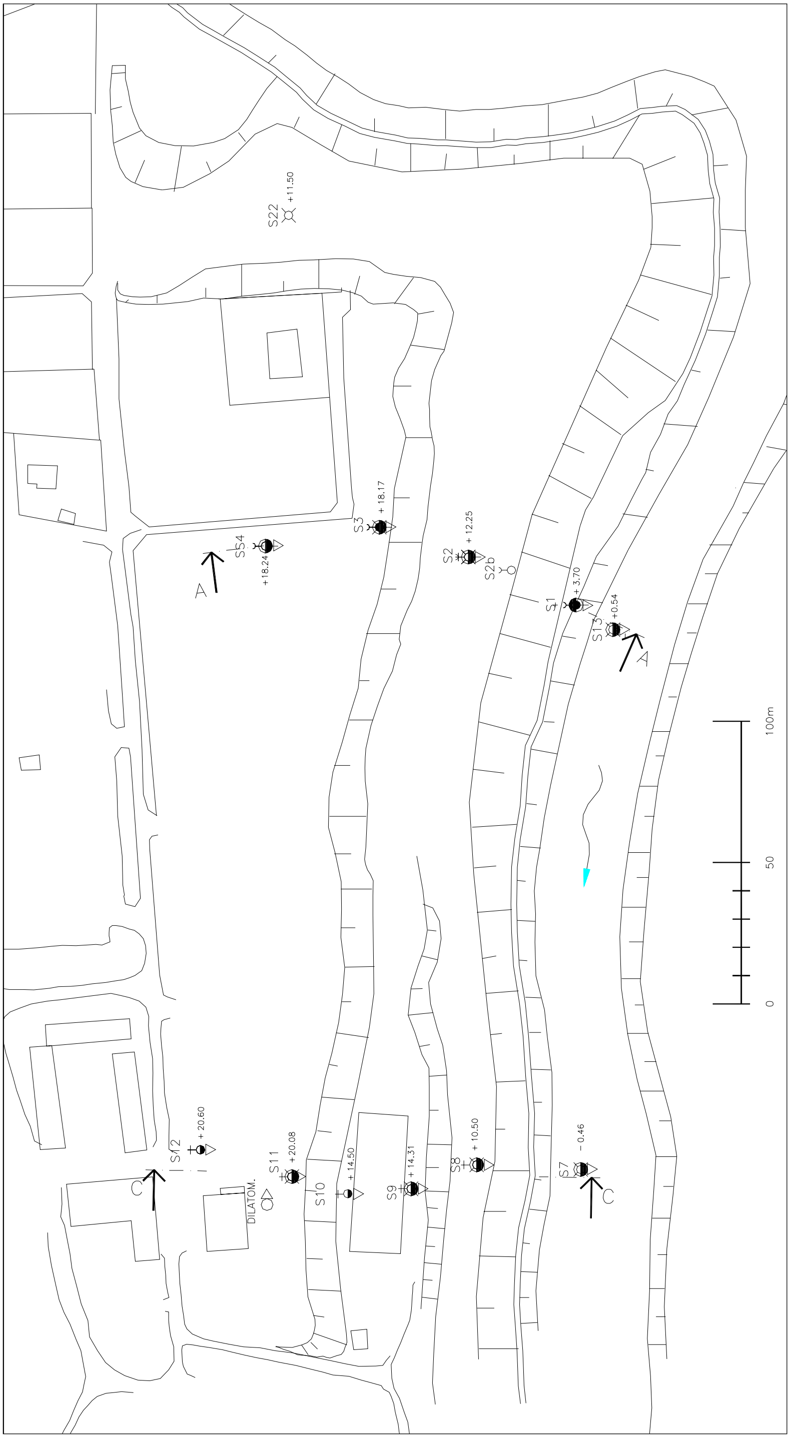


Fig. 4. **Plan of the new investigations.**

The second round of investigations took place in the autumn of 1999. It comprised the rest of the investigations in Section C and supplementary investigations in Section A, which in the meantime had been found to be required. Field vane tests were then also performed in point 22.

Pore pressure measurements have been performed in a large number of points and depths in both sections, enabling a study of the pore pressure distribution in the whole soil mass. Observations have been performed regularly for several years to study seasonal variations as well.

2.6.2 Observations from an inspection of the area

The visual inspection of the area showed that the constructed erosion protection has functioned well. The water transport in the river varies heavily in spite of being partly regulated, and can seasonally be very high. The current is then very strong and the water level reaches almost up to the crest of the erosion protection. However, no damage to this or minor slides or slips along the riverbanks can be observed. The vegetation is fairly abundant in most of the remaining natural slope below the eastern part of the excavated area. It partly originates here from the time before the excavation because the stabilisation works affected parts of this slope only to a limited extent. The clumps of trees consist mainly of birches and alders, Fig. 5.

The vegetation on the excavated areas consists mainly of grass and different kinds of sedges. The ground is generally marshy and for most of the time there is free water or ice at the back of these areas. The excavated areas on the eastern slope were given a very small inclination and here the major part of the vegetation is of the type growing in marshes and fens. The excavation towards the north was given a somewhat larger inclination and most of this area is covered by grass. However, the conditions are wet at the intersection between the excavated area and the excavation slope at its back because of water seeping out of the slope. This is clearly indicated also by a band of sedges growing along this intersection, Fig. 6. That planting of trees has been performed can only be observed in a few isolated groups of trees. There is one on the eastern excavated area at a point where the ground is slightly elevated and the conditions thereby somewhat drier. There are also a couple on the northern area in points where the ground is similarly elevated or close to the crest of the slope towards the river and consequently relatively dry. A few smaller groups of birch trees have also been established on the excavated area and in the excavation slopes. However, these are not planted but from natural growth, Figs. 7 – 9.

Fig. 5. View of the eastern slope towards the river and its erosion protection.

The south-easternmost part of the excavation was performed in steps with two separate terraces at different levels. During the excavation works, erosion protection had had to be applied to cover the slope between the natural ground and the upper terrace to prevent the upper sand and silt deposits from flowing out because of internal erosion. The excavation at the upper terrace had also been deepened and then supplied with a base and paved. This had been done in order to enable the execution of the works and to let the terrace serve as an extended parking area for Åtorp manor house afterwards. In spite of these extra measures, the ground is still relatively soft and heavy vehicles create deep tracks, except in extremely dry periods and when the ground is frozen. The terrace has nowadays ceased to be used for parking. Instead, it is used by the local tennis club, which has constructed two hard tennis courts. This construction covers about half of this upper terrace and has involved a slight rise of the ground in order to obtain sufficient drainage and stiffness, Fig. 10. The lower terrace has the same character as the rest of the excavated area in the eastern part, with marshy ground and vegetation of mainly sedges.

Fig. 6. Band of sedges at the back of the excavated area in the north.

Fig. 7. Planted group of trees in the eastern excavated area.

a)

b)

Figs. 8 a and b. Groups of planted trees close to the crest of the northern slope and in the north-east corner. In Fig. 8b, the European main highway E6 can be seen running on the other side of the river behind the excavated area.

Fig. 9. Natural growth of birch trees in the eastern part of the excavated area.

Fig. 10. The upper terrace in the south-eastern part of the area with tennis courts. Tracks from traffic on the intended parking area are clearly visible.

The ground surface behind the upper crest is mainly horizontal. Of the former houses on the eastern side of the road leading to Åtorp manor house, only a single dwelling house remains in the northern part of the area. The other houses and the cement works have been demolished and the bought-up estates are mostly covered by grass. A few paved areas remain at the location of the cement works, but no other installations. In the south-eastern part, a small warehouse for the carpentry factory remains. The ground around this is paved with sand and gravel for transports of wood to and from the warehouse. No signs of superficial slides or cracks can be observed and have not, as far as is known, been reported apart from a few small slides in the northern slope during and shortly after the execution of the stabilising works.

2.6.3 Field tests

Penetration tests

In Section A, tests have been performed at five points numbered S4, S3, S2, S1 and S13. Point S4 is located about 50 metres behind the upper crest, Point S3 about 5 meters behind this crest, Point S2 at the centre of the excavated area and Point S1 just behind the erosion protection at the toe of the slope down at the river. Point S13 is located in the deepest part of the river channel about 12 metres outside the shoreline at mean water level, see Figs. 4 and 11. Section A is not in a straight line but bends at Point S3 in order to locate Point S4 in meadowland instead of in a paved area in the straight section.

CPT tests were performed in all points. The results of these tests showed that the upper layer of sand and silt is about 5 metres thick at Point S3, 5 metres behind the upper crest, and that it had thinned out to be about 2 metres thick 50 metres behind the crest. No silt or sand was found at Point S2, where about 5.5 metres of soil had been excavated. An original thickness of about 5 metres can thus be assumed to be representative of the upper layer over the whole distance from the present upper crest to the river channel. This assumption is supported by the results of the previous investigations, which indicate that the thickness of the upper layer was about 1.5 metres far behind the present upper crest and about 5 metres at this. However, at one point located in the excavated area those results indicate a 6 metre thick sand and silt layer.

The CPT tests have better ability to penetrate than the previously used total pressure soundings and the very first new investigations with CPT tests in Section A went considerably deeper in several points. At Points S4 and S3, the stop in the

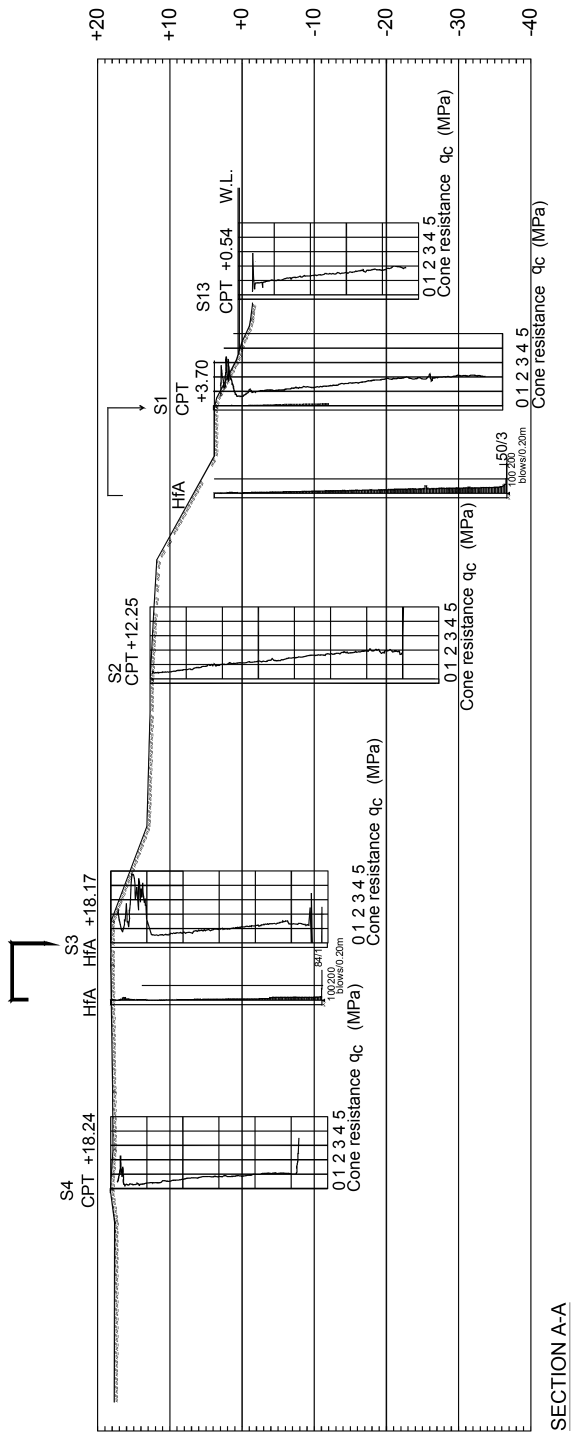


Fig. 11. **Section A.**

penetration corresponded fairly well with the previously assumed level of the firm bottom. However, at Point S2 the penetration went 13 metres deeper than the assumed firm bottom and at Point S1 the assumed depth was exceeded by 18 metres without reaching a distinct stop. The CPT test at this point was terminated when the maximum penetration force was reached at a penetration depth of 37 metres. The results also showed that the clay layer continued down to the depths reached. No stop in penetration was obtained in the test below the river either, but the test was terminated because the maximum penetration force that could be applied by the drill rig standing on the raft was reached, Fig. 12.

The increased penetration depths were somewhat alarming because the previous stops in penetration had been designated as "stop against bedrock or other firm object" and had been assumed to be lower limits for possible slip surfaces. In some cases, the calculated critical slip surfaces had reached and been restricted by these depths.

At those points in Section A where stop in penetration against firm bottom had been reached, the tests were ended by a study of the dissipation of the generated excess pore pressure after stop. These studies indicated that fairly permeable layers had been reached. A few similar studies in the overlying clay showed that this had a very low permeability.

Since the depths to firm bottom are very important, particularly below the slope, the stop levels in Points S2 and S1 were controlled by dynamic probing tests of type HfA. These tests penetrated some metres further and indicated that the stops in the cone penetration tests were obtained at the top of a layer of dense coarse soil between the clay and the bedrock.

The new investigations in Section C were performed at six points numbered S7, S8, S9, S10, S11 and S12. The reason for the larger number of investigation points in this section is that the excavation here was made in two steps with two terraces. Point S7 is located below the river and was investigated already in the first round. Point S8 is located at the centre of the lower terrace and Point S9 is located about 5 metres behind the crest above the lower terrace. From the investigation point of view, it would have been desirable to place Point 9 at the centre of the upper terrace, but this would have interfered with the tennis courts in this location. Since it would be impossible to make investigations here without causing considerable damage, the investigations were performed as far in on the terrace as possible without causing negative effects to the courts. Penetration tests, field vane tests and

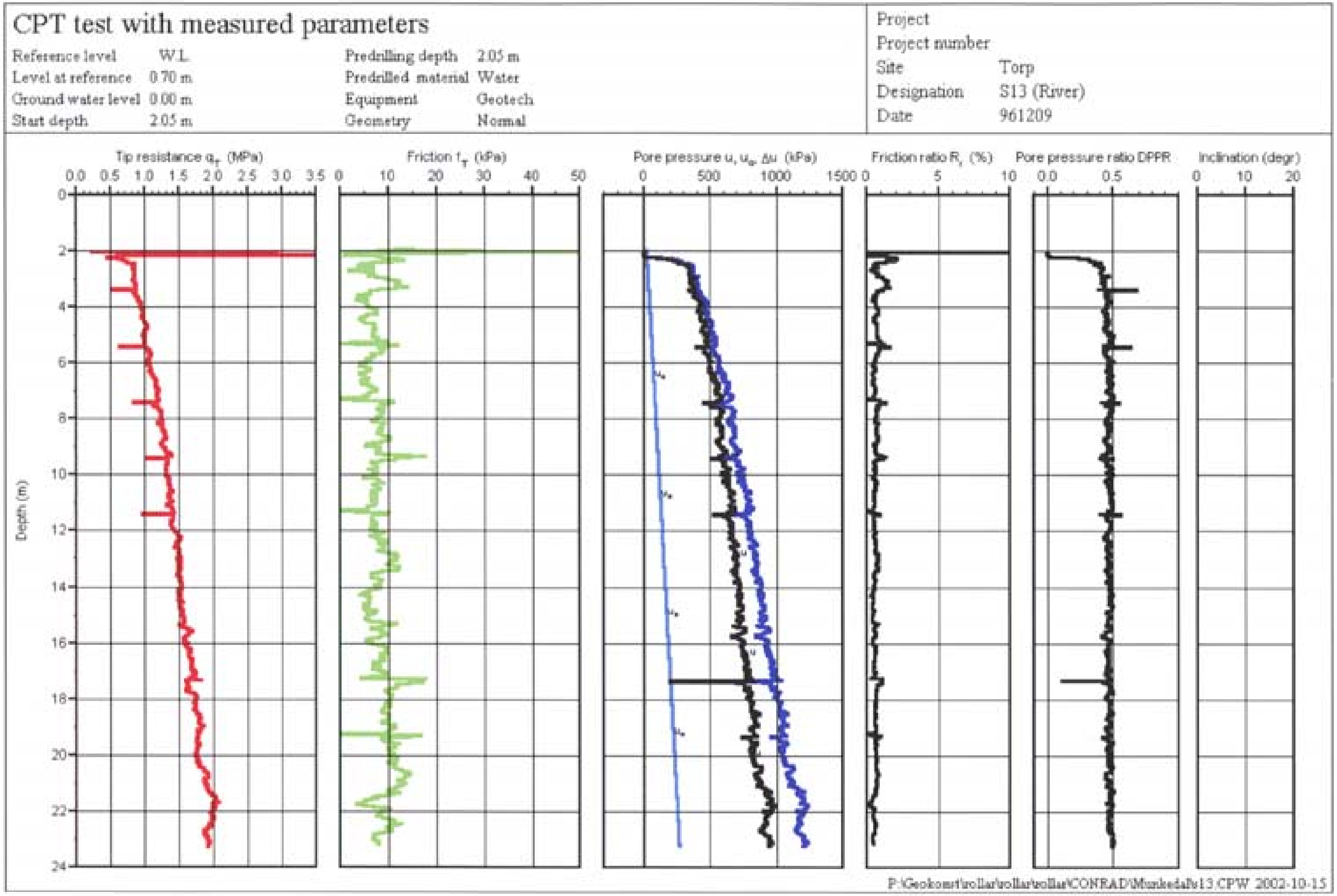


Fig. 12. Results of the CPT tests in Section A presented by use of the program CONRAD. a) Point S13

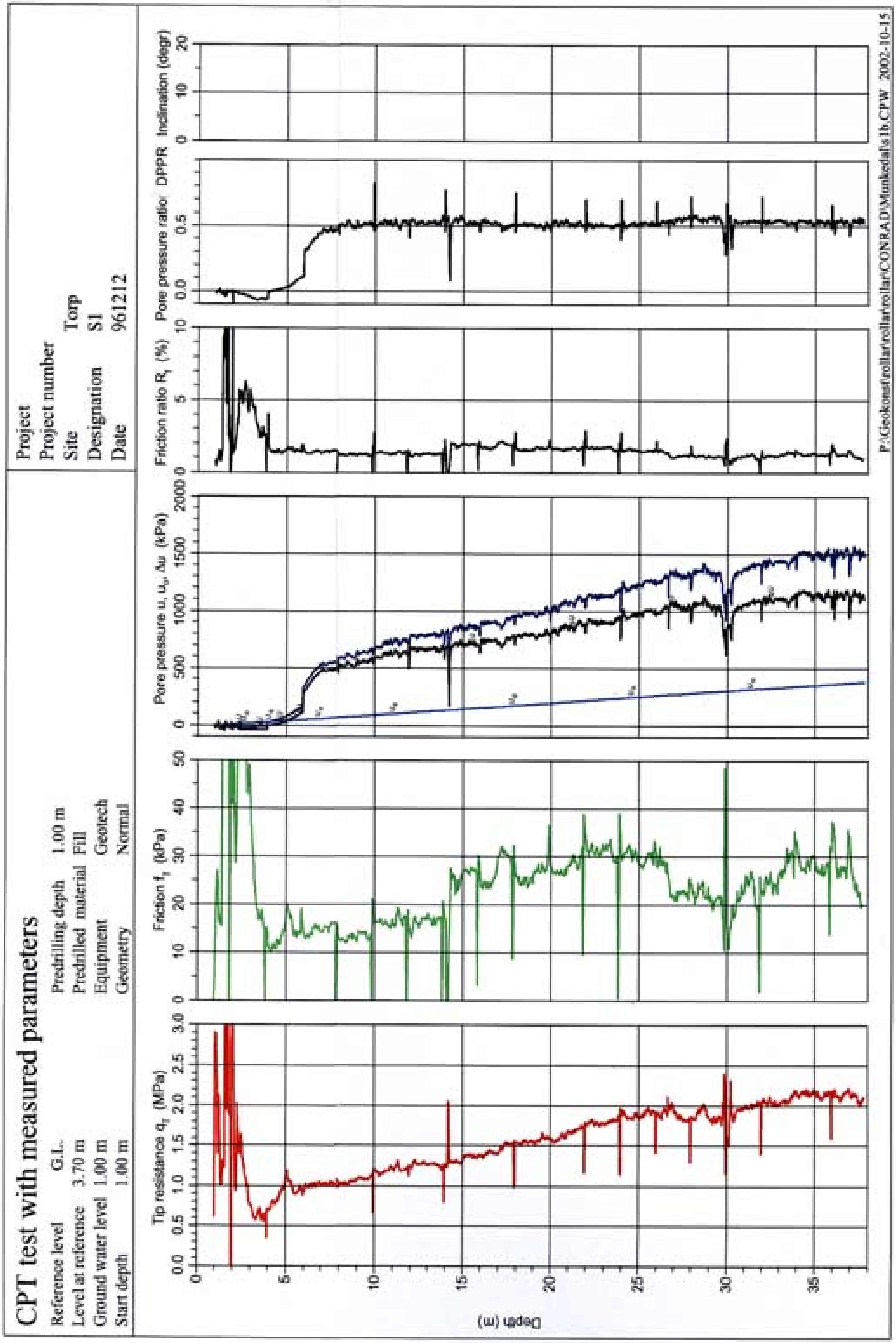


Fig. 12. Results of the CPT tests in Section A presented by use of the program CONRAD. b) Point S1

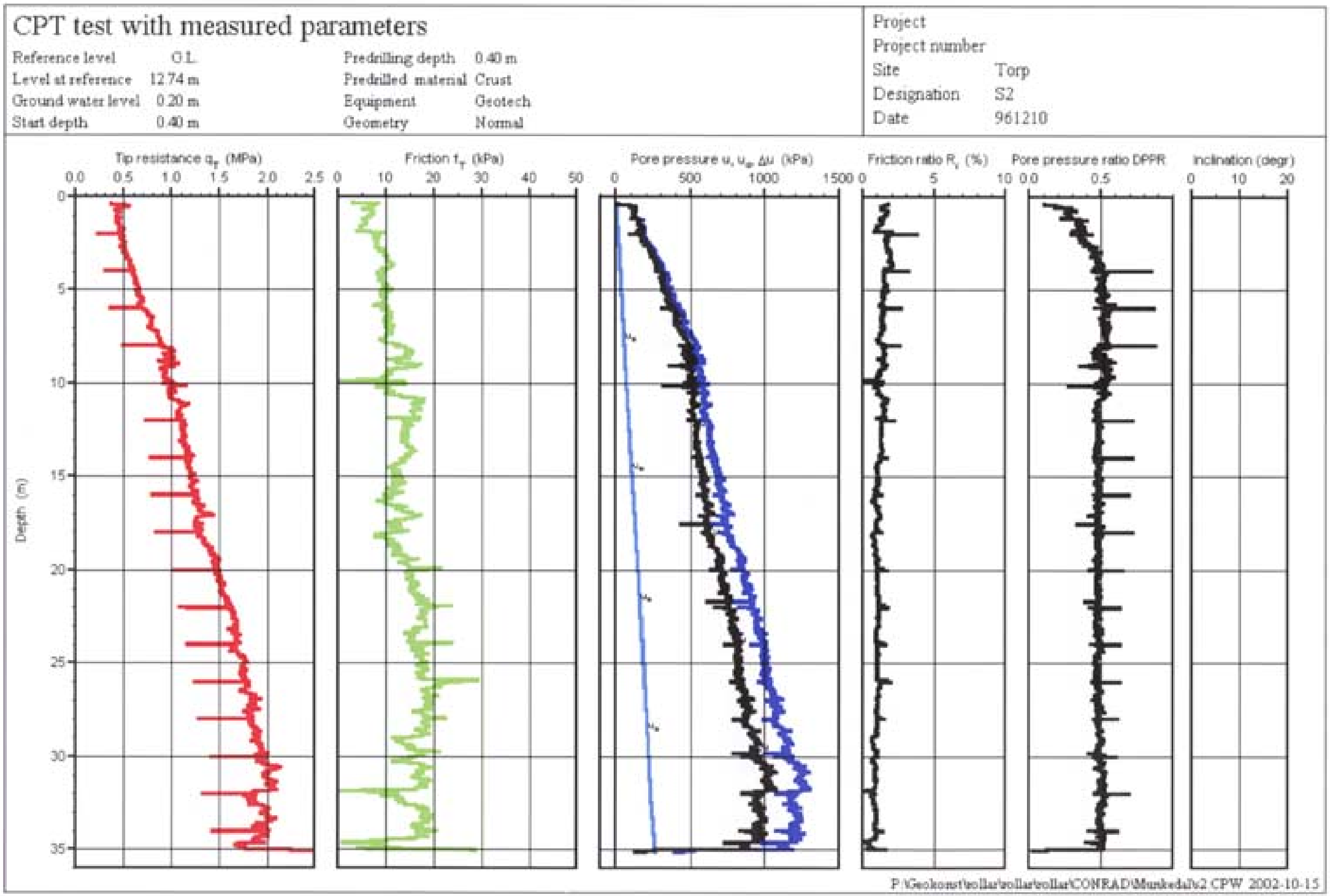


Fig. 12. **Results of the CPT tests in Section A presented by use of the program CONRAD. c) Point S2**

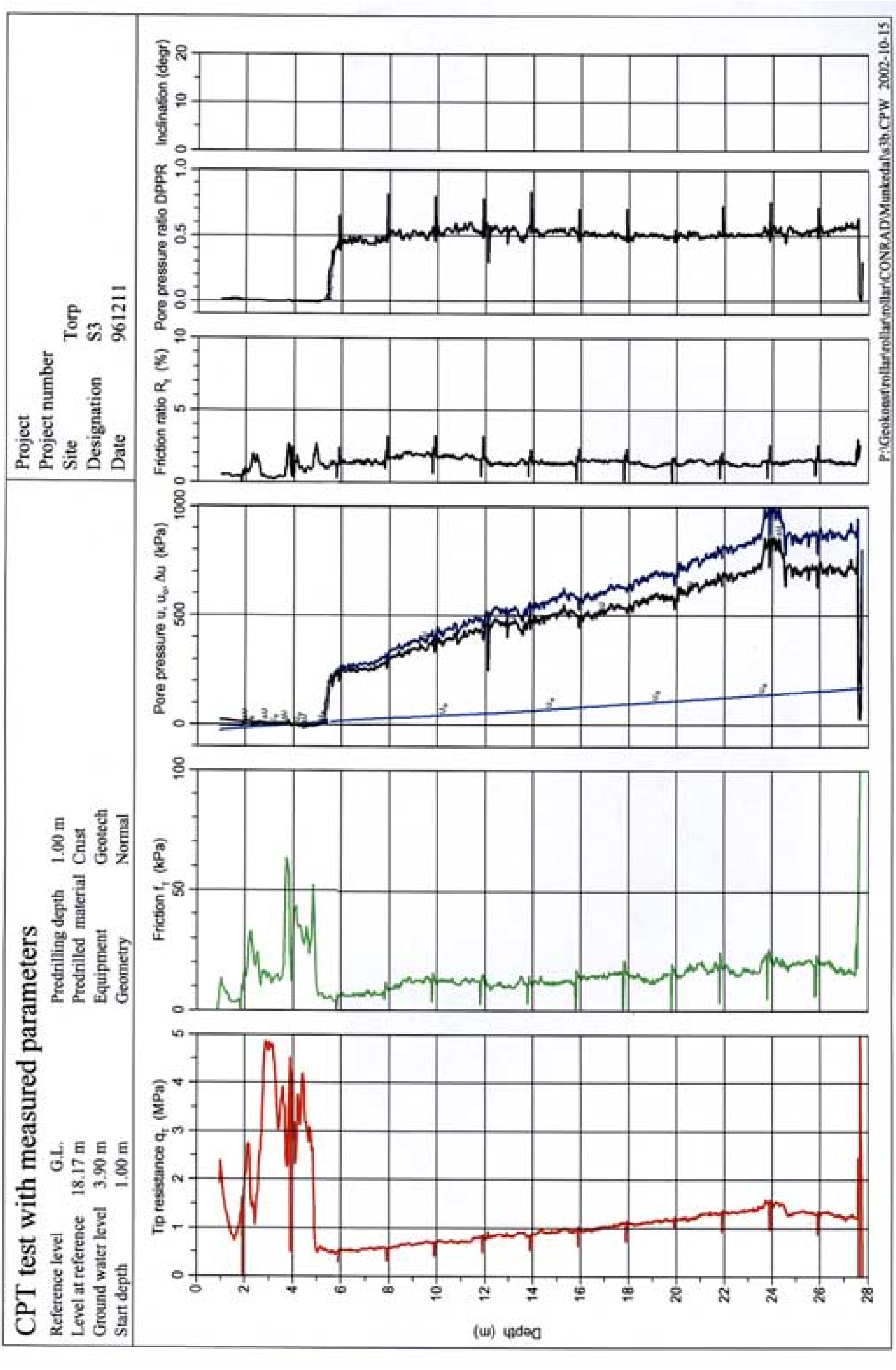


Fig. 12. Results of the CPT tests in Section A presented by use of the program CONRAD. d) Point S3

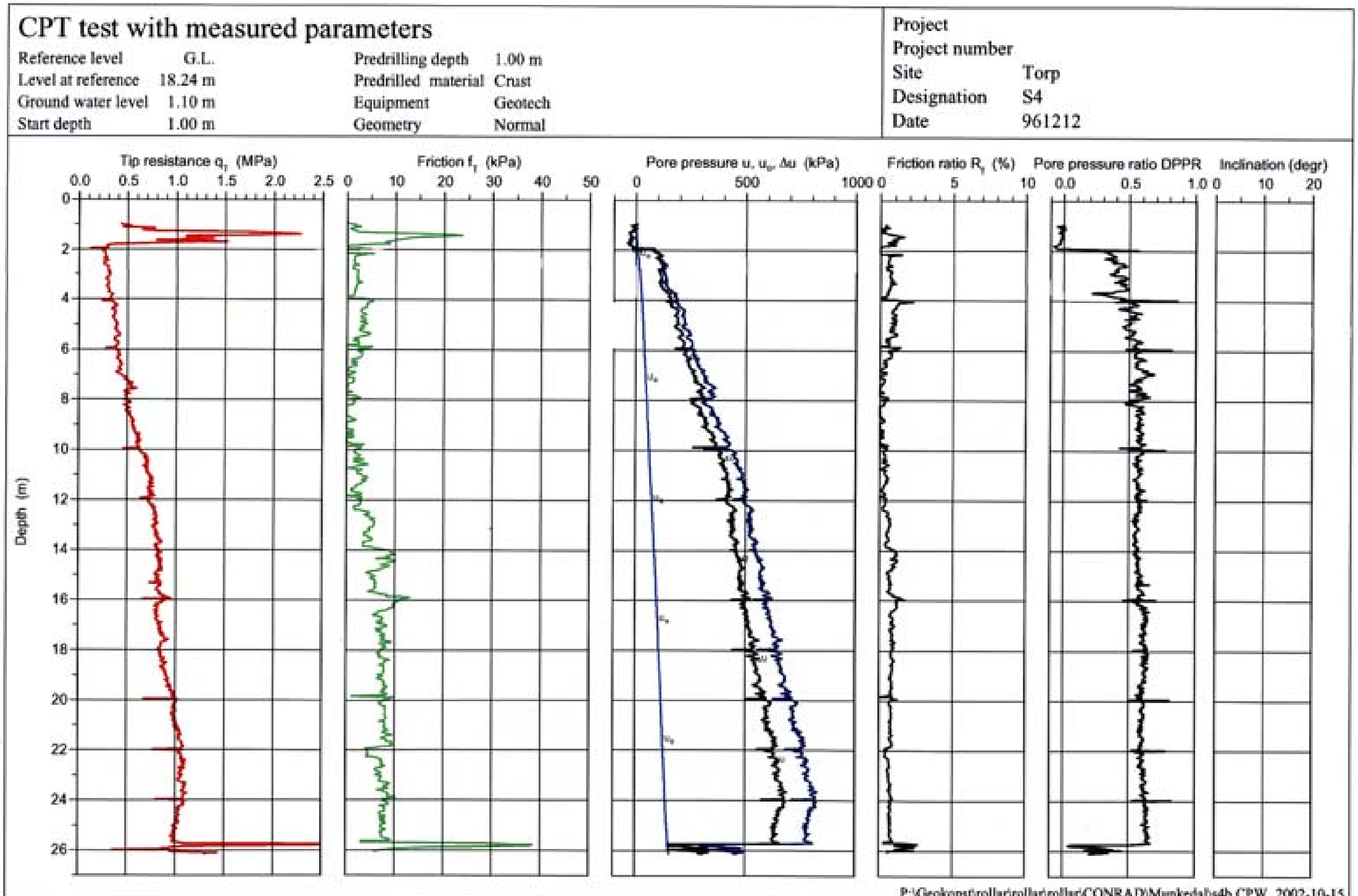


Fig. 12. Results of the CPT tests in Section A presented by use of the program CONRAD. e) Point S4

sampling were performed at the given distance from the crest, whereas the systems for pore pressure measurements were placed directly at the crest. At this place, they could be out of the way for all traffic on the terrace during the observation period. Point S10 is located at the back of the upper terrace, just outside the toe of the slope up to the natural ground level. Point S11 is then located about 5 metres behind the upper crest. The pore pressure systems were placed directly at the crest in this point too in order to be away from traffic to the warehouse. Point S12, finally, is located about 35 metres behind the upper crest and here the pore pressure measurement systems were moved a few metres extra inwards to be out of the way. The points in the new Section C are in principle located a few metres beside the section and points for the previous investigation. They are not exactly in line, however, but have been moved a few metres further at some locations to avoid making holes in asphalted surfaces or felling growing trees, Figs. 4 and 13.

There is no investigation point down at the riverbank in Section C. The reasons for this are that there are no reference values from any previous investigations in this location, except from a total pressure sounding, and that it would have required extensive work and felling of trees to reach this point.

CPT tests have been performed at all investigation points. They all reached large depths without approaching the maximum tip resistance and firm bottom. Instead they had to be terminated when the maximum penetration force was reached. This force was limited by the capacity of the drill rig and the penetration rods or, at the test from a raft in the river, by the available counterweight. No friction reducing equipment was used since the need for this had not been anticipated. At one point, the penetration had to be terminated because the available length of drilling rods of 60 metres had been used up. Most of the stop depths also approximately coincided with the limit for the acoustic signal transfer system, since no amplifier was used.

The CPT tests could have been driven deeper if special precautions had been taken for reduction of rod friction and signal amplification and if more rods had been brought to the site. However, the plan for the investigations and the selection of equipment was based on the results of the previous investigations and nothing in those indicated depths of the size that was found. No dynamic probing test was performed to investigate the depth to firm bottom at the time of the investigation since the available drill rods would not suffice. These findings contributed to the initiation of a new project in which the possible gain in including geophysical methods in the geotechnical investigations in similar geological conditions was to be illustrated (Dahlin et al. 2001).

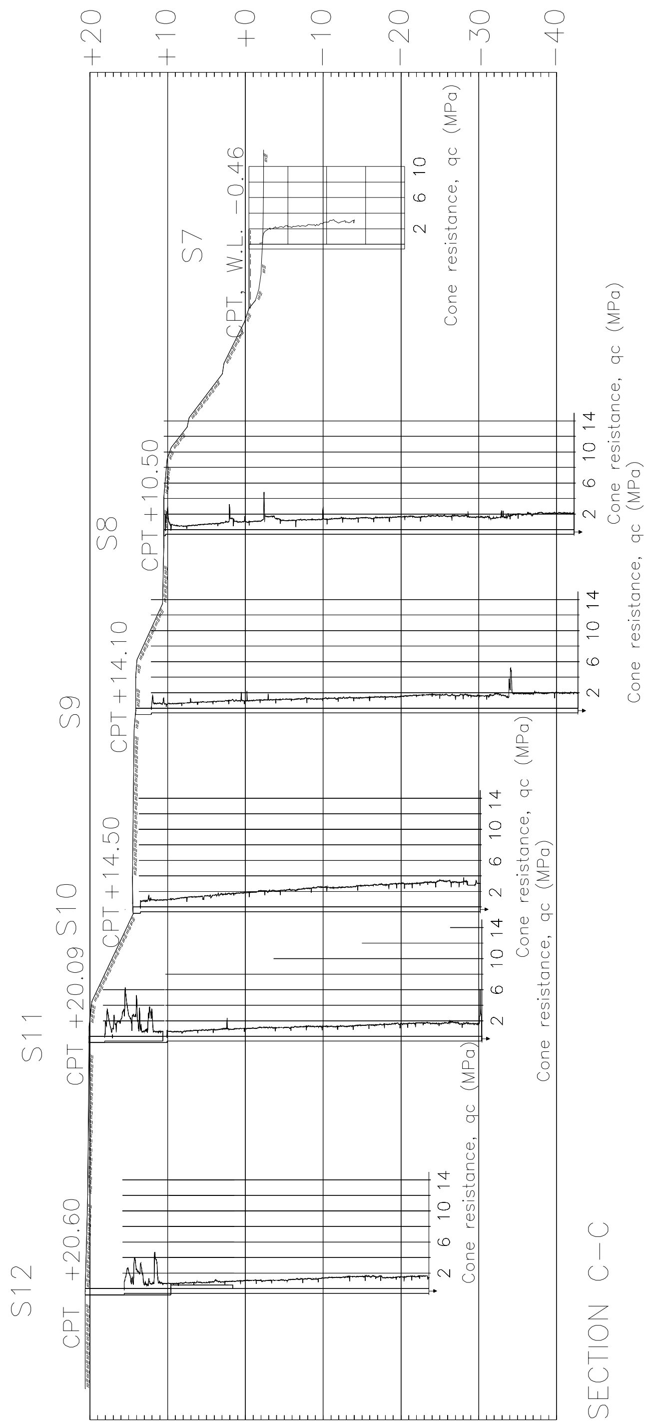


Fig. 13. Section C.

The CPT tests thus reached considerably deeper than the previous penetration tests at all points. At point S12, the CPT test reached a depth of 44 metres below the ground surface, which is 10 metres deeper than before. At Points S10 and S11, the tests reached a depth below the original ground level of 55 metres, which is about 25 metres deeper than before. At Points S8 and S9, finally, the tests reached about 63 metres below the original ground level, which is here about 35 metres deeper than before. The test from the raft in the river only reached 14 metres below the river bottom because of lack of counterweight for the penetration force. The differences in penetration depths in Section C were very serious for the estimation of the stability situation since the assumed firm bottom and an elevation of its level close to the river was shown to be non-existent. The previously calculated critical slip surfaces in Sections C and B were both limited by this assumed "firm bottom".

The results of the CPT tests showed that the upper sand and silt layers are about 10 metres thick at the back of the investigated area. This has decreased to about 8 metres at the new upper crest and it is not found in the other investigation points. The results of investigations before the excavation indicate that the upper layer had a thickness of about 6 metres within the excavated area, and the whole layer had thus been taken away in this area. The fine-grained soil below this upper layer changes in character with depth from clayey silt to high-plastic clay and then becomes siltier again at great depth. However, this gradual change cannot be readily observed in the test results, Fig. 14.

The CPT tests in the clay were performed using a very sensitive probe adapted for tests in clay. The tests through the upper sand and silt layers in the area behind the upper crest were performed with an ordinary 5-tonne cone in order not to damage the sensitive probe. When the clay layer was reached, the more robust probe was withdrawn. The hole was then expanded by use of a mandrel whereupon the test was resumed with the more sensitive probe. In the paved areas in Section C, the pavements and bases with gravel and cobbles were pre-drilled and the holes were cased down to the natural soil. Guiding tubes were used in the tests from the raft in the river to prevent the drill rods from buckling before they got enough lateral support from surrounding soil.

No stops against firm bottom were received in the penetration tests. However, the results indicated a coarser soil layer between the levels –31 and –34 metres, which is about 50 to 55 metres below the original ground surface. The penetration tests were stopped temporarily in this layer at all points where it was reached and the dissipation of excess pore pressure with time was studied. The results indicated that the layer had a relatively high permeability and this was later confirmed by the pore

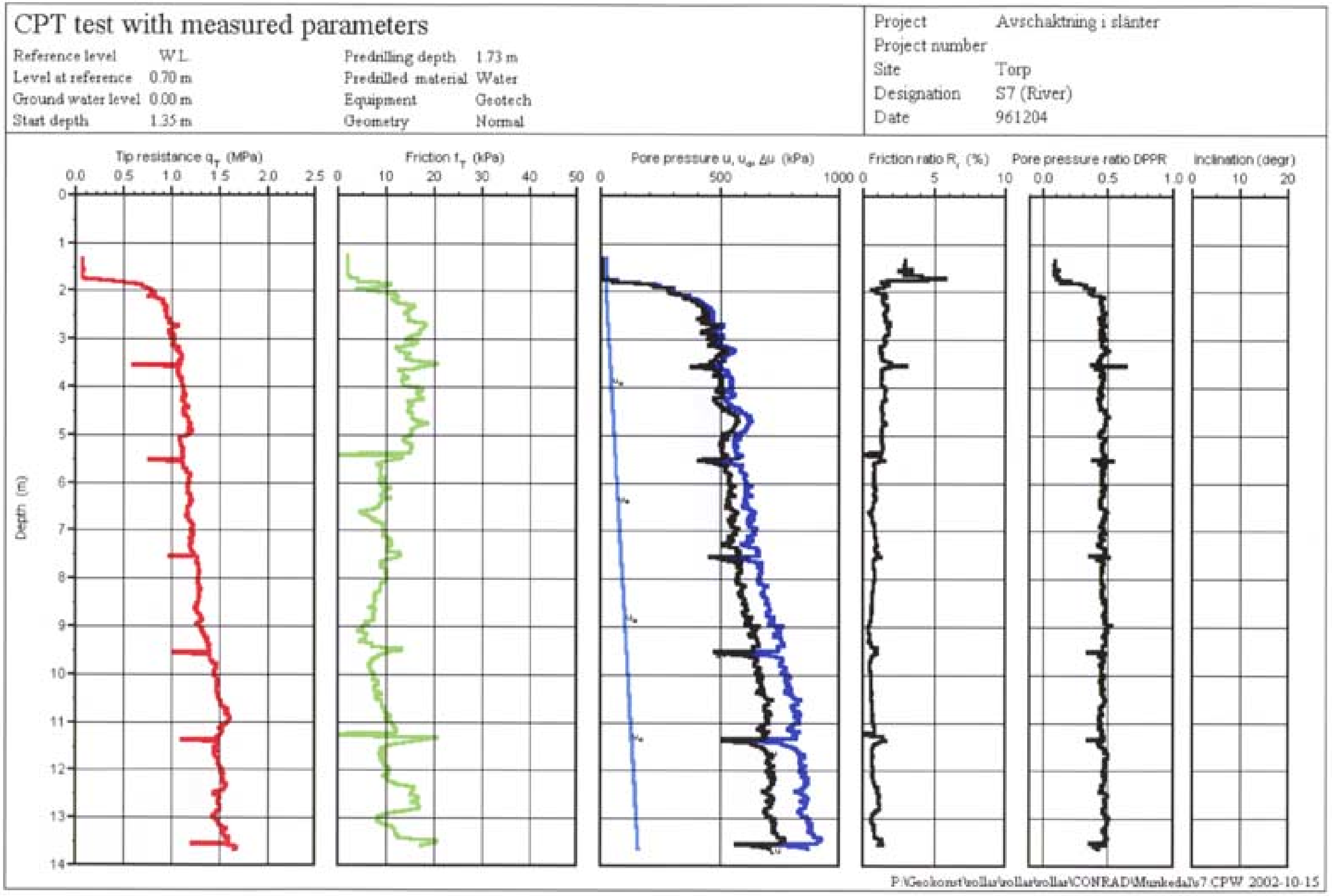


Fig. 14. Results from the CPT tests in Section C presented using the program CONRAD. a) Point S7

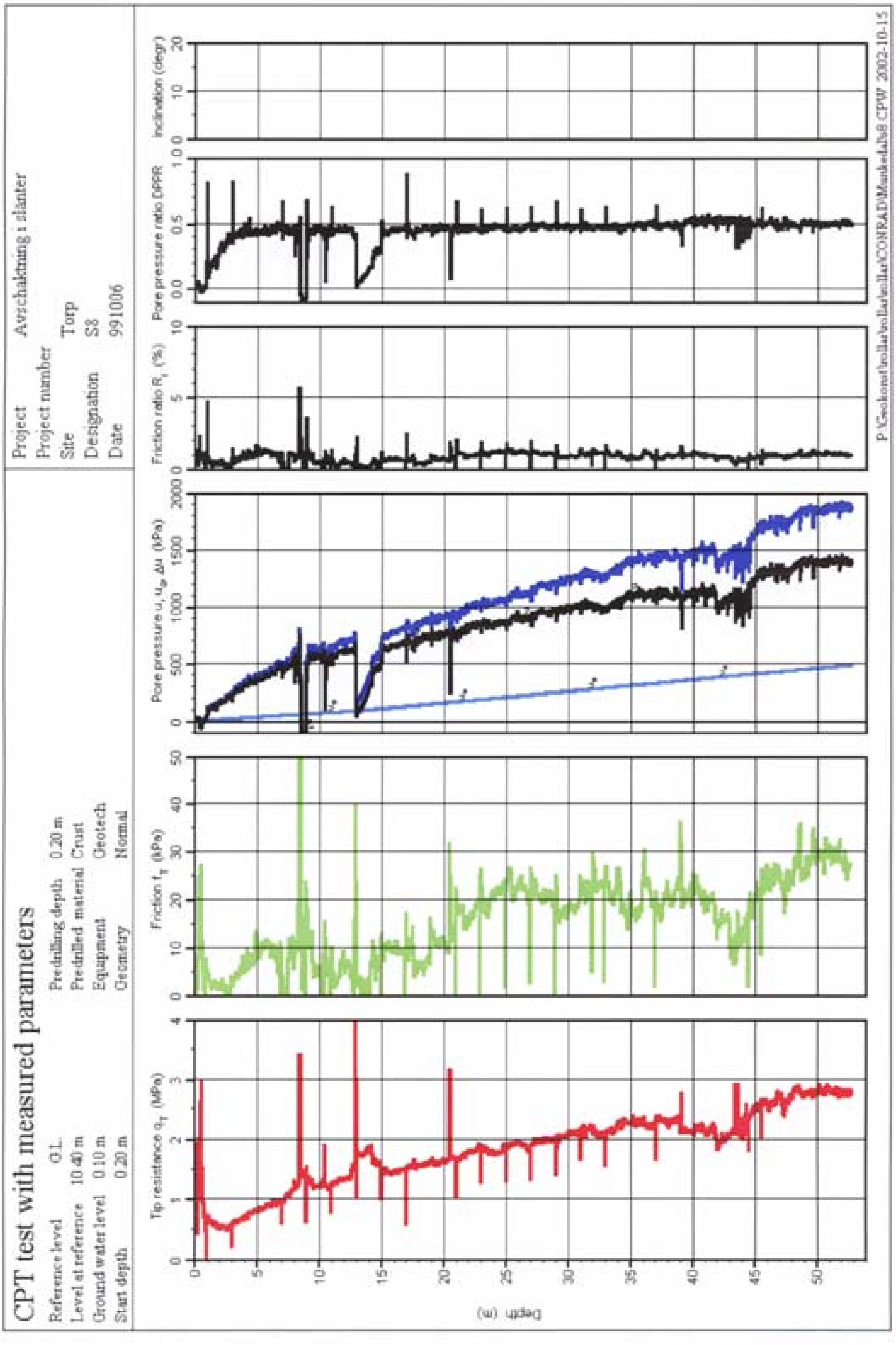


Fig. 14. **Results from the CPT tests in Section C presented using the program CONRAD. b) Point S8**

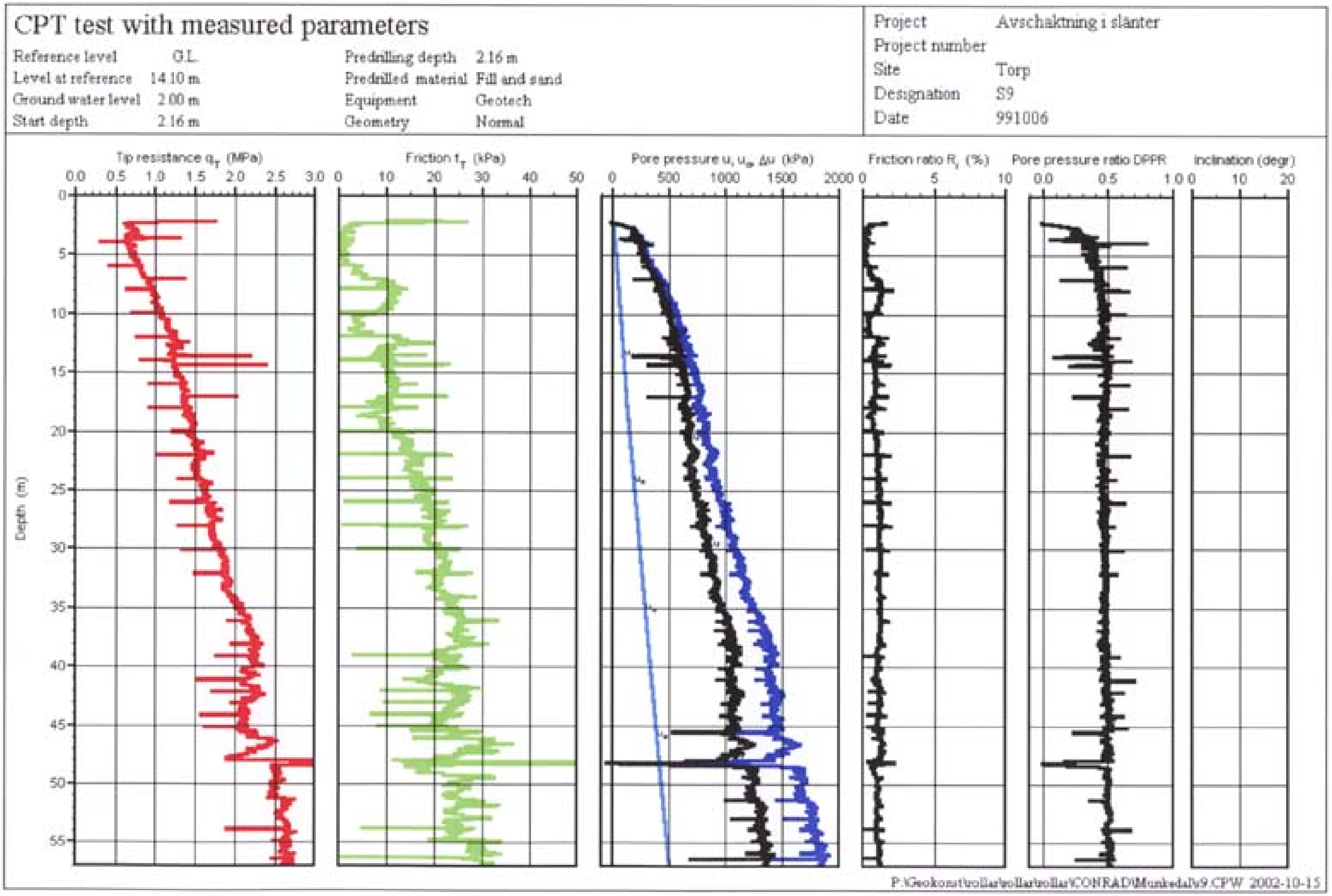


Fig. 14. **Results from the CPT tests in Section C presented using the program CONRAD. c) Point S9**

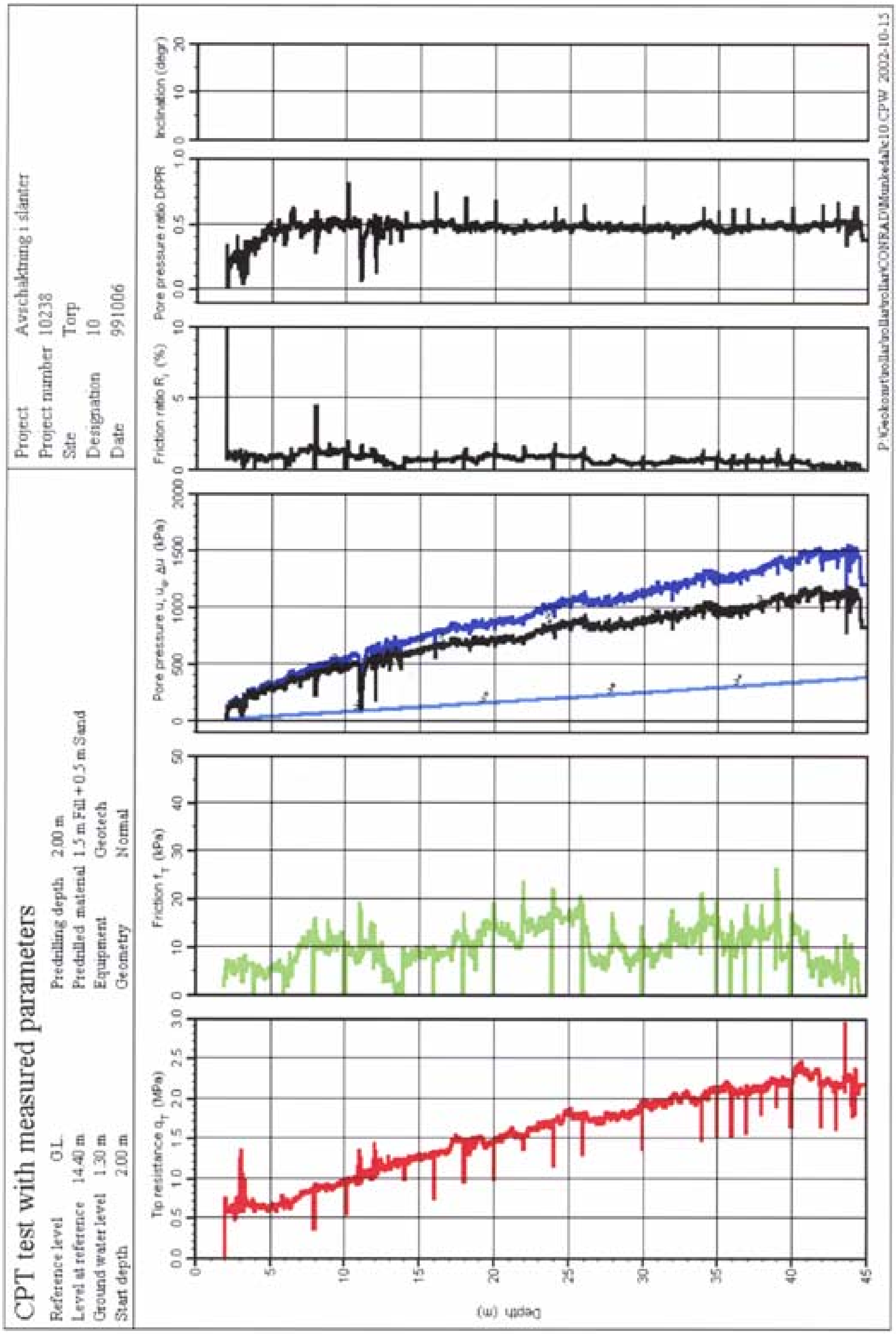


Fig. 14. **Results from the CPT tests in Section C presented using the program CONRAD. d) Point S10**

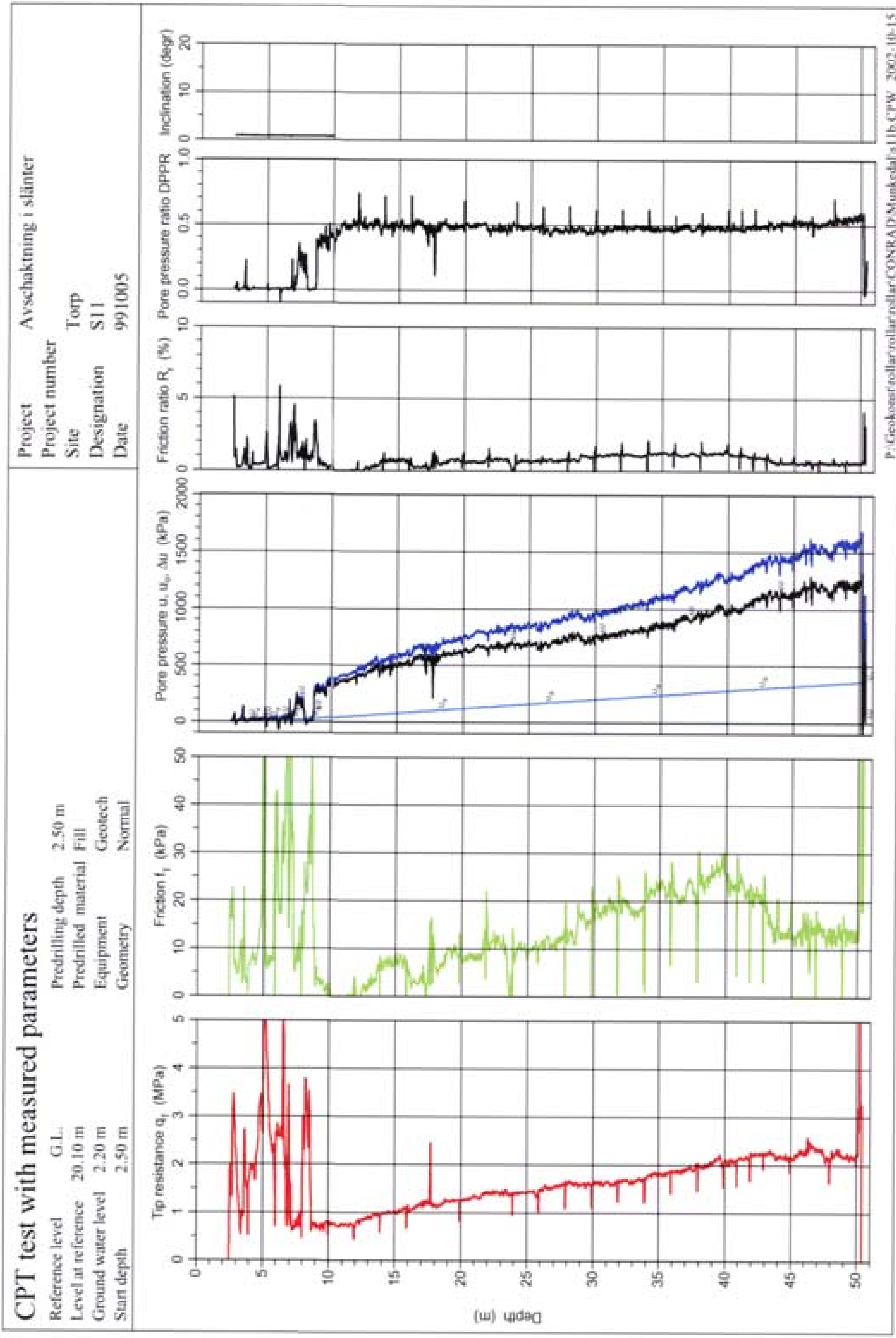


Fig. 14. Results from the CPT tests in Section C presented using the program CONRAD. d) Point S10

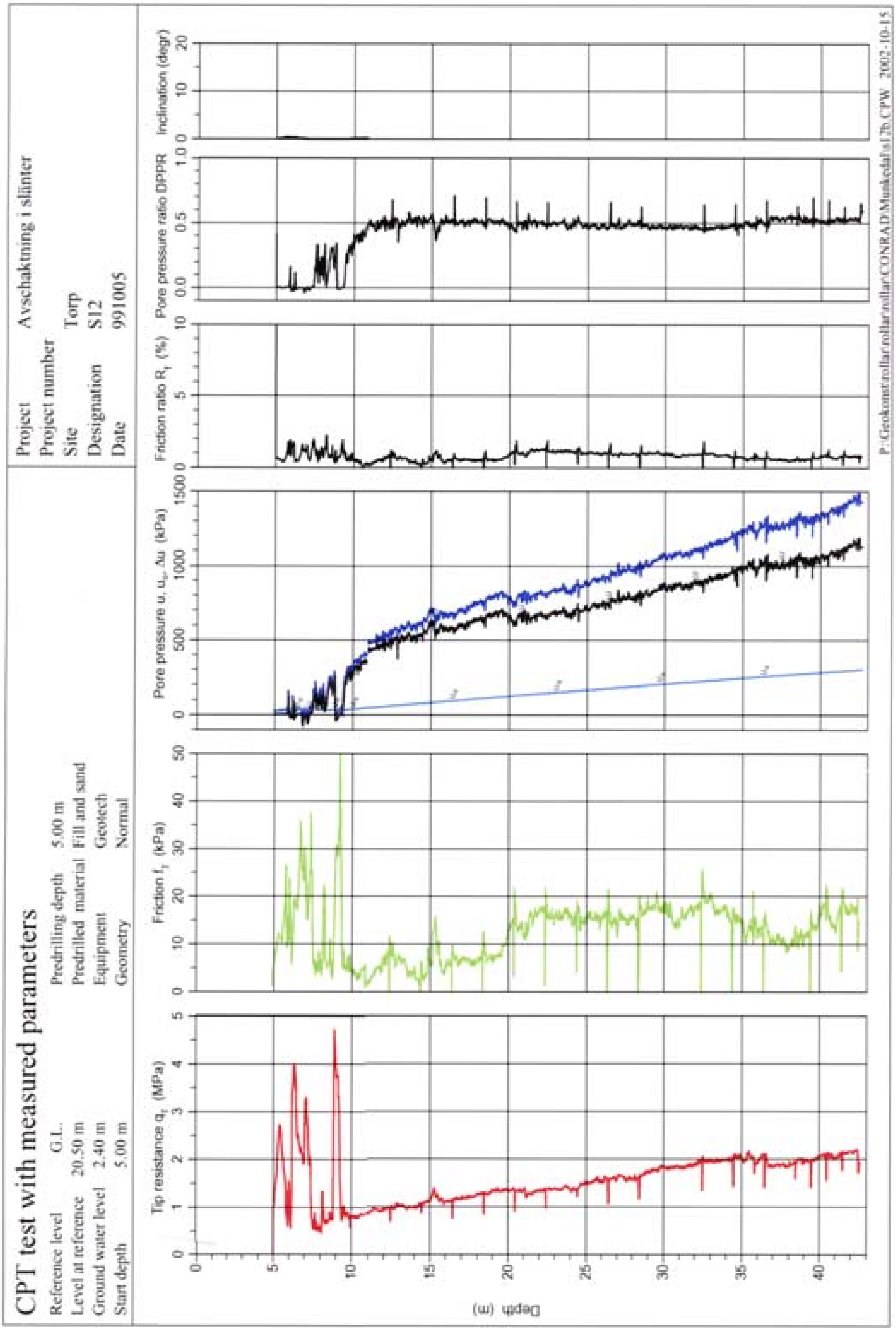


Fig. 14. Results from the CPT tests in Section C presented using the program CONRAD. f) Point S12

pressure observations. This layer thus greatly affects the pore pressure distribution in the section. Below this layer, the clay layers continue.

Dilatometer tests

A dilatometer test was performed as a supplement to the field vane tests and the CPT tests near Point S11 in Section C. This test was primarily intended to check whether this alternative method would give a different trend in the variation of the undrained shear strength with depth, particularly at the transition towards coarser soil in the bottom layers. At the same time, it provided an alternative classification and estimation of the overconsolidation ratio in the field and a measure of the coefficient of earth pressure in the soil. The latter value can, for example, be used in estimation of relevant stresses when reconsolidating soil specimens to in situ stresses in the laboratory.

The results of the dilatometer tests showed a 9–10 metre thick layer of sand and silt followed by normally consolidated clay to large depths. The test reached about the same depth as the CPT tests, which in this location was down to the layer with coarser soil at about 50 metres' depth. The available force from the drill rig was then not sufficient to push it further, Fig. 15.

Geophysical investigations

The geophysical investigations in the Torp area within the new project "Geophysical investigations in slope stability investigations" were performed in 2001 in co-operation with the Swedish Rescue Services Agency, Lund University and Impakt Geofysik (Dahlin et al. 2001). The investigations were performed using the seismic refraction method to determine the level and configuration of the bedrock surface and resistivity measurements to obtain a continuous picture of the variation of the soil layers within the sections.

The measurements were made along two lines, one running in east-west direction and approximately coinciding with Section A and one in a north-south direction along the road between the Kviström bridge and Åtorp manor house, Fig. 16. The location of the first measuring line was selected in order to obtain a direct comparison with the geotechnical investigations in Section A, where the depths to firm bottom in terms of stops against further penetration in a coarse bottom layer were known. The location of the second line was selected to obtain a measure of the general variation of the depth to bedrock along the valley.

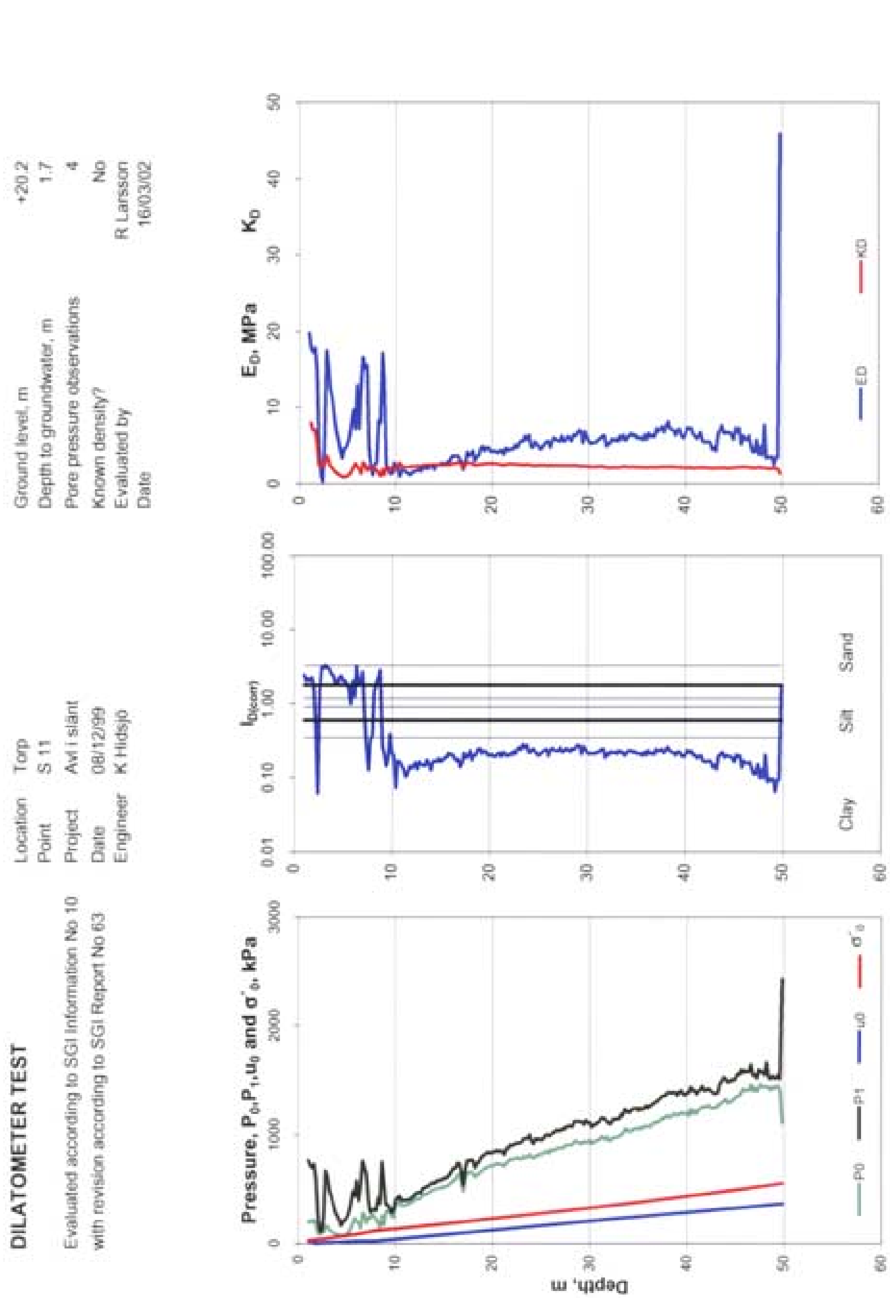


Fig. 15. **Results of the dilatometer test near Point S11, Section C.**
a) base data and parameters.

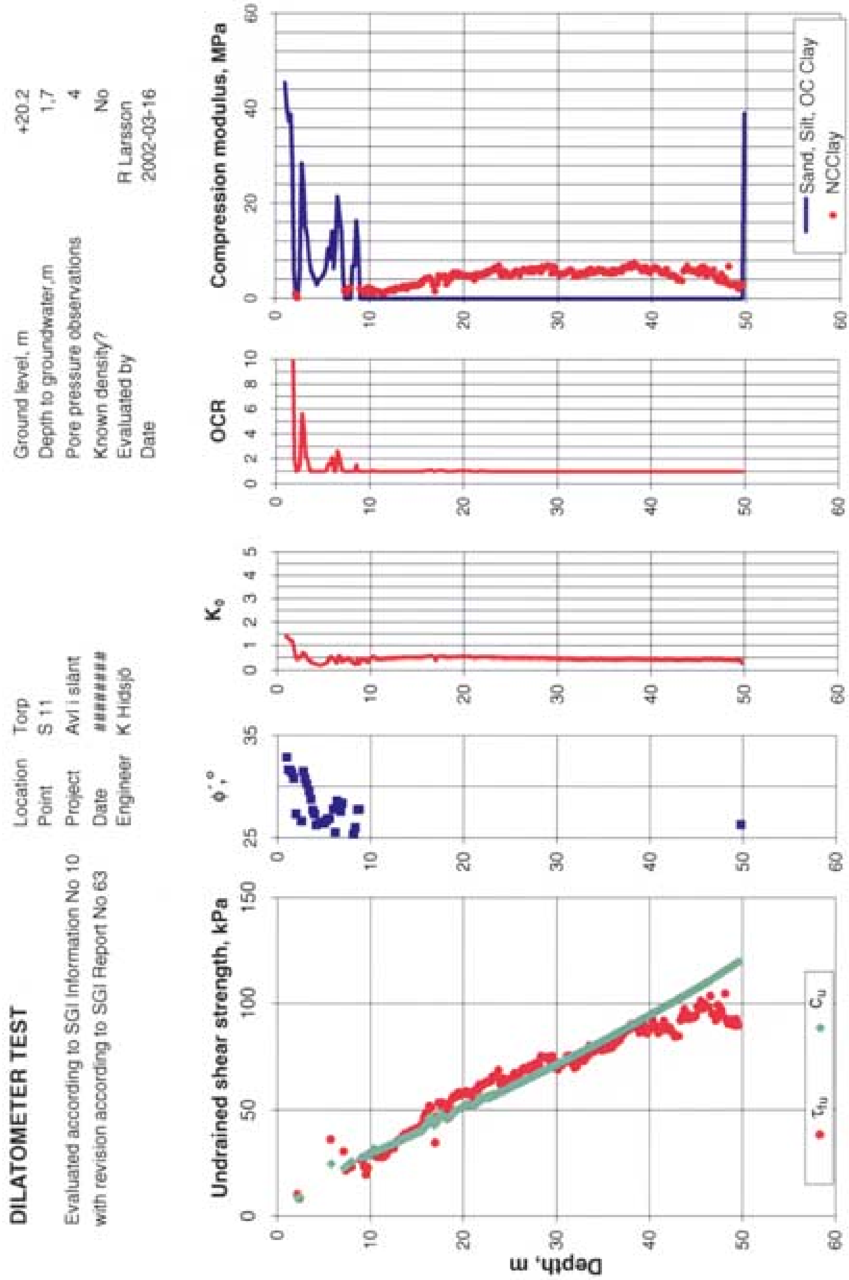


Fig. 15. **Results of the dilatometer test near Point S11, Section C.**
b) evaluated soil properties
(τ_{fu} = undrained shear strength evaluated according to SGI Information No. 10, c_u = undrained shear strength evaluated according to the alternative method, see section "Test results – Shear strength)

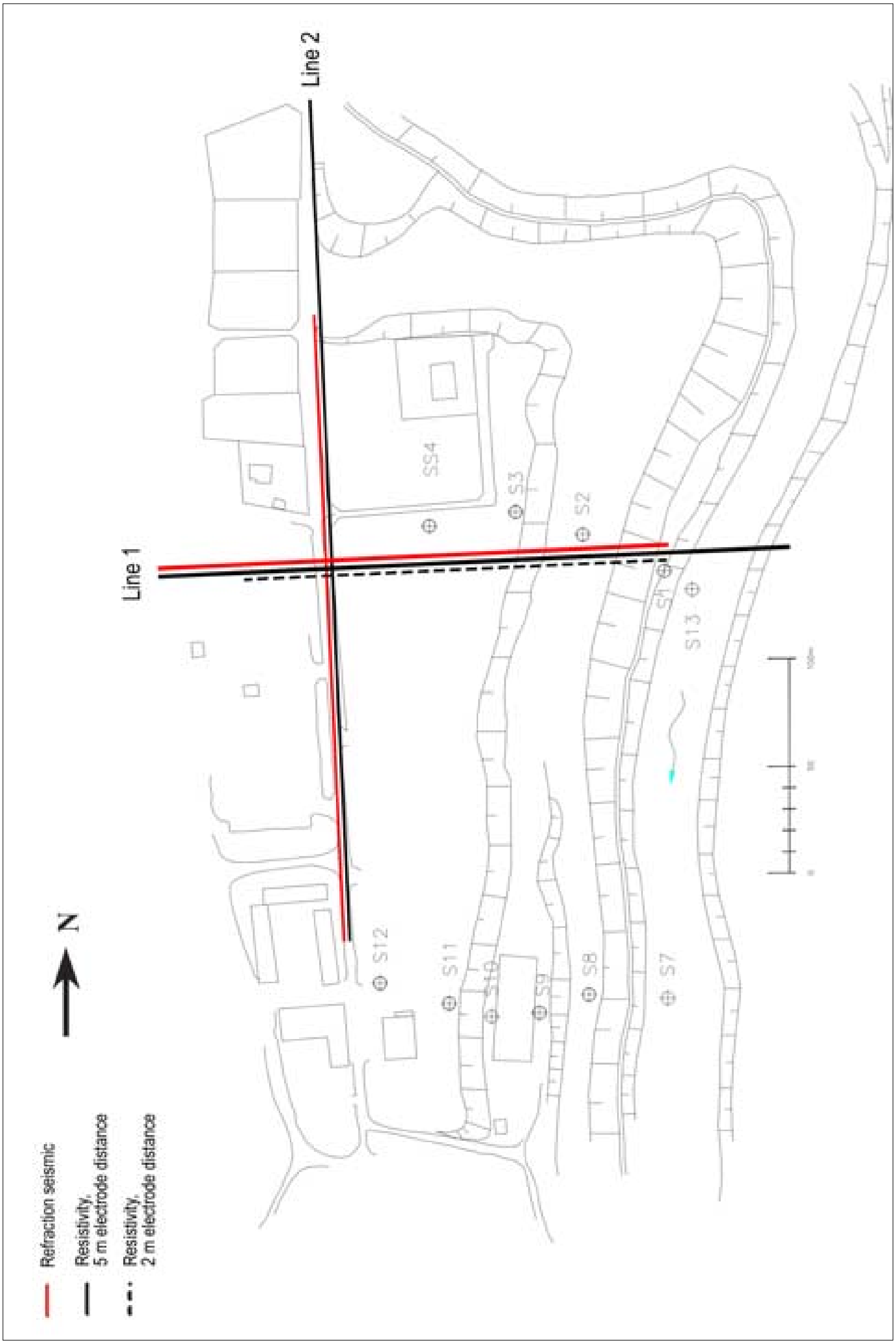


Fig. 16. Measuring lines in the geophysical investigations.

The results obtained by the refraction method showed two distinct refractors, the first being the border between what may be assumed to be not fully saturated and saturated soil in the upper sand and silt layers, and the second being the border between soil and rock. The second refractor shows that the contour of the bedrock in Section A has a similar variation as the assumed firm bottom and that the thickness of the coarse soil layer between the clay and bedrock has a thickness that varies between one or two metres and more than ten metres, Fig. 17a. The results of the measuring line in the north-south direction show that the level of the bedrock varies somewhat locally but in general slopes downward from north to south. The level of the bedrock in the measuring line is thus more than 20 metres deeper in Section C than in Section A, Fig. 17b.

The resistivity measurements showed that variations in resistivity that can be related to variations in salt content often have a larger influence on the results than the composition of the soil in other respects. The results provide a good picture of the variation of the upper sand and silt layers in Section A. The resistivity in these layers is high, Fig. 18. Similar high values are also measured in the superficial layers in the outer slope where the soil is coarser and probably not fully saturated. The variation in resistivity in the underlying clay is assumed to be related mainly to variations in salt content. The clay below the excavated area mainly has a very low resistivity and thereby a high salt content, whereas the clay behind the upper crest has a higher resistivity and can thus be assumed to be more leached. A coupling to the sensitivity of the clay can be observed in such a manner that quick clay may be found in parts of the clay mass where the resistivity is higher than 7 Ωm but does not exist in parts with a lower resistivity.

Field vane tests

Field vane tests were performed to determine the undrained shear strength and its variation in the section and to measure whether this had changed since the time before the excavation. Tests were thus performed in locations in natural ground behind the excavation, in the excavated areas and below the river bottom in each section. Since there were two separate excavated terraces in Section C, tests were performed in both of these terraces. Field vane tests were thus performed at Points S1, S2 and S3 in Section A and Points S7, S8, S9 and S11 in Section C.

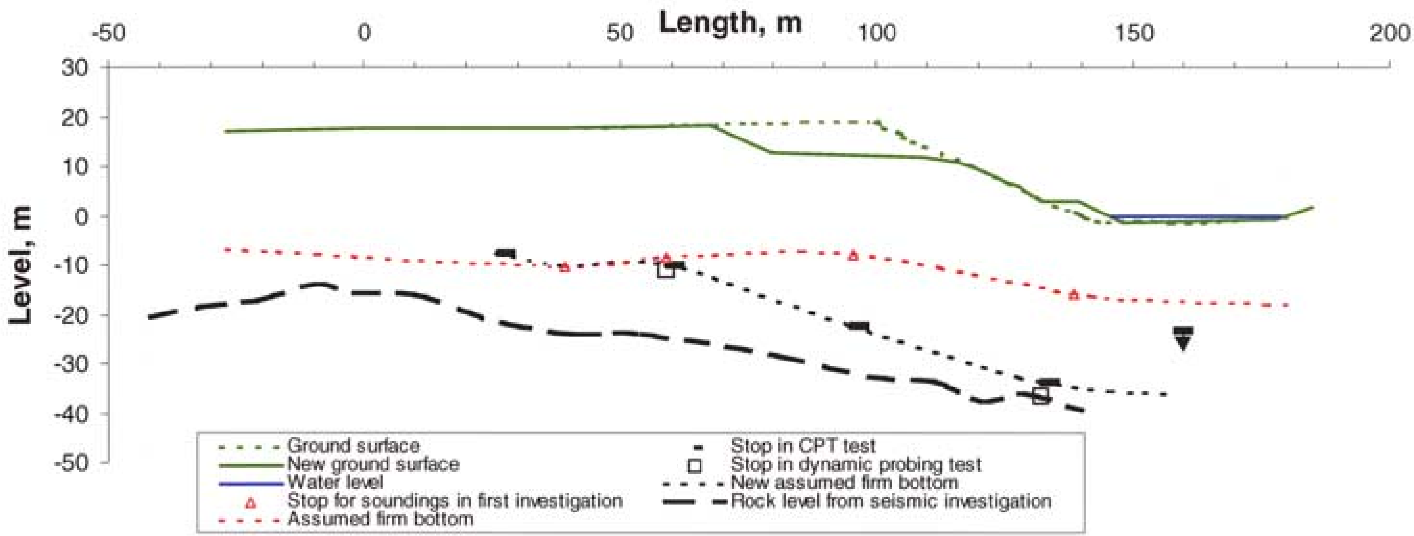


Fig. 17. **Results of different determinations of firm bottom and bedrock.**
a) Estimated levels for firm bottom and bedrock in Section A.
(Arrows below the penetration depths indicate that no firm bottom has been reached)

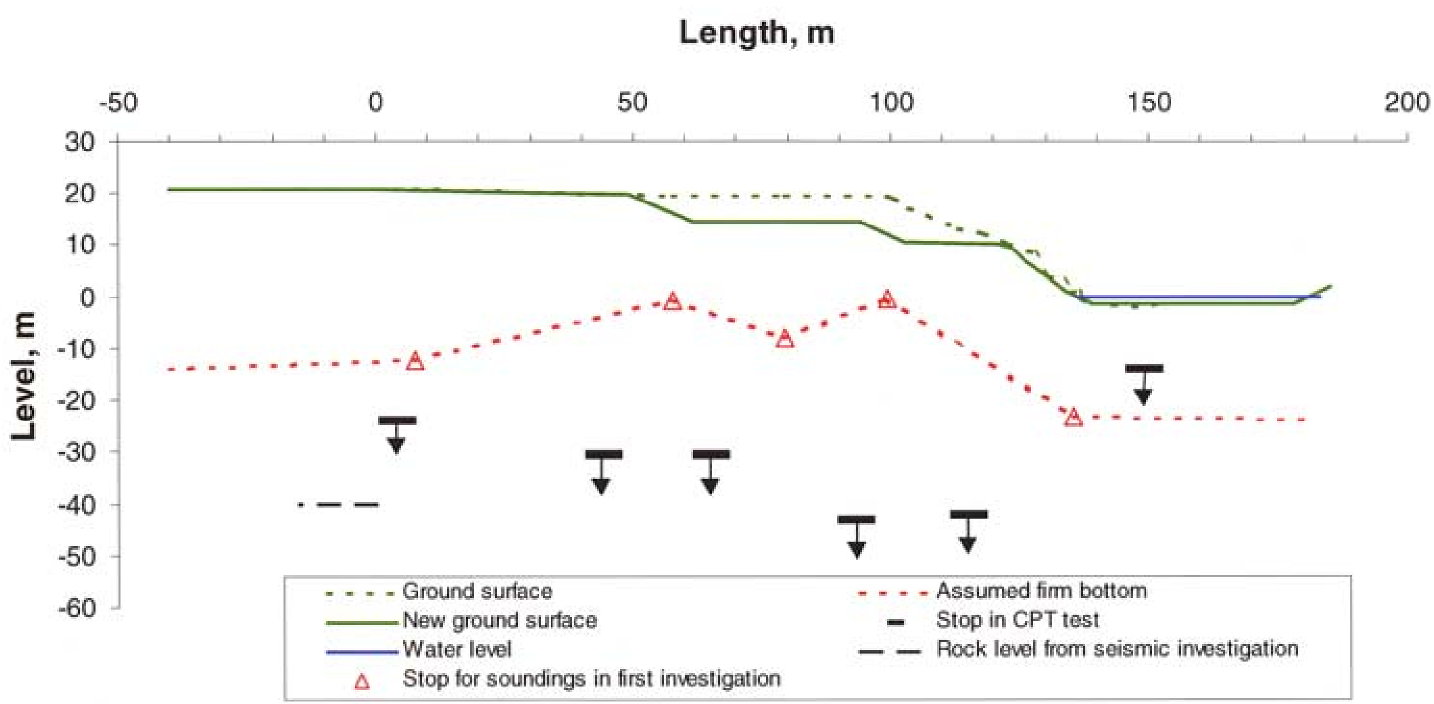


Fig. 17. **Results of different determinations of firm bottom and bedrock.**
b) Estimated levels for firm bottom and bedrock in Section C.
(The level of the bedrock has only been assessed in the point where the seismic measuring line intersects the Section)

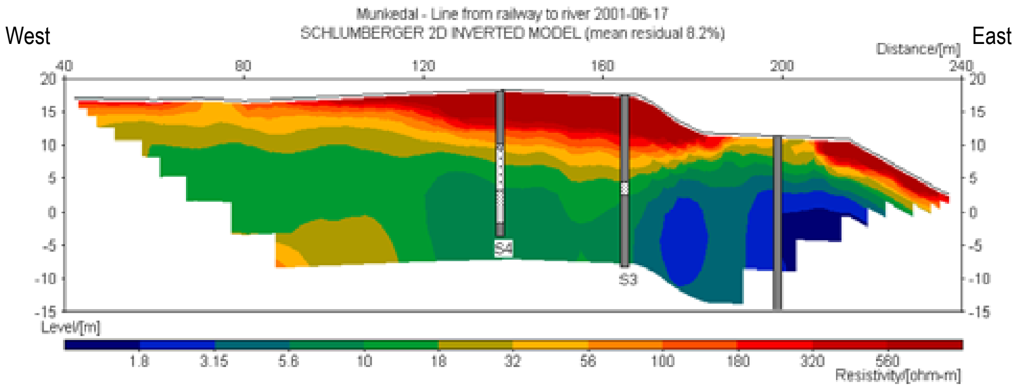


Fig. 18. **Results of resistivity measurements in Section A. The investigation points S4, S3, and S2, in which sampling has been performed, have been introduced together with a scale for the measured sensitivity. A dark screen pattern corresponds to S_t < 50, a medium light pattern S_t = 50 – 100 and a light pattern with single dots S_t > 100.**

In previous investigations before the stabilisation works, field vane tests had been made at Points 1, 2 and 3 in Section A, Point 9 in Section C and at Point 22. New tests specially intended for comparison were therefore made at all these points except Point 1. The latter point is located on the riverbank and no significant change in overburden pressure had occurred in this area since the time of the previous investigation.

A closer study of the results of the previous investigations and those that were obtained in the first round of investigations in Section A showed clear indications of an influence on the shear strength measured from the size of the vanes used and possibly also from the type of equipment. A larger study was therefore performed in connection with the next round of investigations in Section C. In this study, results of both old and new types and makes of equipment and small and normal vane sizes were compared (Åhnberg et al. 2001). The study showed that the type of equipment did not have any significant effect but tests using small vane sizes gave only about 87% of the shear strength measured with the normal vane size in this type of clay. This largely explained the anomalies that had been obtained in the previous test results.

The field vane tests in the previous investigations had only been performed to depths of 20 to 25 metres below the ground surface. The new tests were also stopped at about this depth, partly because the maximum torque for the normal size vane was reached. The variation in shear strength at larger depths was estimated from CPT test results, dilatometer test results, empirical relations and laboratory test results.

Pore pressure measurements

The pore pressure has been measured in a number of measuring stations in Sections A and C. Previous pore pressure measurements had only been made at two levels in a location near Point 2 in Section A. Four new stations were installed in Section A; one behind the erosion protection at the riverbank, one at the centre of the excavated area, one directly behind the upper crest and one about 50 metres behind this crest, i.e. at Points S1, S2, S3 and S4. The number of measuring levels in the different points were 3, 5, 6 and 3 respectively and they were distributed with depth in such a way that complete pore pressure profiles could be obtained.

The pore pressure measurement systems in this section consisted of open tubes with inner hoses of 8 mm inner diameter. The stabilisation of the water levels in these hoses was very slow and the measurements showed that it took several months until those systems that had their filter tips placed in the central parts of the clay layer reached stable conditions. The subsequently measured seasonal variation in pore pressure was very small. The function of these systems was checked after about a year by filling up the hoses with water and measuring the equalisation processes. Again, these processes proved to be very slow and afterwards the water levels in the hoses were stabilised at approximately the same levels as before. Any significant variations were only measured in tips located in superficial layers and in tips in or near the coarser soil layers at the bottom. From these results, it was concluded that this type of system probably had too slow a response to fully register the seasonal variations.

One system was replaced in connection with the function control and a new tip was also installed near the crest of the lower slope from the excavated area to the riverbank. This point was designated S2b. One of the pipes and its hose at Point S1 on the riverbank has later been lengthened with raised tops because the artesian water pressures in the bottom layers caused overflow in the hose.

As a consequence of the results in Section A, only closed pore pressure measurement systems were installed in Section C. Systems of the BAT type were used. They were installed in five stations at Points S8, S9, S10, S11 and S12, i.e. at the centre of the lower excavated terrace, at the crest at the front of the upper excavated terrace, at the back of this terrace, directly behind the upper crest and about 35 metres behind this crest. In the various stations, filter tips were placed at 4, 4, 3, 4 and 3 levels respectively. Their distribution with depth was selected on the basis of the results from the CPT tests in order to clarify the pore pressure distribution in the whole profile. In three stations, filter tips were installed in the assumedly permeable layer located 50 to 55 metres below the original ground surface. The pore pressures have been read off regularly for more than a year in order to study the seasonal variations.

The pressures in the closed systems stabilised within a week and appear to react faster to seasonal variations. However, with time problems have developed with formation of gas in several of the filter tips. This makes the readings very time-consuming because the gas in the tips has to be evacuated and time given for the pressures to re-stabilise at each reading.

In connection with the second round of field investigations, two BAT tips were also installed adjacent to and at the same levels as two open systems at Point S2 in Section A. The purpose of this installation was to obtain a direct comparison between the measured variations with the two systems and a better estimate of the real pore pressure variations in this section.

2.6.4 Sampling

Sampling has been performed in four points distributed from the river channel to the area behind the excavation in each section. The sampling has been performed with a Swedish standard piston sampler type St II using the utmost care when taking and transporting the samples. Judging by the results from the fall-cone tests and the oedometer tests in the laboratory, the taking of undisturbed samples succeeded fairly well.

In the first round of investigations in Section A, sampling was performed only at Points S13, S2 and S4. However, a close study of the results of the field tests and the subsequent laboratory tests showed that the soil conditions vary considerably with distance from the river. Supplementary sampling was therefore performed at Point S3 in the second round of investigations.

In Section C, sampling has been performed at Points S7, S8, S9, and S11. With regard to the great depths, the sampling has in most points been limited to the upper soil layers of primary interest, but at Point S9 it has been continued down to 57 metres depth, i.e. 63 metres below the original ground surface. The purpose of this deep sampling was to determine the character of the clay at large depths and to study the composition of the soil in the coarser layers that were indicated at about 50 metres depth by the CPT tests.

2.6.5 Laboratory tests

Routine tests

The samples have been investigated according to Swedish practice by routine tests of density, water content, fall-cone liquid limit and undrained shear strength and sensitivity according to fall-cone tests. Special attention has also been paid to ocular inspection and classification of the upper sand and silt layers and the embedded layers found at great depth in Section C.

Oedometer tests

A large number of CRS oedometer tests have been performed in order to determine the preconsolidation pressure and its variation within the sections. The permeability of the soil and its variation has also been determined by these tests.

Triaxial tests

Undrained active and passive triaxial tests have been performed on selected samples. The tests have been performed in order to enable the shear strength anisotropy to be taken into account in the stability calculations and to verify the applicability of the empirical estimate of the anisotropy that is normally made. This verification is required if the effects of the anisotropy are to be taken into account according to the guidelines of the Commission on Slope Stability (1995). A number of drained triaxial tests have also been performed in order to determine the drained shear strength parameters and in a corresponding way verify the empirically estimated parameters. All triaxial tests have been performed on specimens which have been consolidated for stresses just below the estimated preconsolidation stresses at the particular levels and have then been unloaded and adapted to the corresponding in situ stresses.

Direct simple shear tests

Undrained simple shear tests have been performed for two purposes. The first purpose was to check the shear strength determinations that had been made with field vane tests, CPT tests and dilatometer tests. The evaluation methods for all these tests use empirical factors in the calculations and the scatter in these factors can be considerable. It has also been found that the results of the different test methods differ particularly greatly in investigations of slopes and that the relations vary between different parts of the slopes. Research concerning the reason for this and how it should be taken into account is currently going on at Chalmers University of Technology (Löfroth 2002).

Direct shear tests were also performed to study the effect of an unloading on the shear strength by laboratory tests. One of the purposes of the project was to study the degree to which the undrained shear strength is affected by unloading. This is often estimated empirically by use of relations that are selected cautiously on purpose. These relations are also to a large extent based on experience from abroad and other types of clay. Series of undrained direct simple shear tests with different degrees of unloading were therefore performed to obtain relevant relations for the soils in the particular slope. Such series were performed on three selected types of clay which, based on the results of the routine tests, were considered to be representative of different layers in the profiles.

All specimens in the direct simple shear tests were first reconsolidated for a vertical stress just below the measured preconsolidation pressure. They were then unloaded to the vertical effective overburden pressure in situ or to the selected vertical pressures in the test series and left to adjust for these stresses. All specimens in a series came from the same level in the same point and were prepared from the middle and lower sampling tubes obtained from the standard piston sampling.

2.7 TEST RESULTS

2.7.1 Soil conditions – variations in plane and profile

The results of the CPT tests showed soil conditions which in most respects were in concordance with earlier descriptions but which in some respects were considerably different. They were also in some respects similar in the two sections but varied greatly between them in other respects.

As in previous observations, the thickness of the upper sand and silt layer was found to vary strongly within the area. From the results of this and previous investigations, it can be estimated that the thickness of this layer has been about 5 metres over the excavated area in Section A. It then decreases with distance from the river, from about 5 metres at the upper crest above the excavation to about 2 metres at a distance of 50 metres behind this. In Section C, the corresponding thickness of the sand and silt layer above the excavated area can be estimated to about 6 metres. However, in this section the thickness of the upper layer then instead increases with distance from the river. It is thus about 8 metres at the upper crest and fully 10 metres at a distance of 35 metres further away.

The thickness of the underlying clay layers was found to vary strongly and mainly be much greater than had previously been assumed. In Section A, it varies from being about 25 metres in the area behind the upper crest and then gradually increasing under the slope to having been about 48 metres down at the river. The thickness of the fine-grained soil layers has decreased because of the erosion and the excavation, but in spite of this there still remains about 35 metres of clay below the excavation and also below the river bottom. The clay is underlain by coarser soil with a thickness that varies from one or two metres to more than 10 metres, which rests on bedrock.

The clay located directly below the upper sand and silt layer contains thin layers and lenses of silt, and varying amounts of organic matter in the form of plant remnants and sand particles. This content decreases gradually with depth and the liquid limit and the water content in the clay increase to maximum values about 15 metres down in the clay layer. From this depth the trend becomes reversed and liquid limit and water content then decrease gradually with depth.

The full thickness of the clay layers has not been established in Section C. However, the soil profile below the upper sand and silt layer has been found to first consist of clay of approximately the same type as in Section A with an original thickness of about 40 metres. Then follows an about 5 metre thick zone in which the clay gradually becomes more silty, contains thin silt layers, contains gradually thicker sand layers and then in the reverse order returns to clay again. The clay is then first of the same grey sulphide patched type as above. About 7 metres further down it turns into a brown-grey varved silty clay of a type that normally indicates a transition to varved glacial clay in the bottom layers. The zone with coarser soil is slightly inclined with a downward slope towards the river. In spite of the erosion, there are at least 35 metres of "homogeneous" clay on top of it also below the river bottom.

A comparison between the CPT test results indicates that the clay in Section C is of the same character at the same levels in all test points. However, in Section A the results at Point S4 are significantly different from the rest of the results in the section. Below 10 metres depth, the cone resistance is clearly lower and the pore pressure ratio is clearly different and generally higher. This indicates a more normally consolidated, a more silty and/or a more sensitive type of clay. That the clay at this point should have a significantly different stress history than that at Point S3 closer to the crest is unlikely, and the results therefore primarily indicate that something in the composition of the clay and/or its other properties is different.

The results of the routine investigations in the laboratory show that the upper sand and silt layer has a density of about 1.9 t/m^3. It consists mainly of sand with an organic content. A few sample tubes with very even-grained material and incomplete saturation and/or large amounts of organic material showed lower values, particularly in Section A. The density in the silty clay directly below is about the same as in the water-saturated sand and silt layer. In Section C, the density then gradually decreases with depth as the silt content in the clay decreases and the water content increases. It reaches a minimum of about 1.65 t/m^3 at a level of –5 metres, which is 25 metres below the original ground surface. From this level it gradually increases again to become about 1.85 t/m^3 at a level of –45 metres. The density is significantly higher within the zone with infusions and layers of coarser soil where a maximum value of 2.16 t/m^3 has been measured, Fig. 19.

The density in Section A in general follows the same trend, but here the differences are considerably greater between the sampling points, Fig. 20. The results indicate that the inclined contour of the firm bottom also has created inclined clay layers. However, the variation in other parameters such as liquid limit and sensitivity indicates that the composition and properties of the clay also vary somewhat. The values that differ most from the general trends in the latter respect are found at Point S4, which is located 50 metres behind the upper crest.

The uniformity of the soil conditions in Section C is illustrated further when the natural water content and the liquid limit in the various boreholes are plotted versus the sampling level, Fig. 21. The different curves then overlap without any significant difference in values or trends except for a certain normal scatter.

A similar uniformity is not found in Section A. Instead the values vary between the sampling points, Fig. 22. Water contents and liquid limits of the same sizes as those on the same levels in Section C are measured at Point S2 below the excavation and at Point S13 below the river bottom. The values are lower at Point S3 just behind the upper crest and at Point S4 50 metres further away the liquid limit is much lower. The variation in water content generally follows the variation in liquid limit except for the central parts of the clay layer at Point S4, where the water content is considerably higher than the liquid limit. The measured variations in water contents correspond to the measured variations in density.

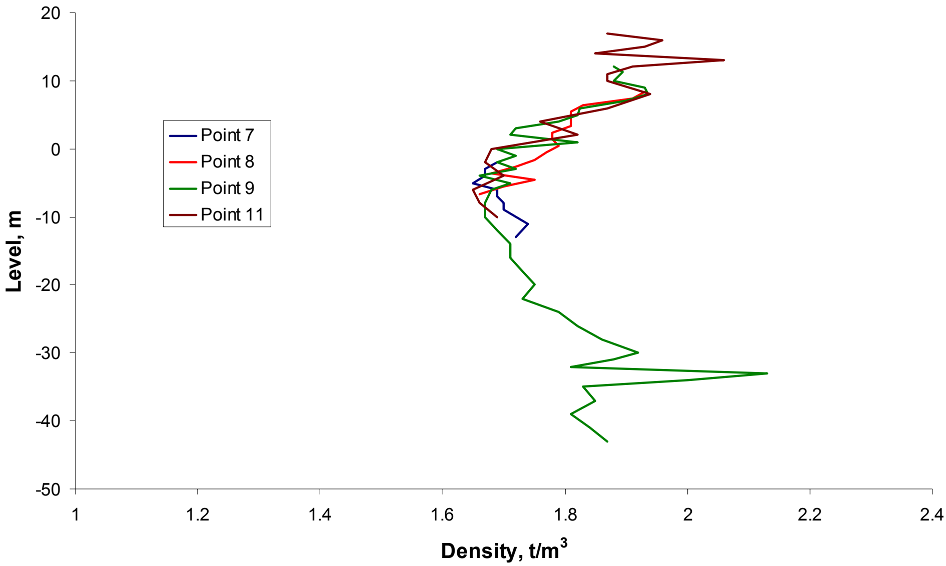


Fig. 19. Measured density in soil samples from Section C.

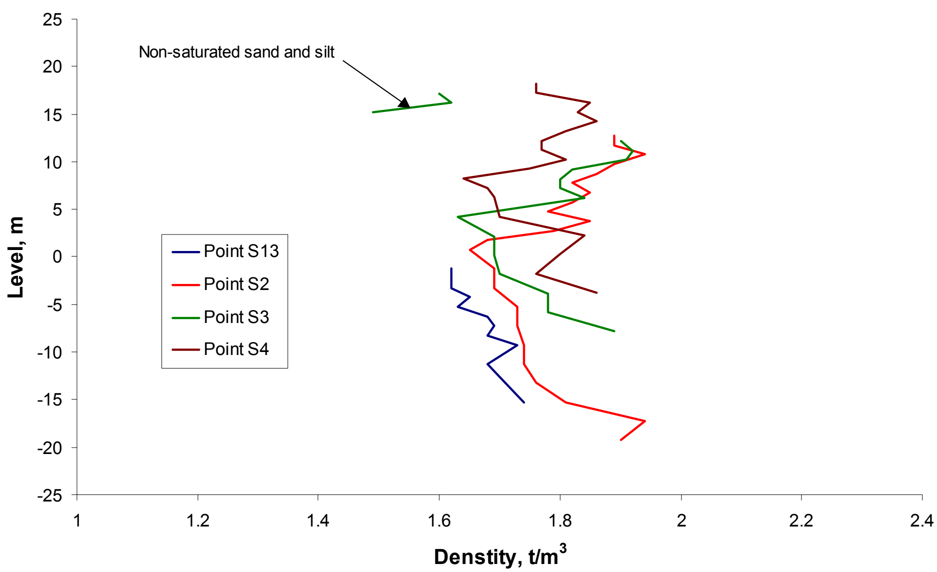
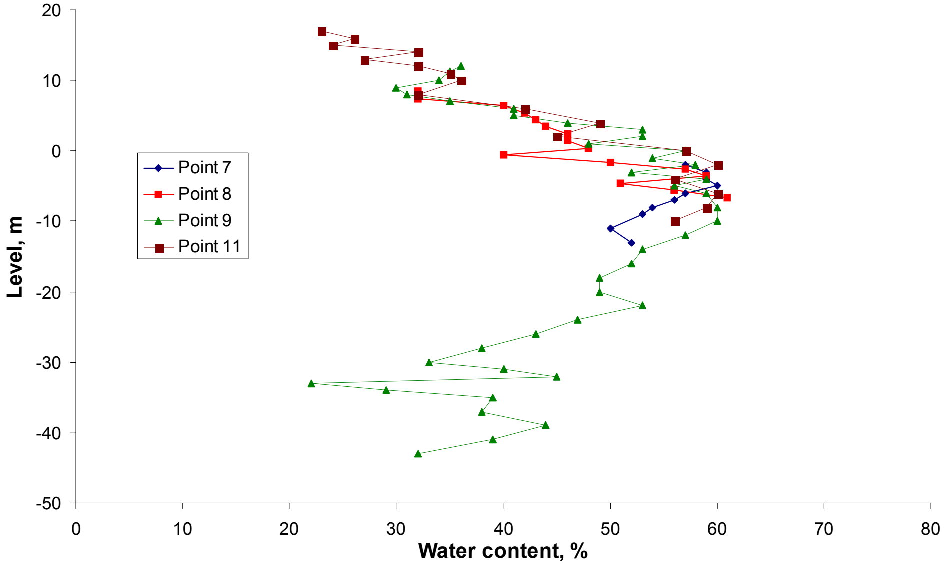
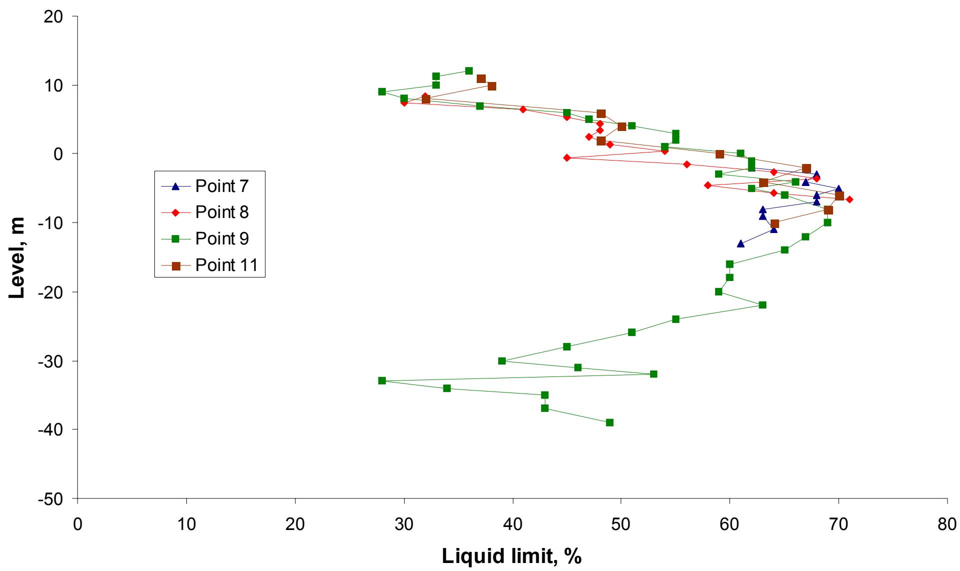


Fig. 20. Measured density in soil samples from Section A.

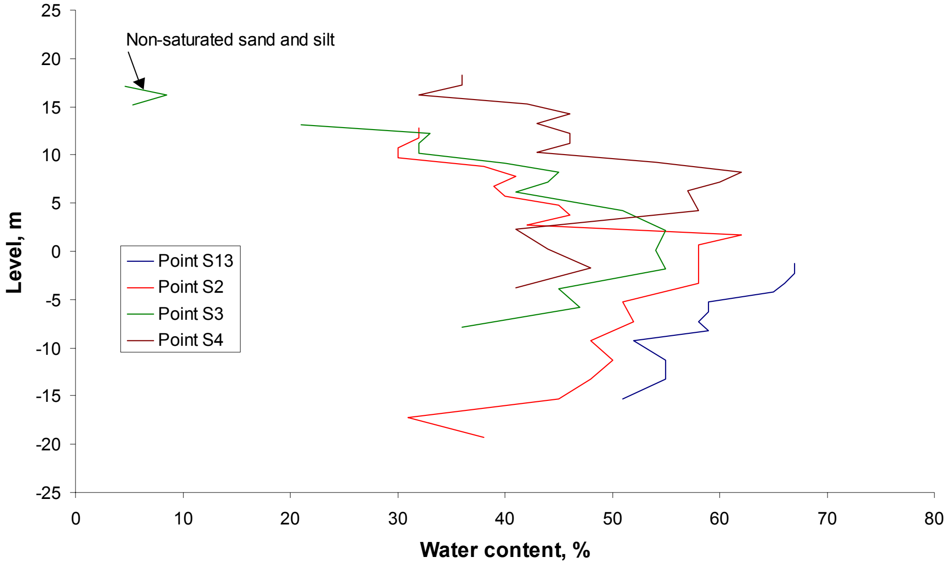


a)

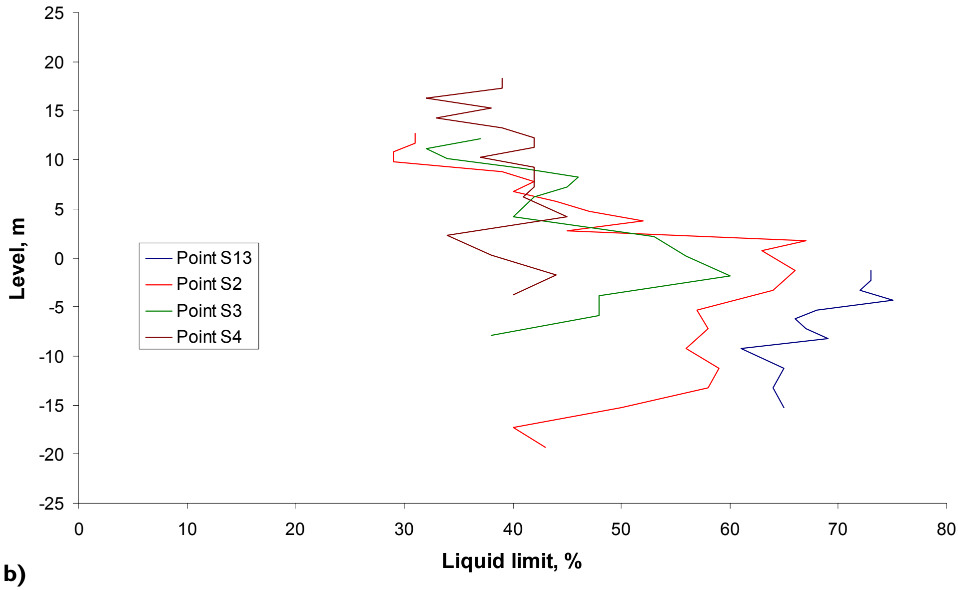


b)

Fig. 21. **Measured water contents and liquid limits in Section C.**
a) water content
b) liquid limit



a)



b)

Fig. 22. Measured water contents and liquid limits in Section A
a) Water content
b) Liquid limit

An alternative and more sensitive way of presenting variations in the general state of the soil is to study its liquidity index. The liquidity index, I_L, is defined as

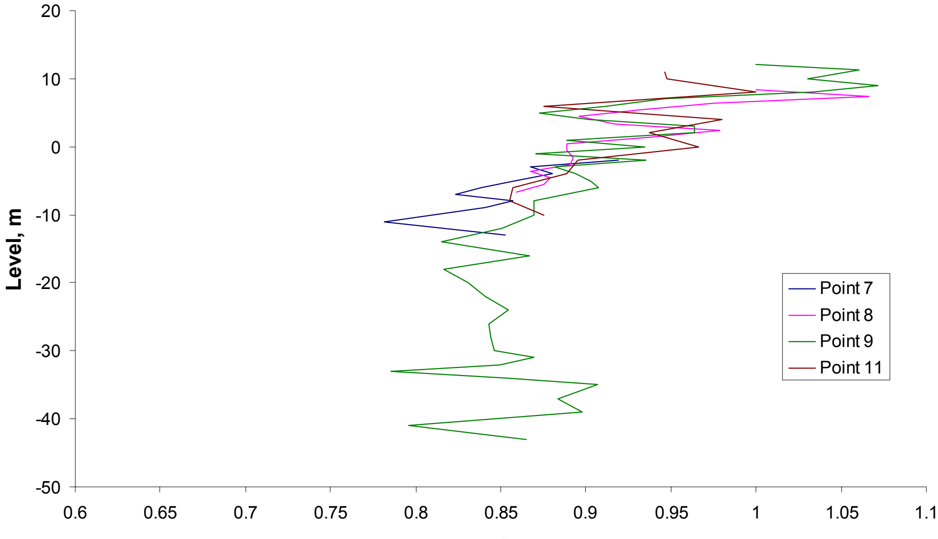
$$I_L = \frac{w_N - w_P}{w_L - w_P}$$

Where w_N = natural water content
w_P = plastic limit
w_L = liquid limit

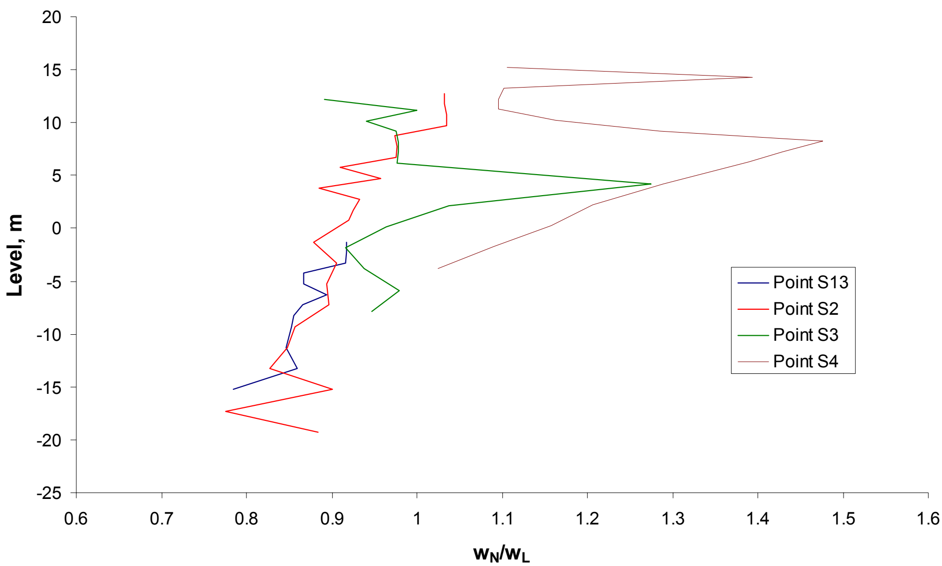
A determination of the liquidity index thus requires that the plastic limit be determined. For simplicity, a quasi liquidity index being the relation w_N / w_L may be used. This has previously been used in Sweden for correlation with sensitivity (Götaälvutredningen 1962), and has here been employed for both sections. The plot of the relation w_N / w_L in Section C shows the same very good correspondence for all points in the section, Fig. 23a. The same type of plot for Section A shows corresponding values for Point S2 below the excavation and Point S13 below the river bottom, Fig. 23b. This indicates a similar structure and "degree of consolidation" in relation to the composition of the clay, which is reflected in the liquid limit. The values from Point S3 behind the upper crest are similar, but at this point there is a zone in the central part of the clay layer with significantly higher values. At Point S4, 50 metres away from the crest, the values are considerably higher throughout the profile with a maximum at the middle of the clay layer.

A high liquidity index often entails a high sensitivity. When the sensitivity measured in Section A is plotted against the sampling level in the same way as the relation w_N / w_L, the same pattern is obtained with an elevated sensitivity at the middle of the clay layer at Point S3 and a high sensitivity throughout the profile at Point S4, Fig. 24a. It is here very high in the middle of the clay layer. The relation is also illustrated when the relation w_N / w_L in Section A is plotted against the sensitivity, Fig. 24b. In Section C, the variation is too small to illustrate this correlation and here all values fall into the same group as the values for Points 2 and 13 in Section A, i.e. relations between 0.8 and 1.05 and sensitivities between 10 and 40.

The indication of another type of clay at Point 4, which was found in the results of the CPT tests, was thus confirmed by the routine tests in the laboratory.

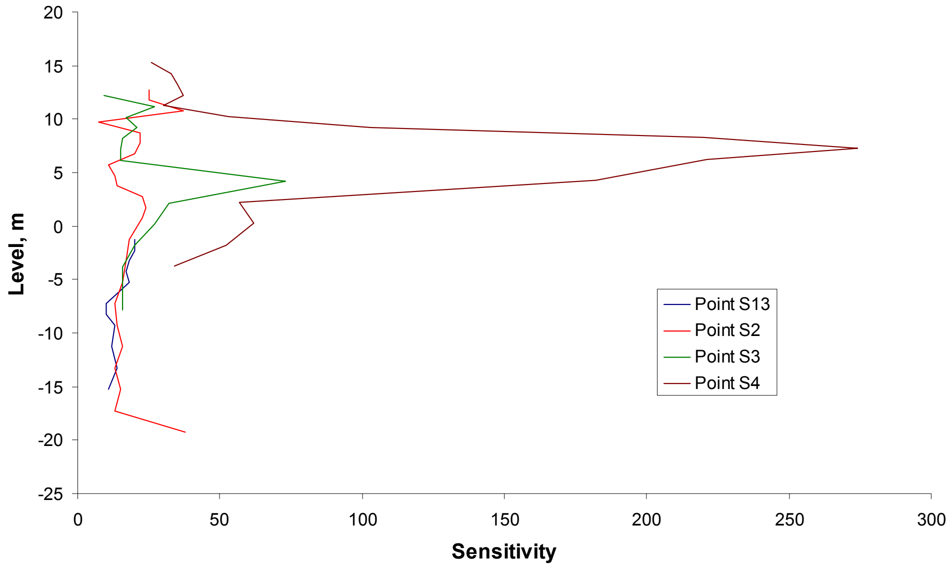


a)

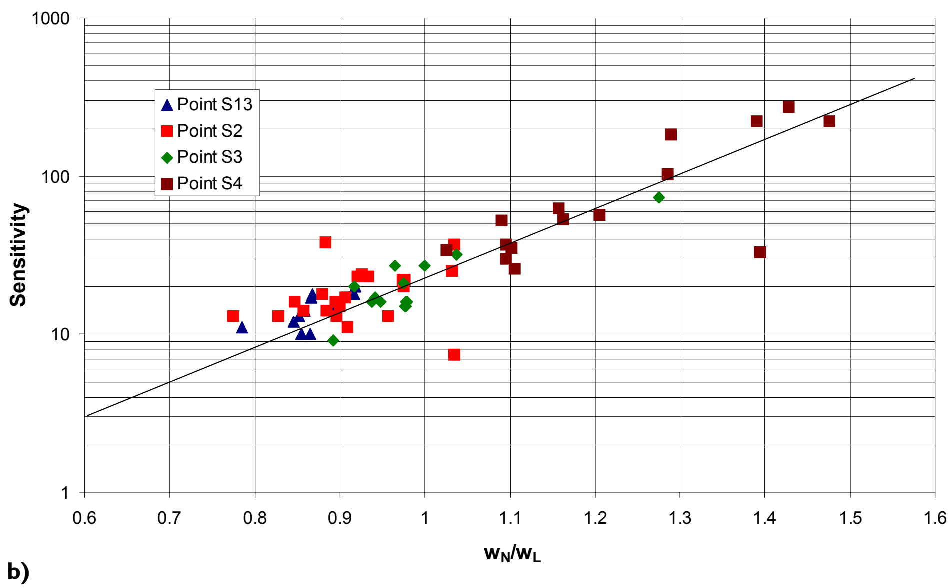


b)

Fig. 23. **The relation water content/liquid limit in the different sampling points in the Torp area.**
a) Section C
b) Section A



a)



b)

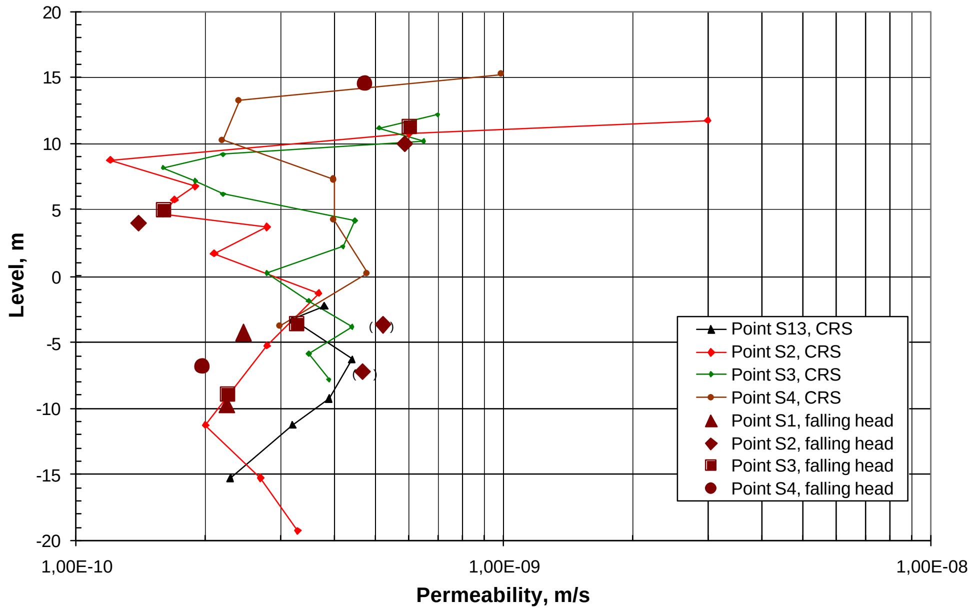
Fig. 24. **Sensitivity of the clay in Section A.**
a) Sensitivity versus level
b) Relation w_N/w_L versus sensitivity

2.7.2 Pore pressure conditions and variations

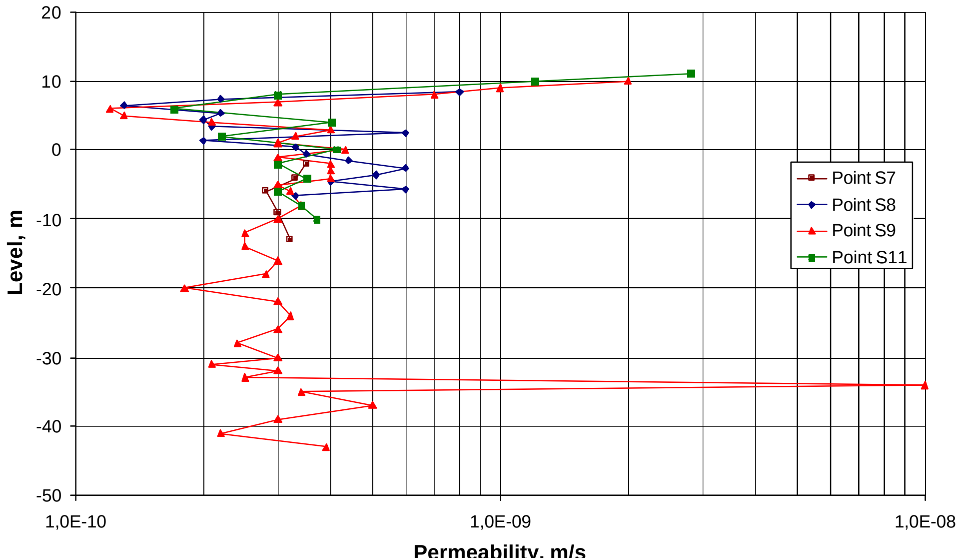
The pore water pressure in the clay can be assumed to be affected by infiltration from the ground surface through the more permeable sand and silt layers. It is also affected by seepage out through the slope and by the pressure levels in the river and in the coarser soil below the clay. In Section C, it can be assumed to be affected by the more permeable zone about 50–55 metres below the original ground surface, which can be assumed to be in contact with water transporting layers on the valley sides. The clay layer has a very low permeability. Determinations of the permeability by the oedometer tests and by "falling head" tests in filter tips in situ show that the permeability is about $2\cdot10^{-10}$ m/s in the silty clay directly below the upper sand and silt layers. It then gradually increases to about 4×10^{-10} m/s within the level interval +2 to –6 metres where the natural water content and the porosity also reach maximum values. The permeability then gradually decreases with depth along with a decreasing water content and denser soil structure and returns to about $2\cdot10^{-10}$ m/s at greater depths, Fig. 25.

A certain time lag can be expected before changes in the water pressures at the boundaries fully affect the pore pressures in central parts of the clay layer because of the low permeability. This also entails that seasonal variations at the boundaries cannot be expected to fully affect the pore pressures in central parts of thick clay layers since their duration is too short (e.g. Berntson 1983).

The pore pressure measuring systems in Section A were installed at the end of 1996 and the readings started shortly thereafter. The readings during the first six months showed irregular and shifting values in that the readings in some systems became stable and then showed only little variation whereas the readings in other systems kept changing in different trends without obviously approaching stabilised values, Figs. 26a and b. A year after installation, a function control was therefore performed and a renewed pore pressure observation started. This was done in such way that the inner hoses in the systems were filled up with water and the new pore pressure dissipation process was studied. At the same time, a few systems were replaced and a completely new system was installed in Point S2b located on the excavated surface close to the lower crest. Once again, it proved to take very long time for the pressures in the systems to stabilise, in many cases up to six months and in some case even longer than that. Thereafter, the pressures have mainly remained stable. A certain seasonal variation has been measured, primarily in systems placed in superficial layers and systems installed close to the draining bottom layer. A few readings indicating larger variations, which are not in agreement with the general pattern, are assumed to be erroneous.

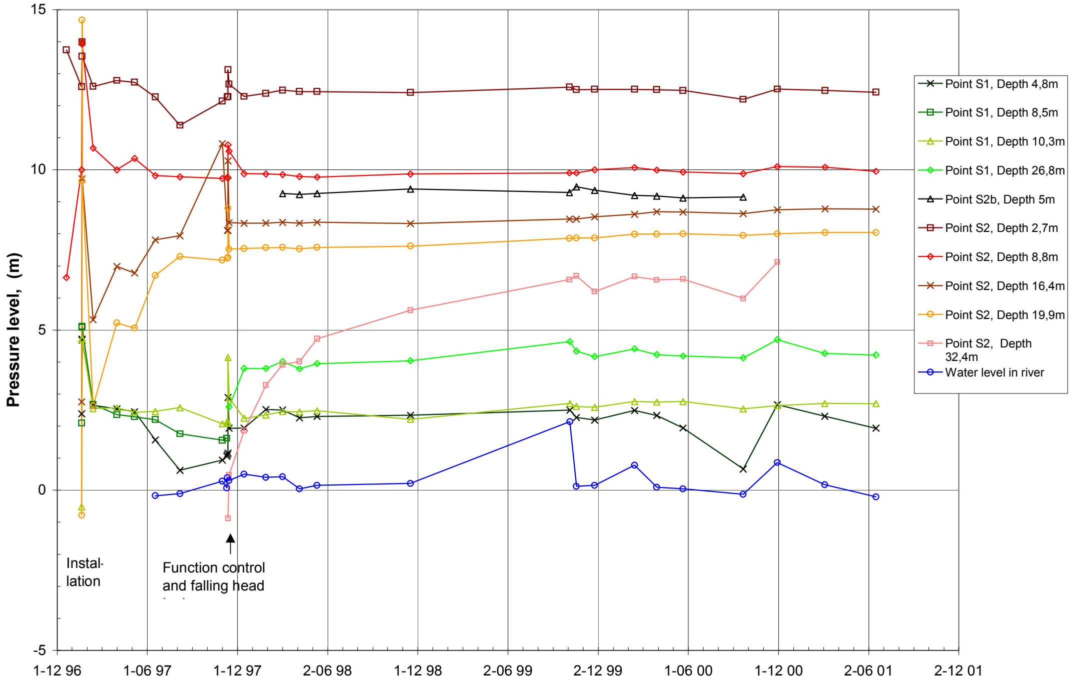


a)

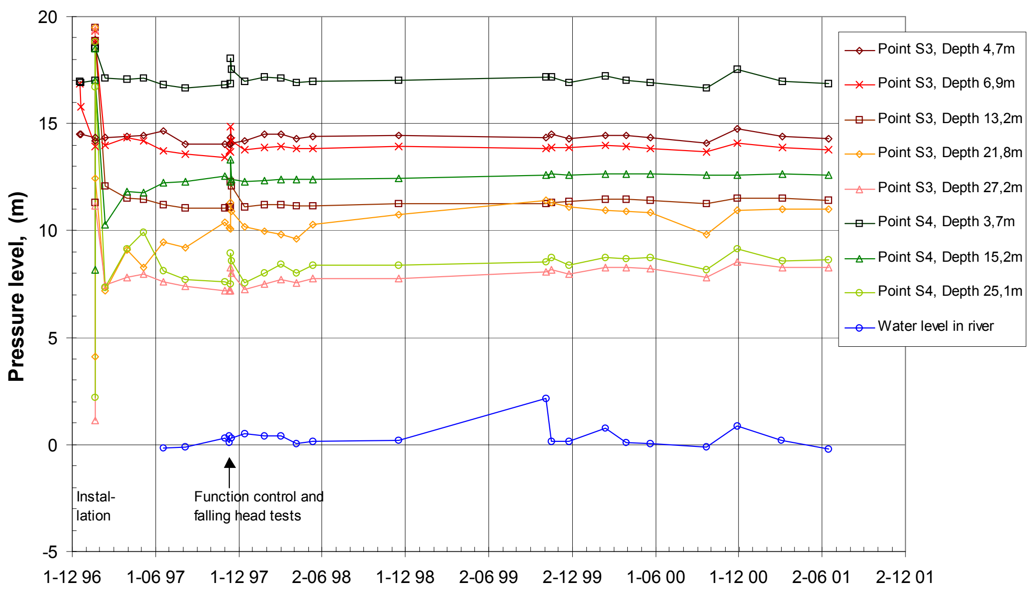


b)

Fig. 25. **Permeability in the test points in the Torp area determined by "falling head" tests in situ and CRS tests in the laboratory.**
a) Section A
b) Section C



a)



b)

Fig. 26. Measured pressure levels in the pore pressure measuring systems in Section A.
a) Points S1 and S2. b) Points S3 and S4.

The precision in the readings appears to have improved with time. This may be because the systems have become fully stabilised for installation effects. It may also be due to the fact that problems related to reading off the systems have been observed and amended and that the same person, who has been aware of these possible problems, has conducted the measurements. A certain regularity in the small variations has thereby been observed. The variations in general follow the variation in external water supply. This is in turn reflected in the water transport and water level in the river, the latter of which has been monitored simultaneously.

The principle for the reading of the open systems is very simple. It uses an electrical coaxial cable which is inserted into the inner hose until it reaches the water surface in the hose. The electrical conduit in the cable is then short-circuited, which is registered by an ampere-meter on the ground. The length of the inserted cable is then measured using markers on the cable and a carpenter's rule. In spite of the simplicity, measuring errors may occur because of condensed water on the inner walls of the hose, contact problems at the end of the cable or in the portable instrument, general problems with moisture or simply errors in measuring the inserted cable length.

The variations measured in pore pressure during the last years generally lie within 10 kPa. A somewhat larger variation has been measured at the upper measuring level at Point S1, which may be assumed to be in relatively good contact with the water level in the river, whereas lower variations are measured in the central parts of the clay layer. The pore pressure distributions at the different points are approximately linear from free groundwater levels in or slightly below the ground surfaces to a pressure level in the permeable bottom layers, which below the river and its banks is at about +7 metres. This means that the pressure level in the permeable bottom layer here is about 7 metres above the mean water level in the river. Further up in the slope and in the area behind this, the pressure level in the bottom layers increases to +8.5 metres as a maximum. This indicates a certain resistance towards water flow in the bottom layers as well. The pore pressures below the excavated area show a certain influence from higher ground in the area behind and a certain extra pressure elevation can thus be seen in the upper layers. The pore pressures have a downward gradient in the upper area and below the excavated part, whereas there are artesian water pressures at the toe of the slope and below the river, Fig. 27.

Compared to the values that were measured near Point S2 before the excavation, the pore pressures have been lowered both below the excavation and close to the upper crest. The effect is evident through most of the clay layer. Before the

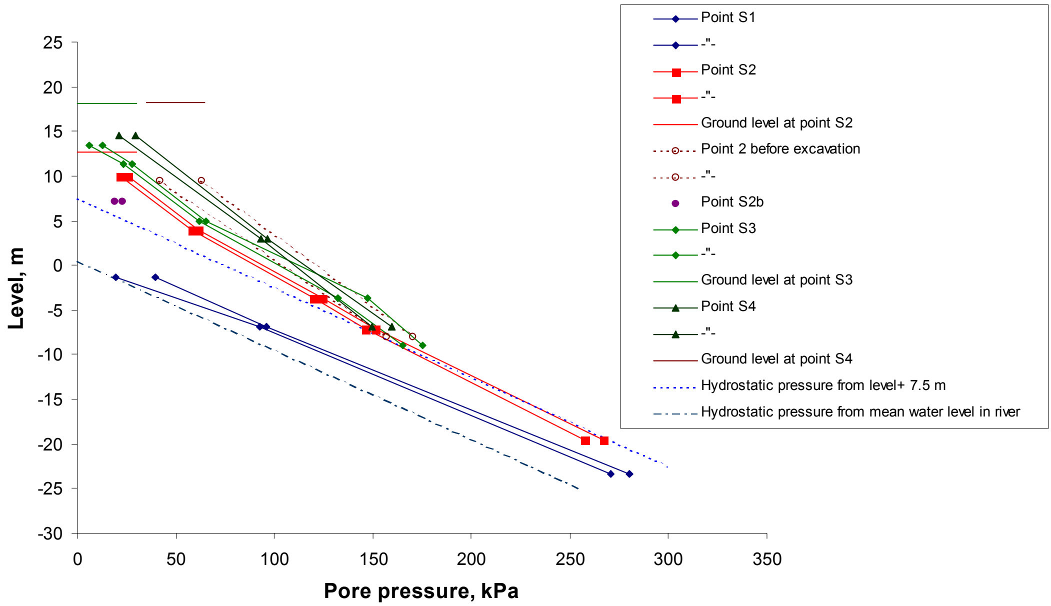


Fig. 27. **Pore pressure distribution and measured variations in Section A.**

excavation, the pore pressures in locations at various distances from the old crest seem to have been similar to those that are measured today at corresponding distances behind the new upper crest. This appears to have been the case in spite of the fact that the upper sand and silt layers were considerably thicker close to the old crest than those that are found behind the new crest.

As mentioned above, the variations in pore pressure that were measured with open systems in the middle of the clay layer were very small. The later installed closed systems showed variations that were considerably larger in a relative sense. These comparison systems were installed at Point 2, i.e. under the excavated area, at 8.8 and 16.4 metres depth. The maximum measured variations increased from about 3 kPa in the open systems to about 20 kPa in the closed ones. The trends in the variations generally agree with the net supply of water to the area. The highest pore pressures are thus measured during late autumn and early spring, whereas the lowest ones are measured at the end of summer and early autumn. This is also reflected by the water level in the river, even if the groundwater conditions in the part of the slope where these pore pressure measuring systems were installed is not directly affected by this level, Fig. 28. The water level in the river is also not entirely related to precipitation and snow melting in the catchment area of the river but to some extent also to the regulation of its water transport.

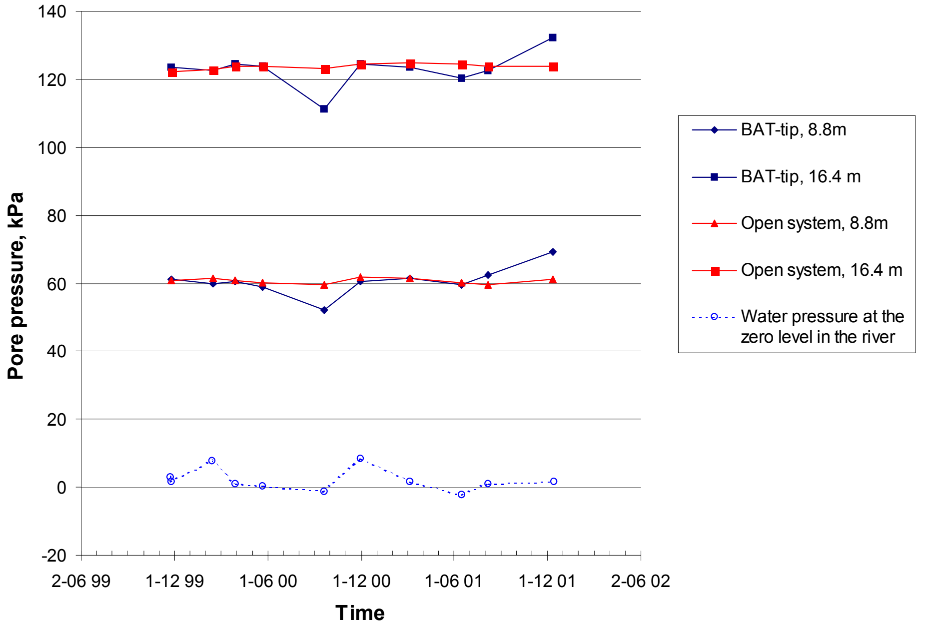


Fig. 28. Pore pressure variations in the clay layer in Section A measured with open and closed systems.

The closed pore pressure systems in Section C were installed during October 1999 and the measurements started one week after installation. The installation effects were then evened out and the pore pressures had stabilised. The pore pressure conditions in this section are primarily affected by infiltration of water from the ground surface and the water pressure in the permeable zone at great depth. No pore pressure systems have been installed on the riverbank, but the pore pressure conditions in the upper layers here can be assumed to be affected by the water level in the river in the same way as in Section A.

The infiltration from the ground surface is strongly affected by the catchment area behind the slope, where water infiltrates through the permeable sand and silt layers and then flows towards the slope on top of the underlying less permeable layers. The up to 10 metres thick sand and silt layer is stratified and contains a relatively impermeable layer, and two separate aquifers with different pressure levels are therefore found within the layer. The seasonal variation of these two pressure levels is about 0.5 metres at a distance of 35 metres from the upper crest. At the crest, where the pressures are lower because of the seepage of water out of the slope below, this variation has practically ceased.

Thirty-five metres behind the crest, the free groundwater levels corresponding to the pressure levels are located about 3 and 6 metres below the ground surface. At the crest, where the ground level is 0.5 metres lower, the corresponding depths are 4 and 6 metres. The water seeping out of the upper slope continues to flow on top of the fine-grained soil below the erosion protection in the upper slope and the coarse base on the upper excavated terrace. Because of this continuous water supply and the limited possibility for evaporation, the variation in the upper groundwater level should be very small in these parts. The water is then streaming down to the lower excavated terrace and in principle follows this surface to the lower crest of the slope down to the river. The ground surface of the lower terrace is very marshy and no significant seasonal drying has been observed. The upper free groundwater level in the slope is thereby almost stationary and significant variations are only found far behind the upper crest and down at the river. The absence of measured variations can to some extent be due to the relatively wet summers during the observation period.

The absence of variations on the upper groundwater level is also reflected in the pore pressures measured in the underlying soil layers. A variation of only a few kPa has been measured in the upper clay layers. However, the variation increases close to the permeable layers at large depths and here a variation of about 10 kPa has been measured, Fig. 29. The pore pressure variation in these layers appears to be largest in the area behind the slope, which is closer to the valley sides, and to decrease towards the river.

Like the conditions in Section A, the pore pressure distribution is approximately linear between the upper free groundwater level and the more permeable zone, Fig. 30. The pressure in the latter zone corresponds approximately to a hydrostatic pressure from a level of +7 metres, which is about the same as for the bottom layers below the river in Section A. As with the conditions in Section A, there is also a certain deviation from the general pattern in the pore pressures in the upper clay layers under the excavated areas, which can be assumed to be due to an influence from the surrounding parts of the slope. The measured gradients are downward in the upper parts of the slope but at the toe of the slope and below the river the pressures are artesian.

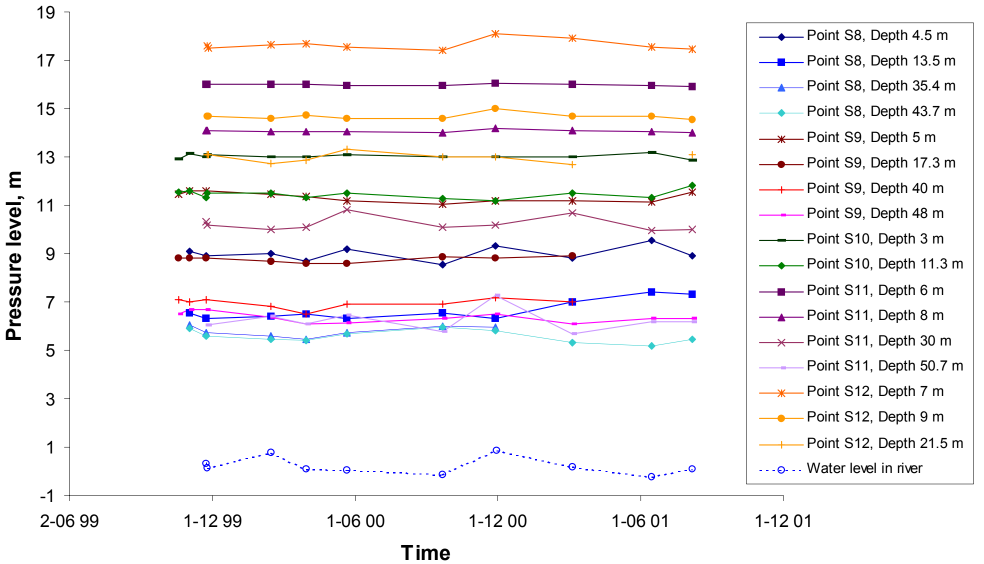


Fig. 29. Measured variation in pressure levels in Section C.

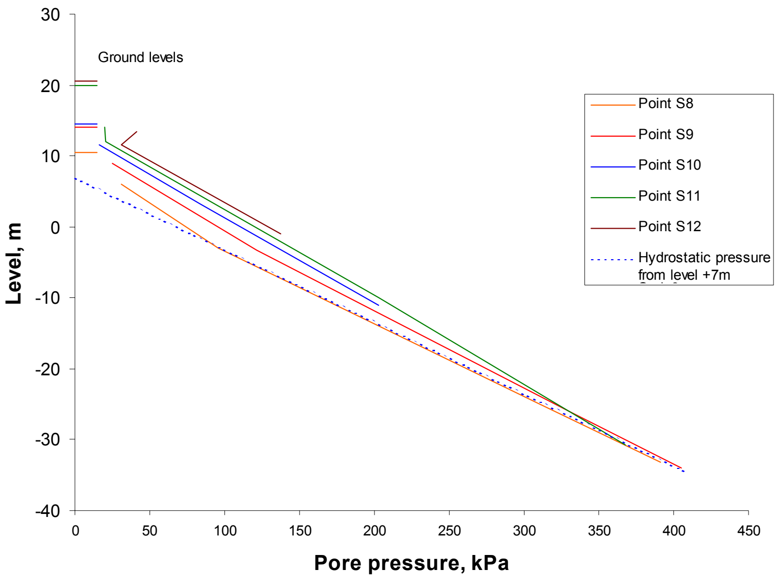


Fig. 30. Measured pore pressure distribution in Section C. The small measured variations are not included in this figure.

2.7.3 Stress history and stress conditions

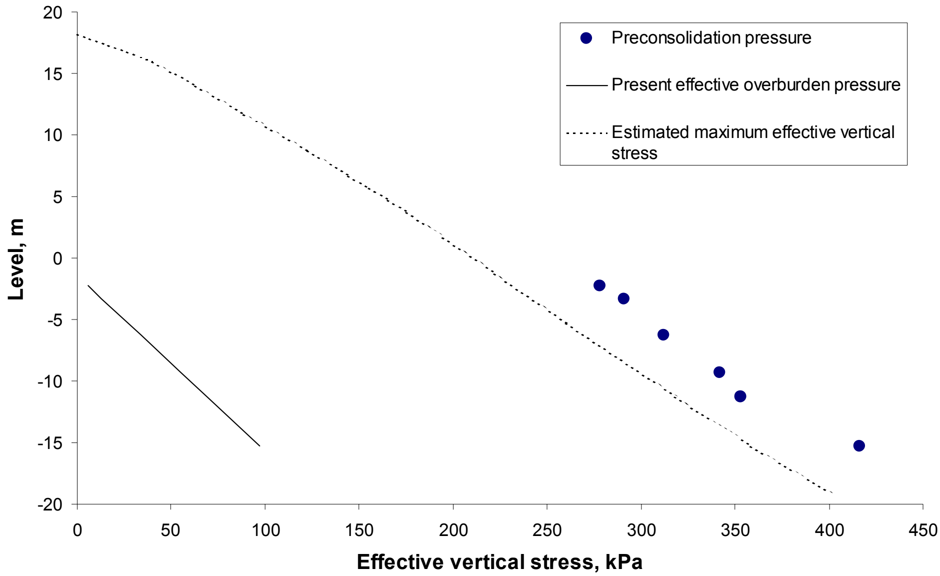
The soil in the sections can generally be assumed to have consolidated for the self-weight of the soil layers. These can be assumed to have had a thickness corresponding to an almost horizontal ground surface level with the surrounding non-eroded and non-excavated ground. The pore water pressures at this initial stage may be assumed to have been higher than at present. The groundwater level particularly in the coarser sand and silt layers close to the river has then been lowered successively as the river has eroded its channel down through the soil layers. The present groundwater situation can only have prevailed since the excavations were performed about 15 years ago. Any significant consolidation for increased effective stresses resulting from the lowered groundwater level after the excavation can only have occurred in superficial layers directly behind the upper crest above the excavated area, i.e. at Point S3 in Section A and Point S11 in Section C. It is more difficult to estimate how the water pressures in the bottom layers have varied during the simultaneous land-elevation and erosion process. However, it may be assumed that the present pressure conditions here represent the lowest pressures so far and that they have in principle prevailed for a long time.

A large number of oedometer tests have been performed. The results of the tests in Section A are presented separately for each borehole since the soil conditions here vary somewhat, Fig. 31.

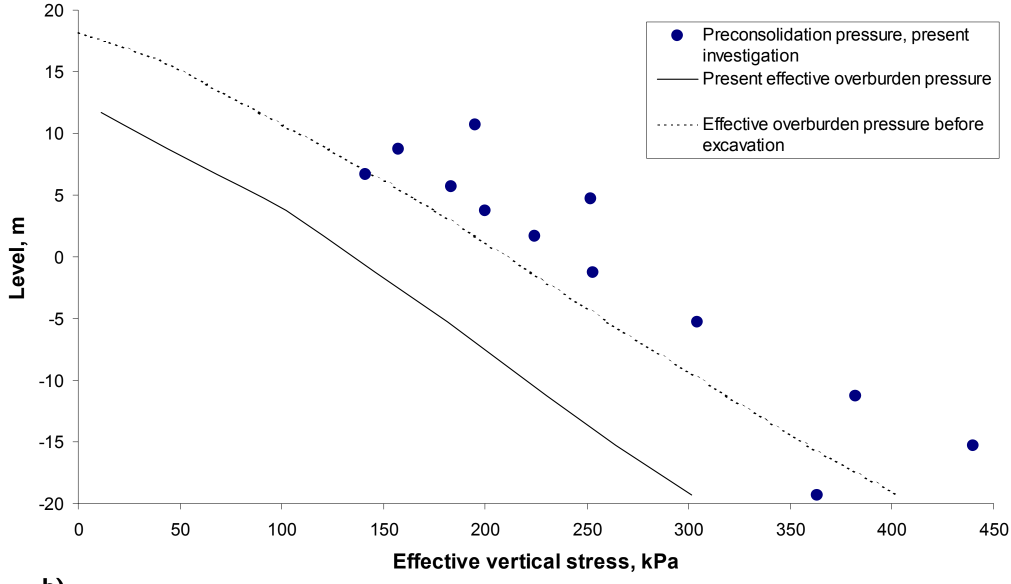
The results at Points S13 and S2 show that the soil has consolidated for the assumed maximum effective overburden pressure and has also obtained a slight overconsolidation in relation to this. Because of the later unloading, the soil in these points is now overconsolidated or heavily overconsolidated in relation to the present stress condition.

The results at Point S3 just behind the upper crest in principle also show that the soil is slightly overconsolidated in relation to the assumed maximum effective overburden pressure. However, in a zone around the level +7 metres, the test results show almost normally consolidated conditions. This zone coincides with the zone where both the relation w_N/w_L and the sensitivity are elevated. The oedometer tests on the two lowest levels also show normally consolidated conditions, but this is assumed to be related to disturbance of the samples, which contained clay mixed with coarser material.

The results at Point S4 generally show a normally consolidated clay. This is in spite of the fact that the estimated maximum effective stresses here are somewhat lower

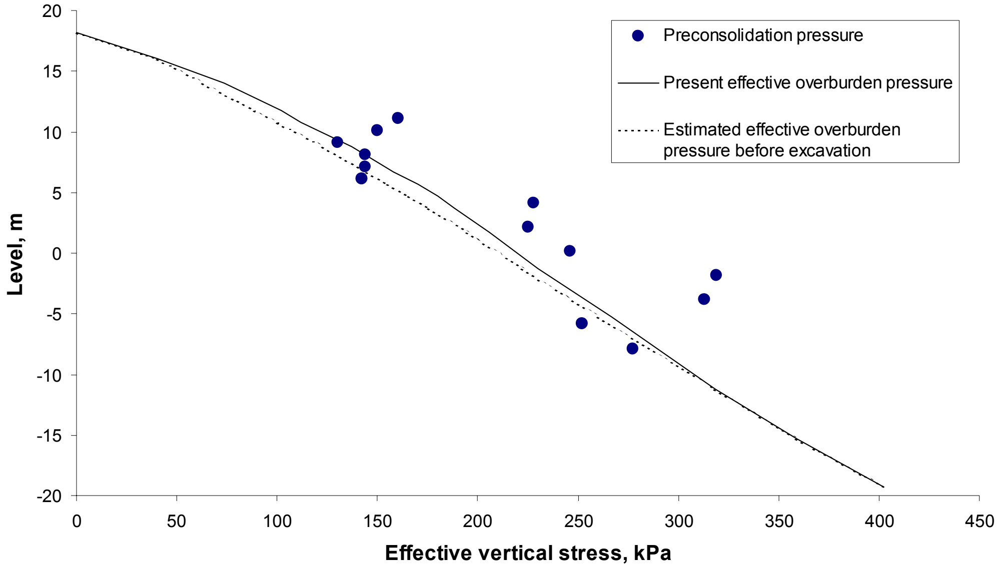


a)

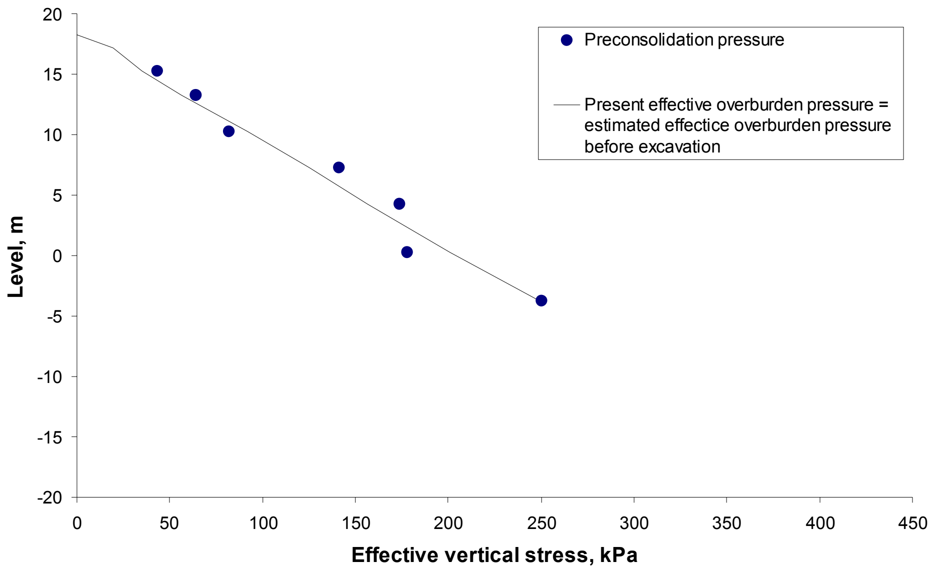


b)

Fig. 31. Evaluated preconsolidation pressures compared to assumed maximum effective overburden pressure in Section A.
a) Point S13 below the river
b) Point S2 in the excavated area



c)



d)

Fig. 31. Evaluated preconsolidation pressures compared to assumed maximum effective overburden pressure in Section A.
c) Point S3 behind the upper crest
d) Point S4, 50 metres behind the crest

because of a thinner sand and silt layer and higher pore pressures. The clay at this point is different from that at the other points in other respects too and has a considerably higher sensitivity and w_N/w_L relation.

A compilation of all results shows that the preconsolidation in the main part of the investigated soil mass is similar and corresponds to an overconsolidation ratio of about 1.15 in relation to the assumed previous maximum effective overburden pressures, Fig. 32. Exceptions are Point S4 and a small depth interval at Point S3, which show normally consolidated conditions for the prevailing stresses. It is possible that a small previous overconsolidation here has been broken down by different circumstances, such as leaching.

The results of the CPT tests also show that the soil behind the excavated area is almost normally consolidated for the assumed maximum previous effective overburden pressure and that the values at Point S4 are lower than at the other points. The results also show that the evaluation method that was proposed in SGI Information No. 15 (Larsson 1991), strongly overestimates the preconsolidation in homogeneous overconsolidated soil and that this increases with increasing overconsolidation ratio.

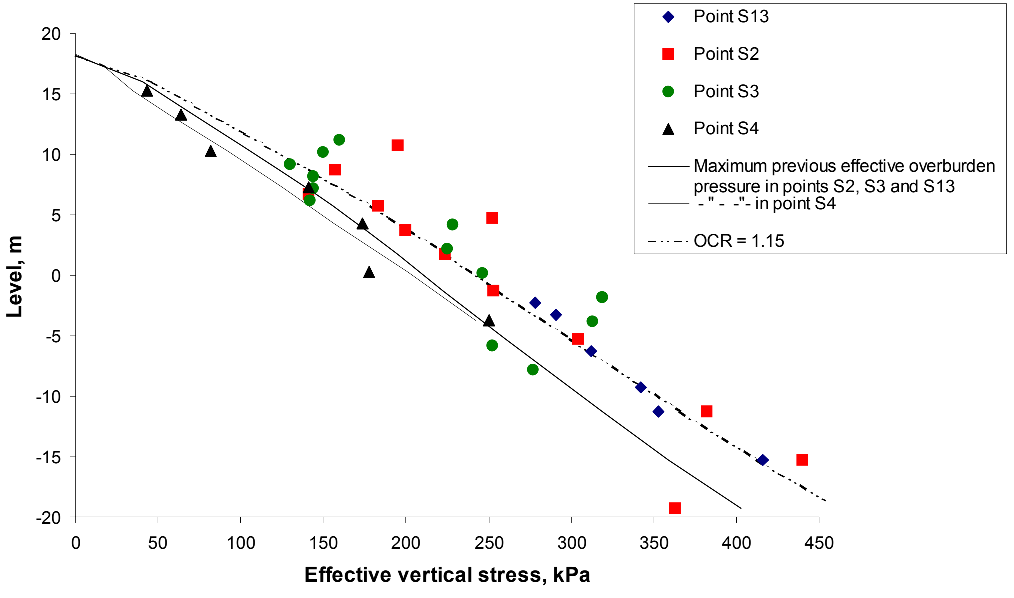


Fig. 32. **Compilation of evaluated preconsolidation pressures from CRS tests in Section A.**

In the evaluation method proposed in SGI Information No. 15, the evaluated preconsolidation pressure is corrected with respect to the overconsolidation ratio. The method was proposed on the basis of the investigations in Swedish clays made by Larsson and Mulabdic (1991) combined with results from Norway and the United Kingdom presented in literature. According to that compilation, the preconsolidation pressure could be evaluated from

$$\sigma'_c = \frac{q_T - \sigma_{v0}}{(1.21 + 4.4 w_L)(1.07 - 0.54 \log OCR)}$$

where σ'_c = preconsolidation pressure
q_T = total tip resistance
σ_{v0} = total overburden pressure
w_L = liquid limit
OCR = overconsolidation ratio

However, the Swedish investigations in principle comprised normally consolidated and only slightly overconsolidated clays with an average overconsolidation ratio of about 1.3, and there was no national experience of overconsolidated soils. The correction for overconsolidation was introduced mainly to take the fissured nature of the overconsolidated dry crusts into account. Later investigations in clay till have shown that this method strongly overestimates the preconsolidation pressure in this type of soil (Larsson 2000). New experience from Canadian clays also shows that no correction should be made for overconsolidation in the evaluation of preconsolidation pressure in this way (Demers and Leroueil 2002). Without correction for overconsolidation the evaluation is simplified to

$$\sigma'_c = \frac{q_T - \sigma_{v0}}{1.21 + 4.4 w_L}$$

When the results in Section A in Torp are evaluated in this way, a relatively good correlation is obtained with the results from the oedometer tests regardless of where in the slope the borehole is located and what overconsolidation ratio the clay has, Fig. 33. No correction for overconsolidation ratio should thus be made in this homogeneous type of clay in which the state of consolidation varies from normally consolidated to heavily overconsolidated and the overconsolidation is a result of a real unloading.

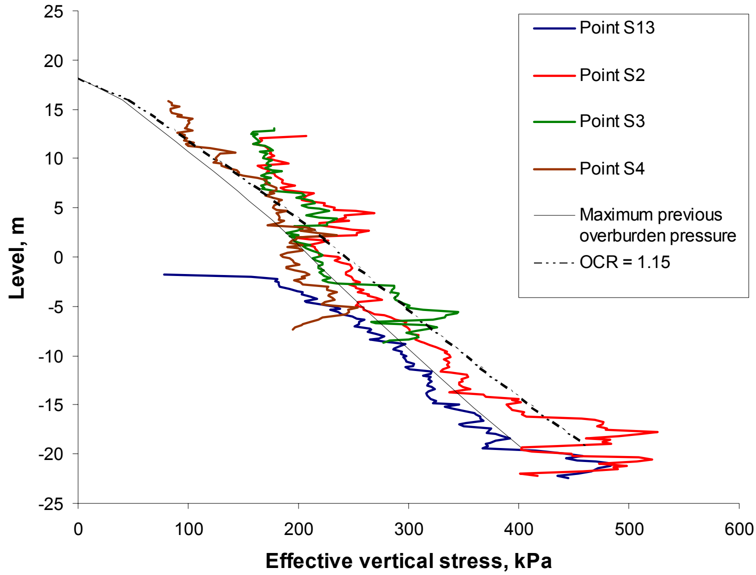


Fig. 33. Evaluated preconsolidation pressures from CPT tests in Section A.

The soil conditions in Section C were very similar in the different boreholes. The results of all oedometer tests have therefore been compiled and show very similar conditions in the section regarding preconsolidation as well, Fig. 34. The results scatter somewhat, but the trends in all test points show that the clay has consolidated for the assumed maximum effective overburden pressures and has become a small overconsolidation ratio of about 1.15 in relation to these. The results of tests on samples taken at great depths at Point S9 fall out of this general picture, but these samples were taken in the zone with coarser soil and embedded sand and silt layers. The samples can therefore be assumed to be significantly disturbed. Also the samples from the clay below this zone may be assumed to be partly disturbed because of infusions of coarser material. Undisturbed samples of a very good quality have been taken with the Swedish standard piston sampler from considerably greater depths in other locations (e.g. Claesson 2003), but this has then been done in more homogeneous and high-plastic clays.

The clay under the river and at the riverbanks has become heavily overconsolidated because of the erosion. The overconsolidation has also increased at Points S8 and S9 because of the excavation. At Point S11, the effective stresses may be assumed to have increased somewhat because of the lowered groundwater level. However,

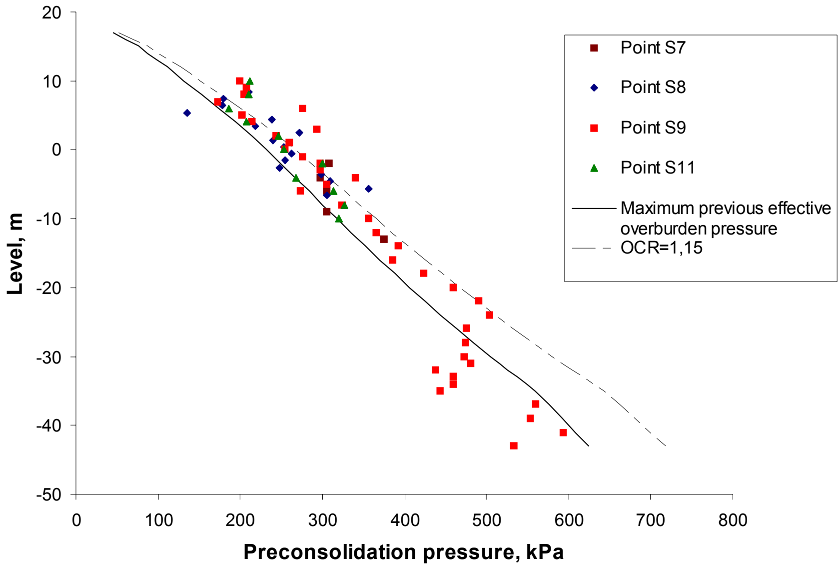


Fig. 34. Evaluated preconsolidation pressures from CRS test in Section C.

this has not entailed that the preconsolidation pressure has been exceeded. Whether or not the stress increase has nevertheless brought any extra consolidation and increase in preconsolidation pressure because of creep effects cannot be inferred from the test results.

The stress and consolidation conditions are thus similar along the investigated area except for the part behind the upper crest in Section A where the clay has obtained different properties, which are probably a result of leaching.

Similar preconsolidation pressures to those evaluated from the oedometer tests were evaluated from the results of the dilatometer test. More strictly, it is the overconsolidation ratio that is evaluated from the dilatometer tests and in the evaluation method employed this ratio can never be less than 1.0 (Larsson 1989). This limitation is to some extent reflected in the results from greater depths, Fig. 35. A partly revised method for the evaluation of the overconsolidation ratio for dilatometer tests in overconsolidated soil is presented further on in the report (Chapter 3.8), but this does not affect the evaluation in the normally or only slightly overconsolidated soil at Point S11.

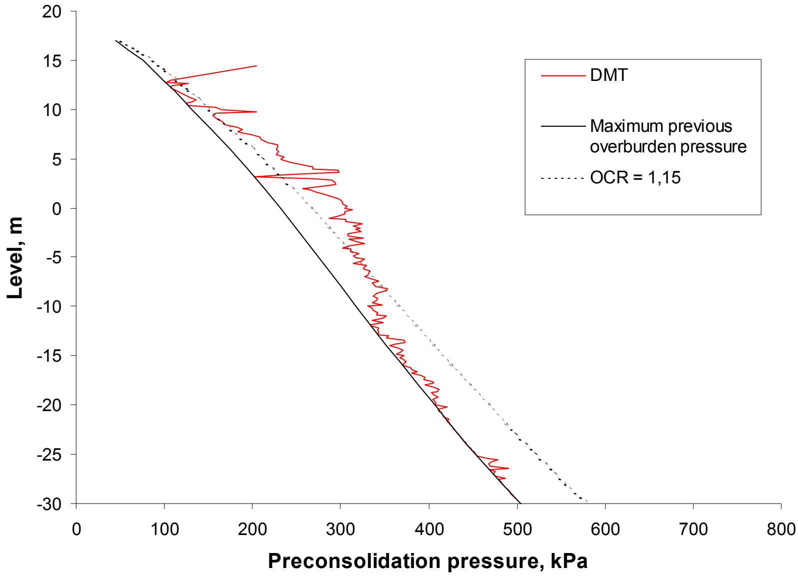


Fig. 35. **Evaluated preconsolidation pressures from dilatometer tests at Point S11 in Section C.**

The preconsolidation pressures evaluated from the CPT tests showed the same overestimation of the preconsolidation pressure in zones with homogeneous overconsolidated clay as in Section A when a correction for overconsolidation ratio was applied. When this correction was omitted, concordant results were obtained which were similar to those obtained from the oedometer tests, even if the values in general were somewhat too low, Fig. 36. However, the empirical evaluation is very sensitive to the given liquid limit and the results lay within the normal band of scatter. The general recommendation, see e.g. SGI Information No. 15, is therefore that the evaluation from CPT tests should be used only in combination with results from oedometer tests. The oedometer tests are then used to determine the value of the preconsolidation pressure on a number of levels and the evaluation from the CPT tests is primarily used as a support for the estimated distribution of the preconsolidation pressure with depth.

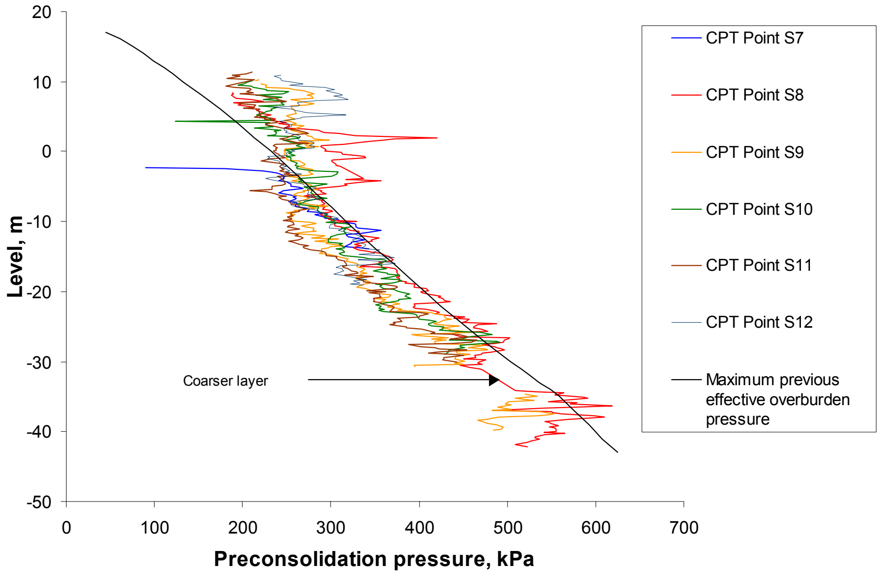


Fig. 36. **Evaluated preconsolidation pressures from the CPT tests in Section C.**

2.7.4 Shear strength

General

The undrained shear strength normally varies with effective overburden pressure, overconsolidation ratio and liquid limit (or plasticity index). According to empirical experience from direct simple shear tests and triaxial tests, the undrained shear strength can be evaluated from

$$c_u = a \cdot \sigma'_v \cdot OCR^b \qquad or \qquad c_u = a\sigma'_c\, OCR^{b-1}$$

where c_u = undrained shear strength
σ'_v = effective vertical pressure
OCR = overconsolidation ratio, σ'_c / σ'_v
σ'_c = preconsolidation pressure

(Ladd et al. 1977, Jamiolkowski et a.l 1985) *a* and *b* are material constants. *a* varies with the mode of loading. In active triaxial tests, it is generally found to be around 0.33 in Swedish clays. In direct simple shear tests, the factor has been found to vary somewhat with the liquid limit and in passive triaxial tests this variation is more pronounced (Larsson 1980). There may also be a small variation with the plasticity for the active triaxial tests (Westerberg 1999). In direct simple shear tests, which also approximately represent the average shear strengths, the factor *a* has an average value for clays of 0.22, but it is normally lower for low-plastic clays and higher for high-plastic and organic clays. The factor *b* normally varies between 0.75 and 0.85.

Series of undrained direct simple shear tests have been performed on clay from three levels in Munkedal. In these series, specimens from the same sample tube, or from the middle and lower sample tubes from the same point and level, have been consolidated for an effective vertical stress just below the preconsolidation pressure and then unloaded to different overconsolidation ratios. After adjustment to the new vertical stresses, the undrained shear strength has been measured. These series have been performed on samples from level +7 metre at Point S2 in Section A and on levels +2 and –8 metres at Point S9 in Section C. The liquid limits varied from 42 % to 69% between the samples, i.e. within a fairly limited range. The results from the test series coincided with the general picture and the evaluated factors *a* and *b* ranged from 0.21 to 0.23 and 0.77 to 0.85 respectively, Fig. 37. No particular trend for variation of the factors with liquid limit could be observed from these results.

Apart from these tests, a number of ordinary direct simple shear tests have been performed. In these tests, the specimen are first consolidated for stresses just below the preconsolidation pressure and then unloaded to the in situ effective vertical stress. Thereafter the specimens are sheared to failure in undrained conditions. The results of these tests have been plotted together with the results from the special test series, Fig. 38. The compilation shows that the undrained shear strength in direct simple shear in the investigated soil in the Torp area with a good approximation can be expressed with the general equation and the factors $a = 0.22$ and $b = 0.8$.

All relations between consistency limits and other soil properties are here using the liquid limit in accordance with Swedish practice. For inorganic clays, an approximate translation to plasticity index can be made using the relation $I_P \approx 0.8(w_L - 0.18)$

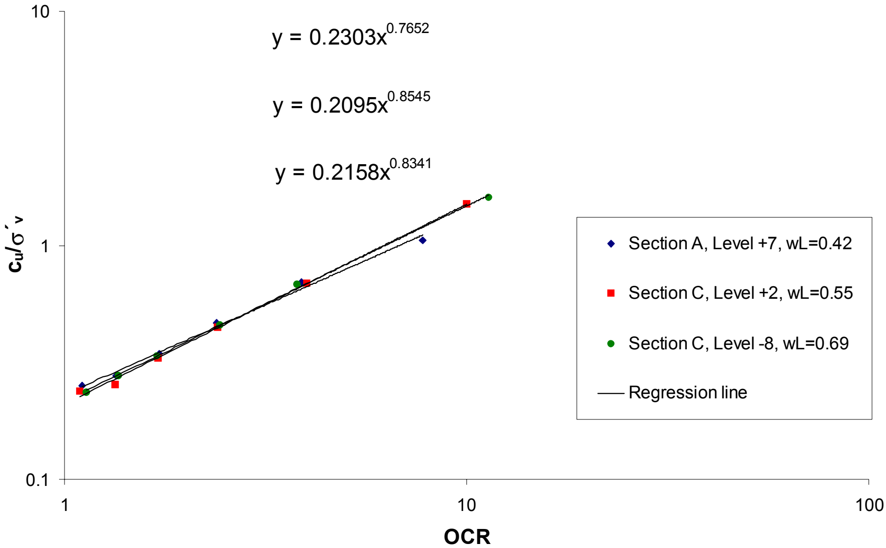


Fig. 37. **Results of series of direct simple shear tests with different overconsolidation ratios.**

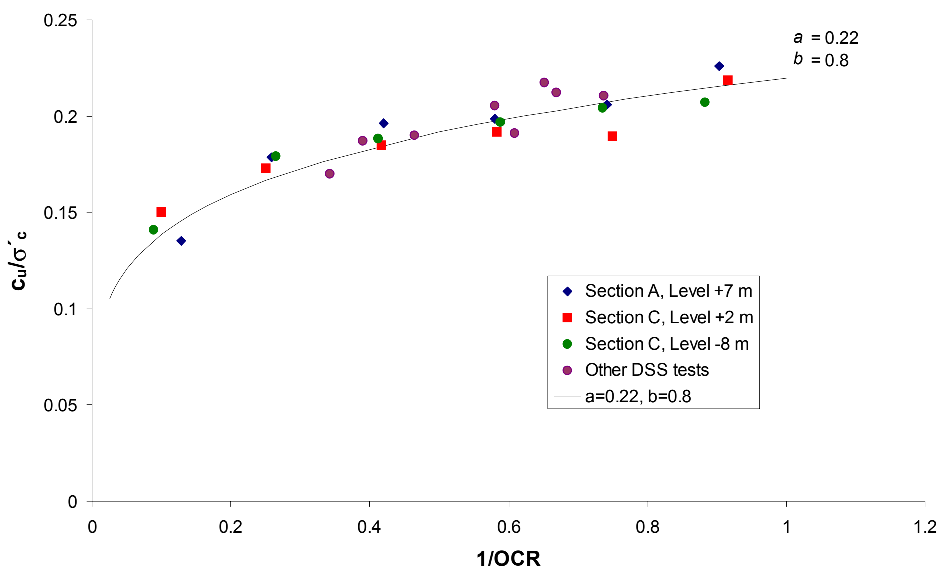


Fig. 38. **Measured shear strengths in direct simple shear tests on clay from Sections A and C in Torp normalised against preconsolidation pressure and plotted versus ratio of unloading, (1/OCR).**

Shear strength determinations in Section A

The undrained shear strength in Section A has been determined by field vane tests, CPT tests, fall-cone tests on undisturbed samples and supplementary direct simple shear tests and triaxial tests on reconsolidated specimens in the laboratory.

Evaluation of the undrained shear strength from CPT tests is normally made by the equation

$$c_u = \frac{q_T - \sigma_{v0}}{N_{KT}}$$

where the cone factor N_{KT} varies depending on what shear strength is referred to, i.e. active shear strength, shear strength at direct simple shear or some other case. For the case of direct simple shear, which normally also corresponds to the corrected shear strength from field vane tests, it has empirically been found that N_{KT} for Swedish slightly overconsolidated clays, OCR ≈ 1.3, can be written

$$N_{KT} = 13.4 + 6.65 w_L \qquad \text{(Larsson and Mulabdic 1990)}$$

where w_L is the liquid limit in decimal number.

The relation $c_u = a \cdot \sigma'_v \cdot OCR^b$ is well established and is incorporated in most basic models for the shear strength of clays, e.g. SHANSSEP (Ladd and Foott 1974) and Critical State Soil Mechanics (Schofield and Wroth 1968, Wood 1991). However, the preconsolidation pressure is also evaluated from the net cone resistance $(q_T - \sigma_{v0})$ from the CPT test divided by an empirical cone factor. This means that either the evaluated shear strength or the preconsolidation pressure, or both, should be corrected for the overconsolidation ratio. In the previously mentioned evaluation according to the guidelines in SGI Information No. 15, the evaluated preconsolidation pressure was corrected. Since it has now been shown that no such correction should be made, it follows that the evaluated undrained shear strength should be corrected instead. No rules for how this should be done have been found in previous literature, but in order to be consistent and compatible it should follow the same principles as in the other shear strength determinations and established soil models. In accordance with this, the evaluation equation can be rewritten as

$$c_u = \frac{q_T - \sigma_{v0}}{N_{KT}} \cdot \left(\frac{\sigma'_c}{1.3\,\sigma'_{v0}} \right)^{b-1}$$

where q_T = total tip resistance

σ_{v0} = total overburden pressure

N_{KT} = cone factor = $13.4 + 6.65w_L$

w_L = liquid limit in decimal number

σ'_{v0} = effective overburden pressure

σ'_c = preconsolidation pressure (can be evaluated from the same CPT test as described in the previous section with the restriction that $OCR = \sigma'_c/\sigma'_{v0} \geq 1$)

1.3 = the overconsolidation ratio for which N_{KT} is empirically determined

b = material constant, set to 0.8 from empirical experience or calibrated through laboratory tests

The correction entails that significantly lower undrained shear strengths are evaluated in heavily overconsolidated clays whereas marginally higher values are evaluated in clays with unusually low overconsolidation ratios, i.e. OCR < 1.3. However, for most "normally consolidated or slightly overconsolidated" clays the correction does not entail any significant difference.

In the evaluations made in this project, the material constant *b* has been set to 0.8 unless some other value has been specified in the text.

Field vane tests were performed at Points S13, S2 and S3 and were doubled at the two last points. No field vane tests have been performed at Point S4, but only fall-cone tests at the routine testing in the laboratory and a CPT test. These tests gave generally lower values of the undrained shear strength than those at corresponding levels in the other test points did.

Vane tests had been performed adjacent to Point S1 in a previous investigation and, since the stress conditions had not changed much since then, no new tests were performed here.

A comparison between the different shear strength determinations in each point shows that a fairly good agreement between the different methods was obtained at Point S3 behind the upper crest, Fig. 39. The field vane tests and the fall-cone tests

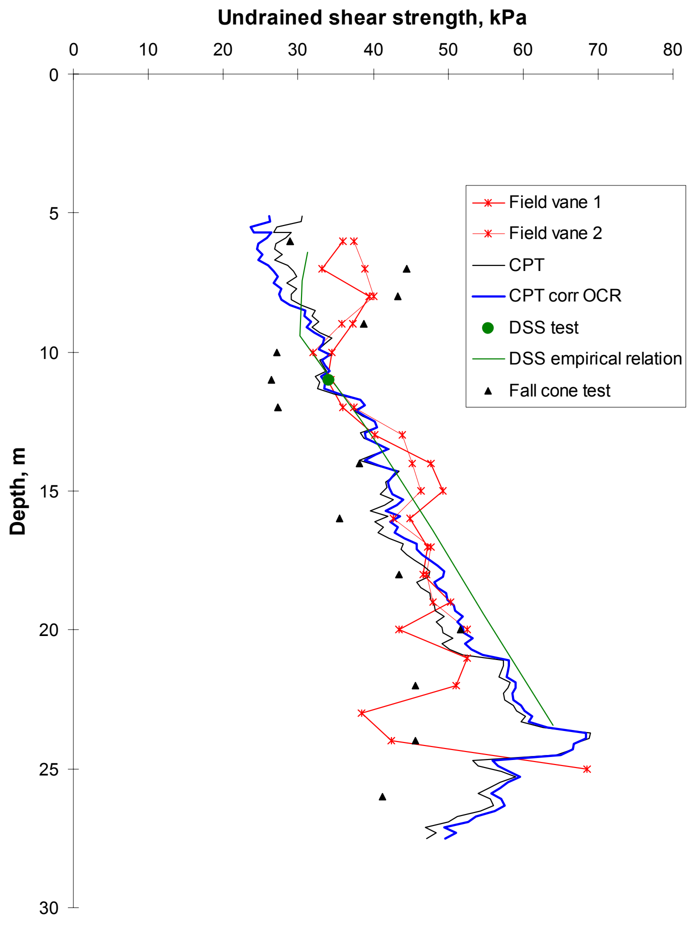


Fig. 39. **Results of shear strength determinations at Point S3 behind the upper crest.**

gave higher values than the CPT test and empirical values for direct simple shear down to a depth of about 9 metres. This may be related to the presence of silt layers, coarser particles and organic matter that may have affected the results. At larger depths, the agreement is relatively good apart from a few odd values. At greater depths the fall-cone tests gave lower strength values, but this is a normal effect of the stress relief at sampling. At this point, the soil is only slightly overconsolidated and the correction of the CPT evaluation for overconsolidation ratio had a very small effect.

Roughly the same relations between the shear strength determinations were obtained at Point S2 on the excavated area. The zone with higher shear strength values from field vane tests and fall-cone tests is here limited to the upper 4 metres, since about 5 metres of soil at the top have been taken away, Fig, 40. A large number of direct simple shear tests have been performed on specimens from this point and the results from both these and the verified empirical relation can be used for comparison. The upper part of the soil profile at this point is significantly overconsolidated. The correction of the CPT tests for this had a significant effect and gave a better correlation with the results from the direct simple shear tests in this part.

The shear strength determinations at Point S1 on the riverbank are not fully compatible. The vane tests were performed from a ground surface which was lower and closer to the river, whereas the CPT tests may be affected by the erosion protection and fills in connection with this. The soil at this point is overconsolidated throughout the profile and the correction of the CPT values entails a considerable difference at all depths, Fig. 41.

The correlation between the different shear strength determination varies at Point S13 below the river, where the overconsolidation ratio is largest. A reduction in undrained shear strength because of the unloading corresponding to that according to the empirical evidence for direct simple shear was only obtained in the fall-cone tests and the corrected CPT tests, whereas the field vane tests gave higher values. However, the trends for the shear strength at larger depths are similar, Fig. 42. The overconsolidation ratio at this point is so high that the absolute value of material parameter *b* has some significance in the evaluation. A use of the empirical value of 0.8 generally gave somewhat lower strengths than what corresponds to direct simple shear, whereas use of a factor of 0.85, as calibrated in the direct simple shear tests on this level, resulted in a good agreement.

According to experience, field vane tests evaluated in the common way do not normally show the same reduction in shear strength at unloading as laboratory tests (e.g. Jamiolkowski et al. 1985). However, a compilation of all shear strengths measured by field vane tests in the section shows that according to these results too there is a strong reduction in shear strength at the riverbank and below the river bottom because of the unloading that has occurred here, Fig. 43. On the other hand, no significant difference between Points S2 and S3 as an effect of the more moderate unloading that has been made here can be inferred from the compiled results. The comparison is obstructed by the influence of the silt layers and organic content in the upper layers and also by the partial reduction in strength around the

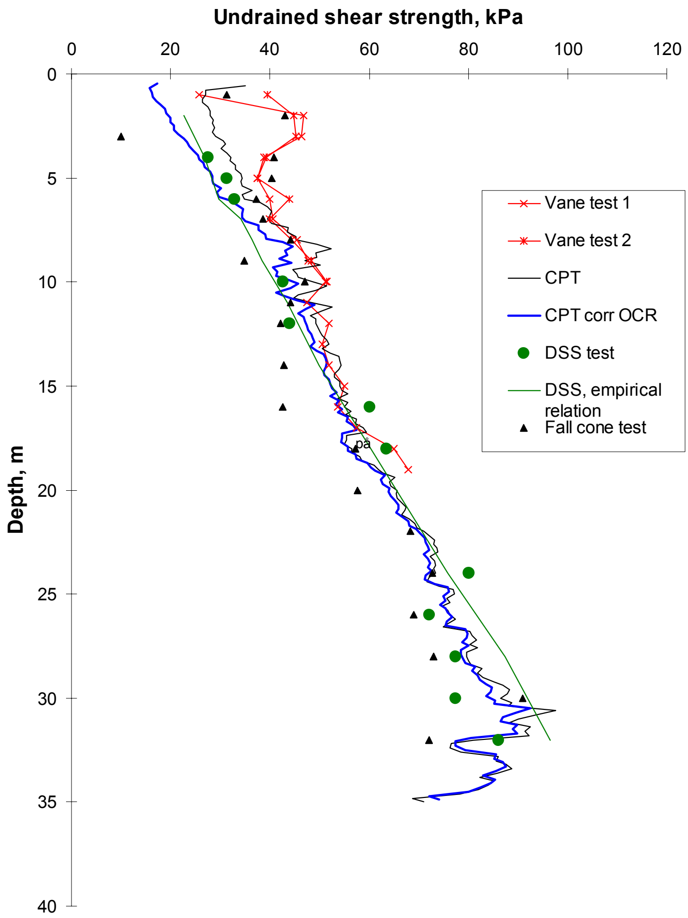


Fig. 40. **Results of shear strength determinations in Point S2 in the excavated area.**

level +7 metres at Point S3 that is indicated by the results of the oedometer tests. A detailed comparison is also rendered difficult by the fact that the liquid limit at the same levels varies somewhat between the test points. According to empirical experience, the shear strength before unloading should thereby have been somewhat larger in the lower parts of the slope and have decreased gradually with distance from the river. However, even if an effect of the unloading on the evaluated shear strengths from the field vane tests can be observed, it is generally much smaller than what could be expected from the common soil models.

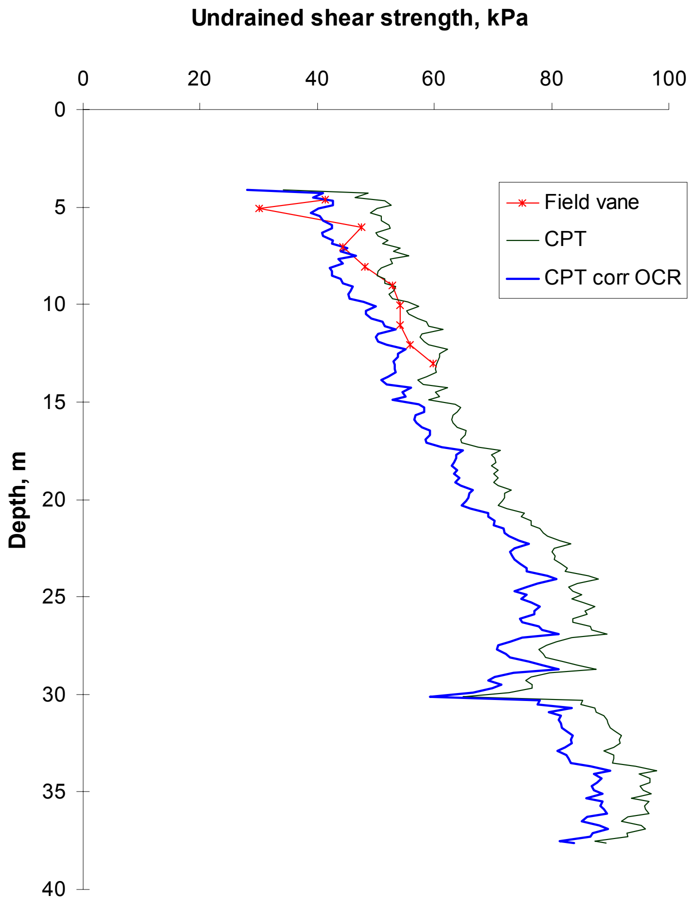


Fig. 41. Results of shear strength determinations at Point S1 on the riverbank.

A compilation of the results from the CPT tests show that the shear strength far behind the upper crest is lower than at the crest, Fig. 44. No significant effect of the unloading can be detected when no correction is made for the overconsolidation ratio. The effect of the unloading can then only be seen in the uppermost couple of metres below the river bottom. The scatter in the results is relatively large and the possible evaluated effects of the unloading are far lower than those expected from the empirical evidence. When the results are corrected for the overconsolidation ratio, a considerable effect of the unloading is obtained, particularly below the river and the riverbank but also in the superficial layers below the excavation. The picture is disturbed by the significantly lower values at Point S4, where the clay has

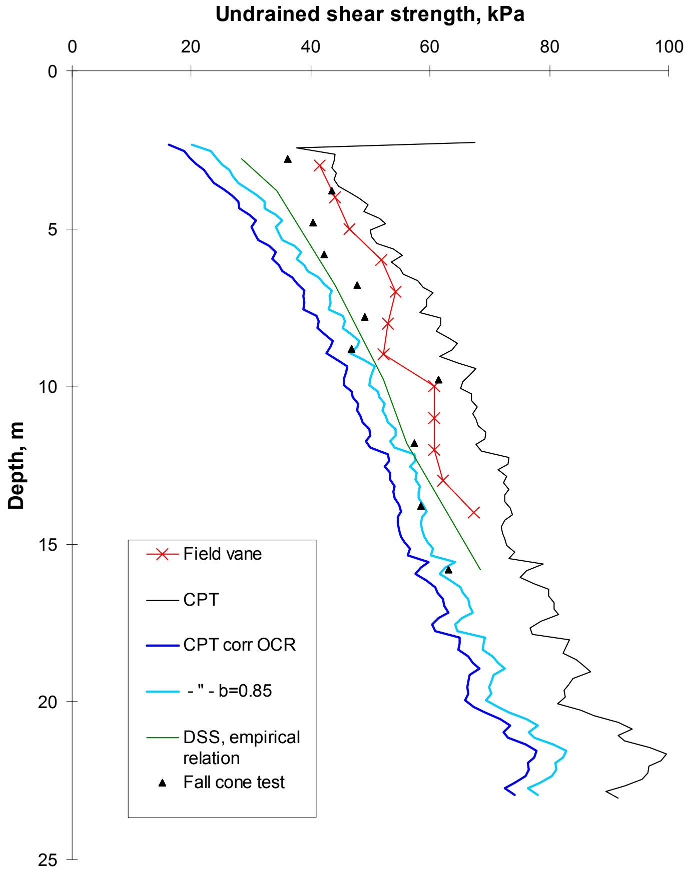


Fig. 42. **Results of shear strength determinations at Point S13 below the river.**

been affected by other factors. When the results from this point are overlooked, the results are in general agreement with what could be expected according to established soil models.

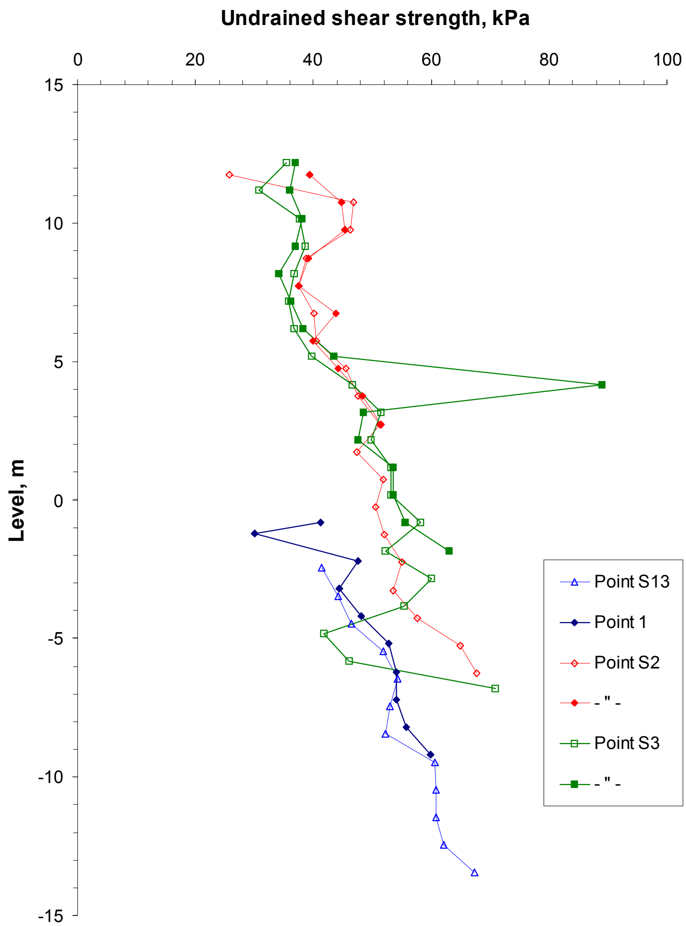


Fig. 43. **Compilation of the shear strengths determined by field vane tests in Section A. (The values in Point 1 come from a previous investigation.)**

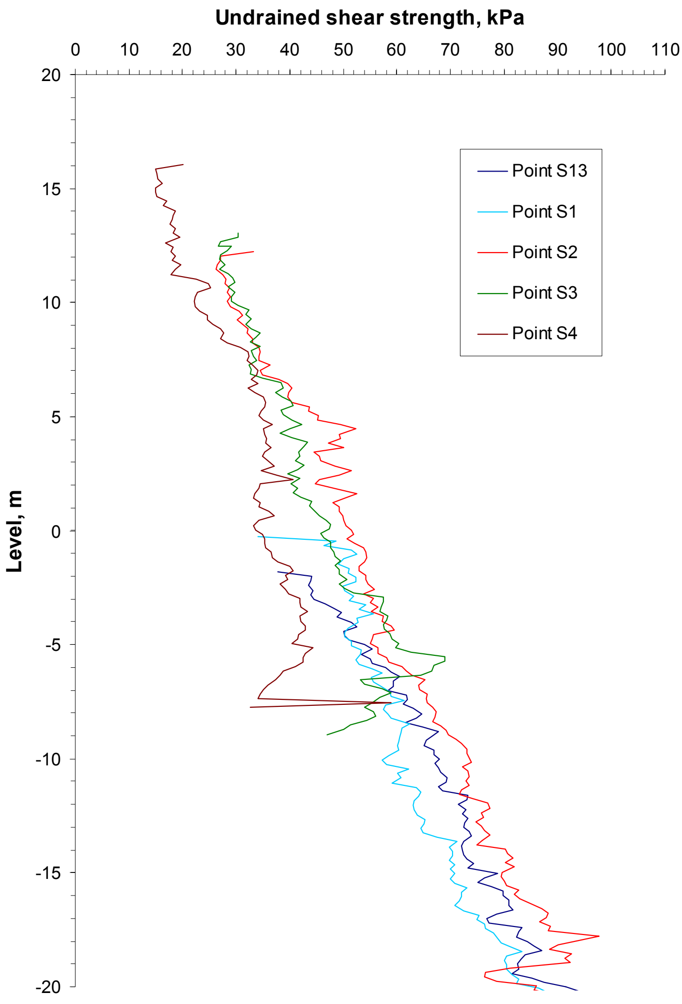


Fig. 44. **Compilation of shear strengths evaluated from CPT tests in Section A. a) without correction for overconsolidation ratio**

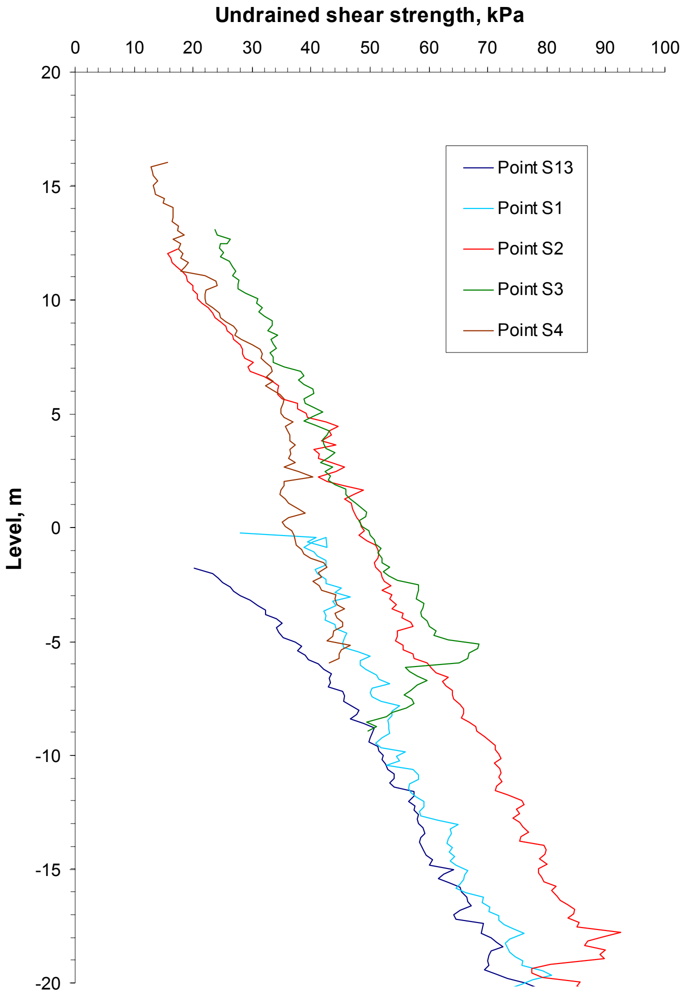


Fig. 44. **Compilation of shear strengths evaluated from CPT tests in Section A. b) with correction for overconsolidation ratio**

Anisotropy

The anisotropy of the shear strength often plays a significant role in assessments of slope stability. It is often estimated empirically, but has to be verified before being used in the final assessment. A number of active undrained triaxial tests have therefore been performed on specimens from different depths in Point S2 in Section A. The specimens were reconsolidated at stresses just below the preconsolidation stresses and then allowed to adjust for the in situ stress conditions before the undrained strength tests were performed.

The active undrained shear strength of clay is estimated empirically in the same way as the shear strength at direct simple shear but with a factor *a* of 0.33. The active shear strength in Munkedal is thus estimated as

$$c_{u_{ACTIVE}} = 0.33 \cdot \sigma_v \cdot OCR^{0,8}$$

A comparison between the measured shear strength values in the triaxial tests and empirically estimated values is shown in Fig. 45. The agreement is good and the validity of the empirical relation can be considered as confirmed.

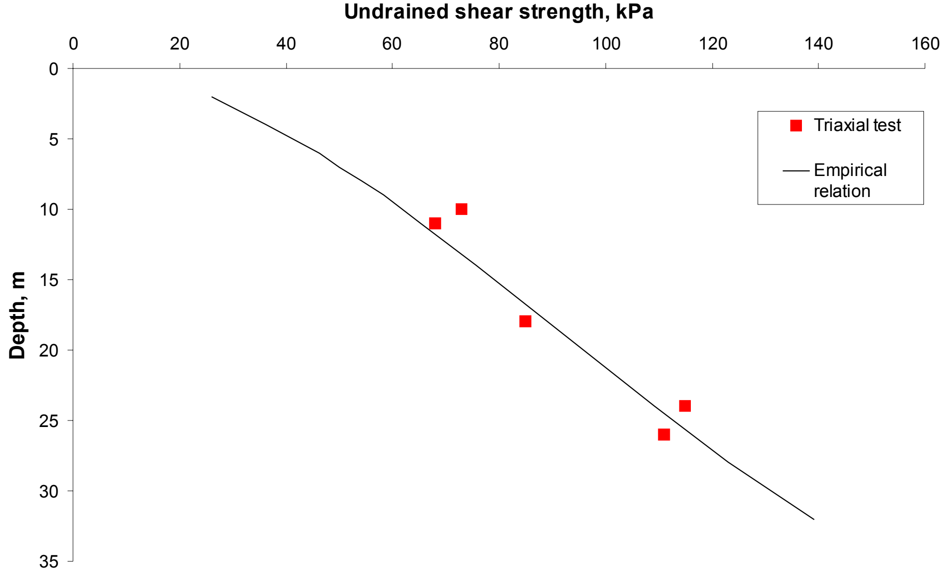


Fig. 45. Active undrained shear strength at Point S2, Section A.

Effective shear strength parameters

The effective shear strength parameters c' and ϕ' have been evaluated from the effective stress paths in the undrained tests. The value of ϕ' is normally assumed to be 30° and c' is estimated as $c' \approx 0.1\,\tau_{fu}$ or $c' \approx 0.03\sigma'_c$ from empirical relations for Swedish clays. The triaxial tests were performed on specimens from different levels and the evaluated values varied correspondingly. The stress paths corresponded well to a friction angle of 30° and a c' value of about 6 kPa for a depth of 10–11 metres and about 9 kPa for the depth interval 18–26 metres. The corresponding values from the empirical relations are 5–6 kPa and 7–10.5 kPa respectively.

Undrained tests on water-saturated specimens are performed with the restriction that the volume is constant and the evaluated friction angle corresponds to ϕ'_{cv}, (cv – constant volume). The mobilised shear strength measured in drained tests corresponds to the friction angle at constant volume plus the effects that develop because of the change in volume of the specimen during shear. At low stresses, i.e. below the preconsolidation pressure, the soil will dilate (increase its volume). This effect is positive for the shear strength and can be expressed as a c´ value in addition to the friction at constant volume. The influence of the dilatancy (or contractancy) that occurs under drained conditions can thus not be measured directly in undrained tests. A number of drained tests with different stress paths have therefore been performed on specimens from depths of 14 to 22 metres at Point S2. The stress paths have been selected in such a way that the effective stresses at failure have not exceeded the preconsolidation stresses and have varied in such way that the influence of the stress level within this range could be studied. Within this stress range, the value of c' was found to vary between about zero just at the preconsolidation pressure and 14 kPa at very low stresses. The relation between c' and the stress level is curved. Within the stress region in which the drained shear strength is lower than the undrained shear strength, and thereby may be the governing strength, the c' values from the drained tests were equal to or somewhat larger than the values evaluated from the stress paths in the undrained tests.

The empirical relations that are normally used for Swedish clays were thus found to be applicable also to this type of clay.

Shear strength determinations in Section C

The undrained shear strength in Section C has been determined in a corresponding way by field vane tests and CPT tests in the field and fall-cone tests, direct simple shear tests and triaxial tests in the laboratory. The shear strength has also been evaluated from the dilatometer tests at Point 11. According to the recommendations in SGI Information No. 10 (Larsson 1989), the undrained shear strength, τ_{fu}, is evaluated from dilatometer tests as

$$\tau_{fu} = \frac{p_1 - \sigma_{h0}}{10.3}$$

where p_1 = expansion pressure in the dilatometer test
σ_{h0} = the horizontal earth pressure at rest (calculated from the test results)
10.3= empirical factor for clay (9.0 for gyttja)

In international practice, the evaluation of undrained shear strength from dilatometer tests is normally made using the general equation for how the undrained shear strength varies with effective overburden pressure and overconsolidation ratio (Marchetti 1980). The empirical values $a = 0.22$ and $b = 0.8$ are then normally used and the undrained shear strength is calculated as

$$c_u = 0.22\ \sigma'_{v0}\ OCR^{0.8}$$

The effective overburden pressure is estimated from the results of the dilatometer test or is calculated using measured densities and pore pressures. The overconsolidation ratio is evaluated from the dilatometer test results. As for the estimation of preconsolidation pressure, it is then important to use a relevant method for the estimation of the overconsolidation ratio. In this report, both methods of estimation of the undrained shear strength have been used, and it has become evident that only the latter method estimates a reduction because of unloading corresponding to the effect that is indicated by the general equation.

Field vane tests have been performed at Points S7, S8, S9 and S11 and CPT tests at Points S7, S8, S9, S10, S11 and S12. The aforementioned larger investigation with comparison of different field vane equipment and vane sizes was also performed at Point S9.

The shear strength values evaluated at Point S7 below the river have about the same sizes and relations between the different methods as was found at Point S13 with the corresponding location in Section A, Fig. 46. A strong reduction in shear strength in relation to what would be obtained in normally consolidated clay with the same preconsolidation pressure was found here too. The effect of the unloading is thus strong and the CPT tests and the fall-cone tests yielded values corresponding

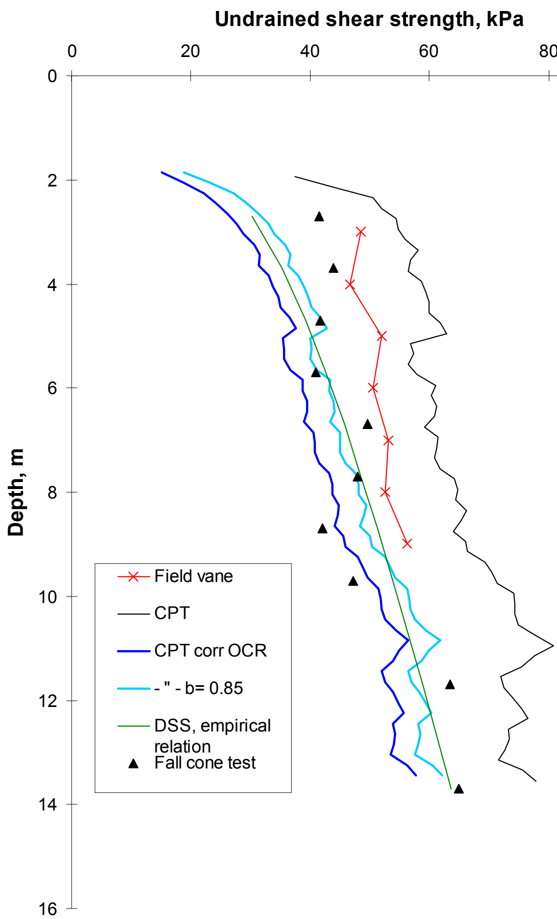


Fig. 46. **Shear strength determinations at Point S7 below the river in Section C.**

to those expected from empirical relations. The field vane tests gave higher values, particularly in the upper soil layers. As with the trends at Point 13, the different shear strength determinations tend to converge at great depths.

The shear strength determinations agree better at Point S8 on the lower excavated terrace, Fig. 47. A direct comparison is obstructed at a number of levels with embedded layers with sand and shell remnants. These have resulted in enhanced tip resistance in the CPT tests, whereas they have rather resulted in too low values in the field vane tests and the fall-cone tests at the same levels. Correspondingly low values of the preconsolidation pressures have also been measured in the CRS oedometer tests, which indicates that the samples have been partly disturbed at these levels. The influence of the correction of the CPT test is moderate and entails that the results come closer to what can be expected at direct simple shear.

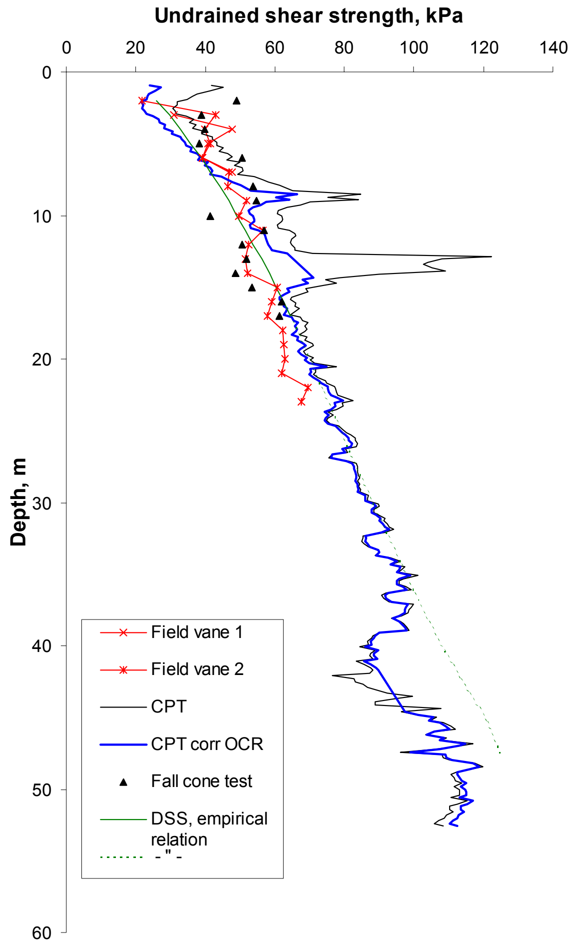


Fig. 47. **Shear strength determinations at Point S8 on the lower excavated terrace in Section C.**

Point S9 is located on the upper excavated terrace. A number of direct simple shear tests and triaxial tests have been performed here, apart from the usual tests. At this point, the determined shear strength values vary in relation to each other depending on depth and type of soil, Fig. 48. In the upper 7 metres or so, the field vane tests yield considerably higher values than can be expected from empirical data. The soil in this depth interval consists mainly of clayey silt with infusions of sand. Below this follows about 6 metres of silty clay with a relatively high density and low water content and liquid limit. In this layer, the field vane tests and the direct simple shear tests agree well, whereas the CPT tests yield somewhat higher values. At a depth of about 13 metres, there is a pronounced change in the character of the soil to a more high-plastic clay and below this depth the shear strength values determined by the different methods agree well. The values from fall-cone tests are generally

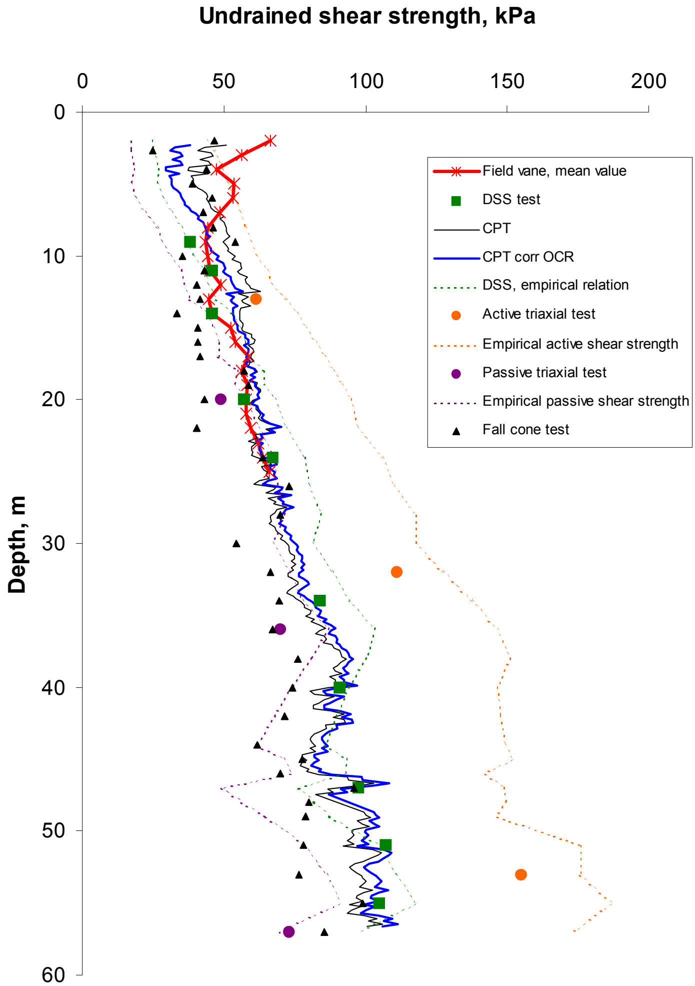


Fig. 48. Shear strength determinations at Point S9 on the upper excavated terrace in Section C.

lower from about 13 metres' depth, which is normal. The pattern is similar to that found at Point S8, but there were no embedded sand layers at Point S9. The vane tests have been stopped at 25 metres' depth, but the CPT test and the sampling has been continued to almost 60 metres' depth. The corrected results from the CPT test and the direct simple shear tests agree well throughout the profile, except for the already mentioned discrepancy in one of the upper soil layers. The correction for overconsolidation ratio in the CPT tests has a significant effect in the upper soil layers and generally entails a better correspondence with the results from the direct simple shear tests.

The undrained shear strengths that were determined by active and passive triaxial tests in order to study the ***shear strength anisotropy*** agree fairly well with what can be expected from empirical experience. According to this, the undrained passive shear strength can be expressed in the same way as the shear strength at active shear and direct simple shear, i.e.

$$c_{u_{PASSIVE}} = a \cdot \sigma_v \cdot OCR^b$$

Where a for passive shear in clay can be expressed as

$$a \approx 0.05 + \frac{0.28 w_L}{1.17}$$

and b is about 0.8.

The values measured in these tests are included together with values evaluated from the empirical relations in Fig. 48. Considering the large variation in liquid limit when the whole profile is studied, the empirical evaluation of a values at direct simple shear, described in e.g. SGI Information No. 3, has also been used. The results of both the triaxial tests and the direct simple shear tests tend to be somewhat lower than the empirically evaluated values. However, the differences are moderate and the relations between the different modes of shear are as expected. The empirical relation for the shear strength anisotropy is thus applicable in this section too, even if a certain caution should be exercised concerning the absolute size of the shear strengths estimated by the empirical relations.

The effective shear strength parameters have been evaluated from the effective stress paths in the active undrained triaxial tests. The tests were performed on specimens from 13, 32 and 53 metres' depth below the ground surface. The results agree well with the assumption of a friction angle of 30° and the evaluated c' values were 4, 10 and 13 kPa for the different depths. The values evaluated with the empirical relations $c' \approx 0{,}1\tau_{fu}$ and $c' \approx 0{,}03\sigma'_c$ are 5–7.5, 8–12 and 11–16 kPa respectively for these levels. In this case too, the empirical relations yielded values of an appropriate size.

Point S11 is located just behind the crest of the upper slope. Apart from the usual field tests, a dilatometer test has also been performed near this point. The pattern with higher strength values from field vane tests and fall-cone tests in the clayey silt was found once again, in this case down to a depth of 14 metres, Fig. 49. Below this depth the results coincide fairly well, except for the usual lower values from the fall-cone tests below a certain depth. In the upper soil layers, the dilatometer tests evaluated according to SGI Information No. 10 yield significantly lower values. However, it is well known that this evaluation method elaborated for clay is not valid for silt where it gives too low values. At larger depths, higher values are generally obtained by the alternative method of evaluation even if the trend for development of shear strength with depth is similar. The soil below the upper sand and silt layers is here classified as normally consolidated or only slightly overconsolidated throughout the profile, and the correction of the CPT test results has only a marginal effect.

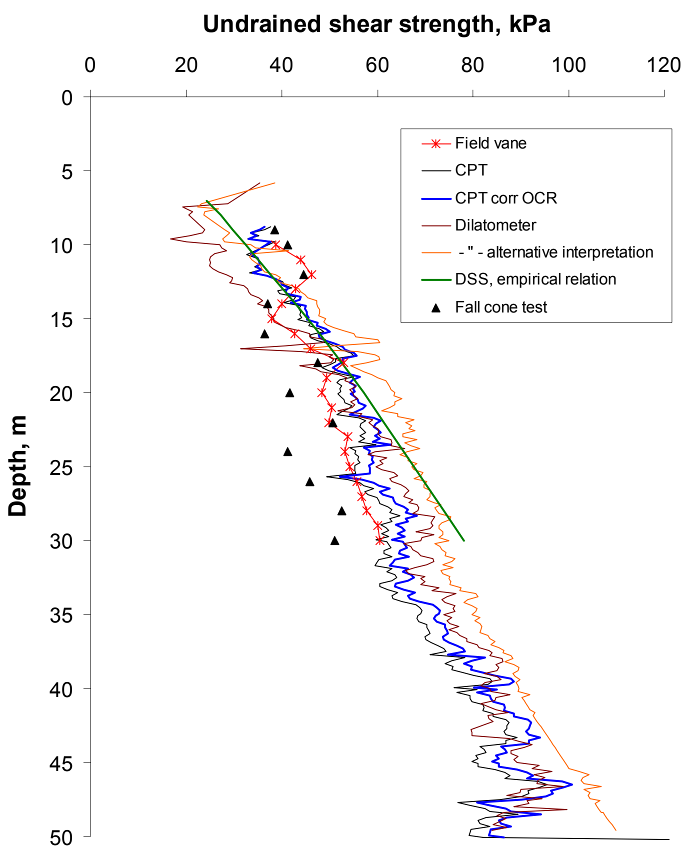


Fig. 49. **Shear strength determinations at Point S11 behind the upper crest in Section C.**

A compilation of the results of the field vane tests in the different test points in Section C shows that the shear strength is very similar at the same level in the upper part of the slope, Fig. 50. On the other hand, the effect of the unloading because of the erosion is obvious at the toe of the slope below the river where the shear strength is considerably lower. It is more difficult to trace an effect of the excavations in this compilation. This may to some extent be due to the fact that the upper layers, in which an effect could be expected, consist mainly of clayey silt with infusions of sand. The shear strength values from the field vane scattered in this layer and did not follow the normal pattern for clay soils. A certain effect, which reaches down to a maximum depth of 7 metres (level +3.5 metre), may possibly be seen in the values from Point S8. The shear strength values from the field vane tests in this point

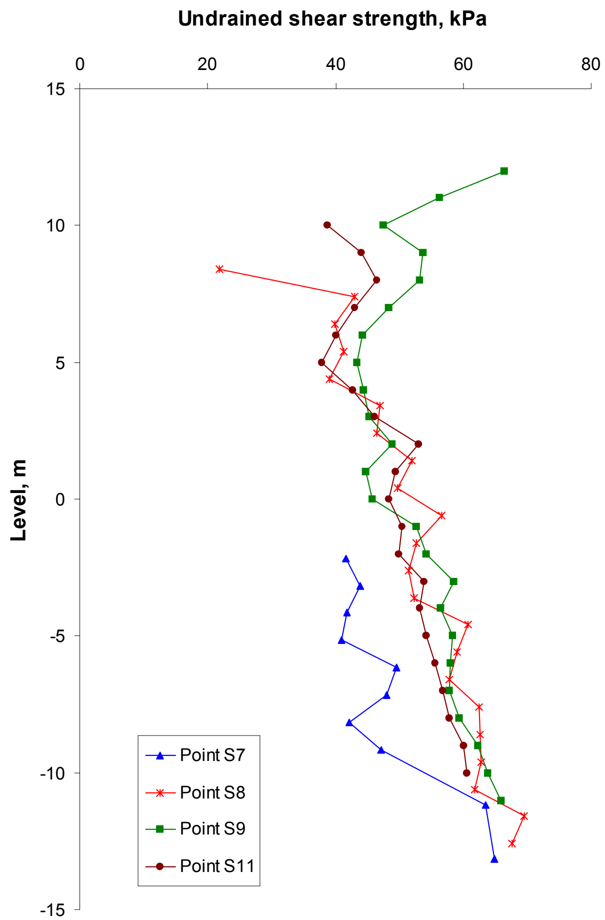


Fig. 50. **Compilation of shear strength values from the field vane tests in Section C.**

are also close to what would be expected from empirical effects of unloading. However, even if the effects are obvious below the river, they are still considerably smaller than would be expected from the general soil models.

A corresponding compilation of the shear strength values from the CPT tests also shows that the undrained shear strength properties are similar throughout the slope, Fig. 51. Without correction for the overconsolidation ratio, there is a certain spread in the results between the test points, but not in any particular pattern. When the different curves for shear strength evaluated without correction for overconsolidation versus level are studied, a certain effect of the unloading can be seen in the test below the river. However, according to these results the effect should only reach

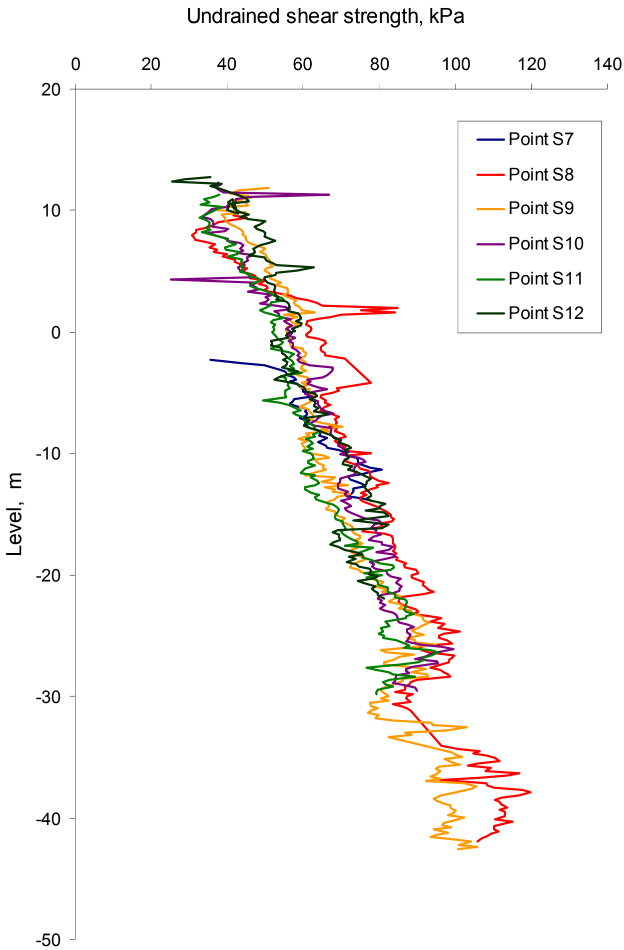


Fig. 51. **Compilation of evaluated shear strengths from CPT tests in Section C. a) without correction for overconsolidation ratio**

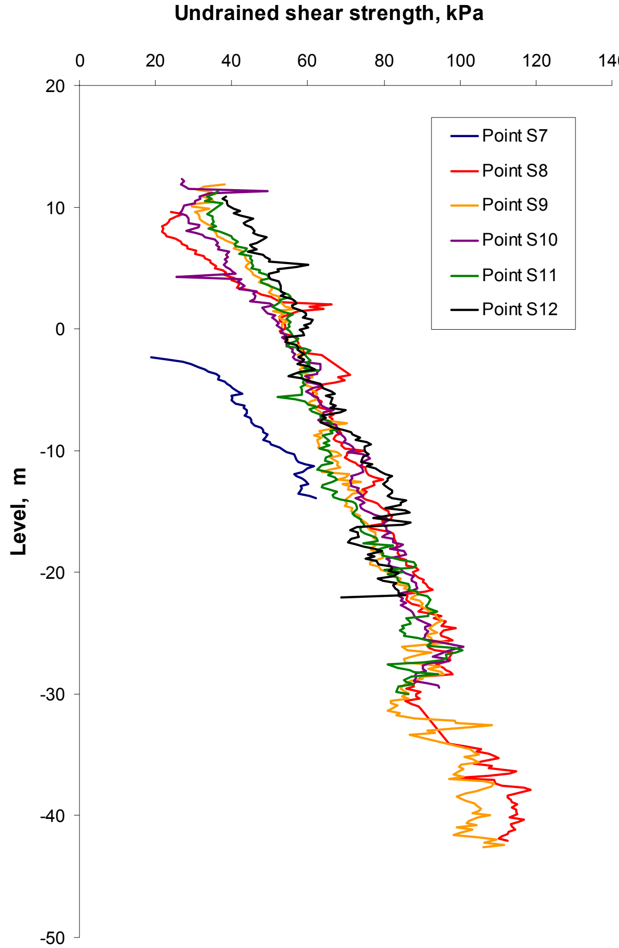


Fig. 51. Compilation of evaluated shear strengths from CPT tests in Section C. b) with correction for overconsolidation ratio

some metre below the river bottom. After correction of the results, the expected pattern appears with a considerable reduction of the shear strength at Point S7 below the river and reductions also in the upper soil layers below the excavated terraces at Points S8, S9 and S10. The results at Point S11 directly behind the upper crest may likewise be marginally affected by the unloading.

The results show that in this type of clay an unloading has only a moderate effect on the undrained shear strength measured by field vane tests. The effect is almost non-existent for the shear strengths evaluated from CPT tests unless these are corrected for overconsolidation ratio.

2.8 CHANGES IN SHEAR STRENGTH

An attempt was made to trace possible changes in the undrained shear strength because of the excavations by performing new field vane tests in those points where such tests had been performed in the investigations prior to the stabilisation works. This was done at Points 2 and 3 in Section A, Point 9 in Section C and Point 22 in the northern part of the area.

Point S2 is located on the excavated terrace in Section A where about 5.5 metres of soil have been removed. A comparison between the tests before and after the excavation shows a certain decrease in shear strength down to a depth of about 5 metres below the present ground level, Fig. 52. The values are then about equal at greater depths. Below the level 0 metres, the values from the previous investigations are considerably lower. This is probably a result of a change to a smaller vane size. When the values are corrected for the effect of a such change, which was measured in the special project concerning these effects, the values from the two investigations become about equal also below this level.

The corresponding comparison at Point 3 behind the upper crest in the same section shows very small changes, Fig. 53. The possible changes in this point are in the opposite way and the measured shear strength rather tends to increase after the performed actions. This is not quite impossible since the groundwater level has been somewhat lowered at this point, resulting in an increased effective overburden pressure, but the possible effect should be marginal.

An additional comparison was made in Section A. Field vane tests had previously been performed at the riverbank close to Point S1. In the new investigation, field vane tests were performed below the river bottom where another about 2 metres of erosion has occurred. The comparison shows marginally higher strength values below the riverbank, Fig. 54. However, the tests below the river bottom have been performed from a floating raft. The test levels are therefore not quite exact and a minor adjustment of the curves level-wise would bring them to almost coincide.

Point S22 in the northern part of the area is compatible with Point S2 since it is located on the excavated terrace and the thickness of the excavated soil layer was about the same. At this point, SGI had performed investigations before the excavation including field vane tests. Here, almost identical results were obtained in the new field vane tests, Fig. 55.

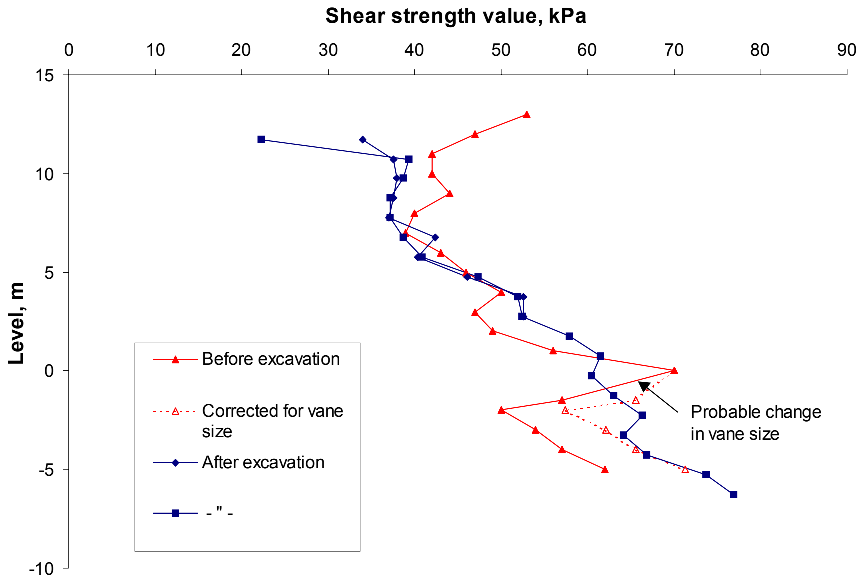


Fig. 52. **Comparison between shear strength values measured by field vane tests before and after excavation at Point 2 in Section A.**

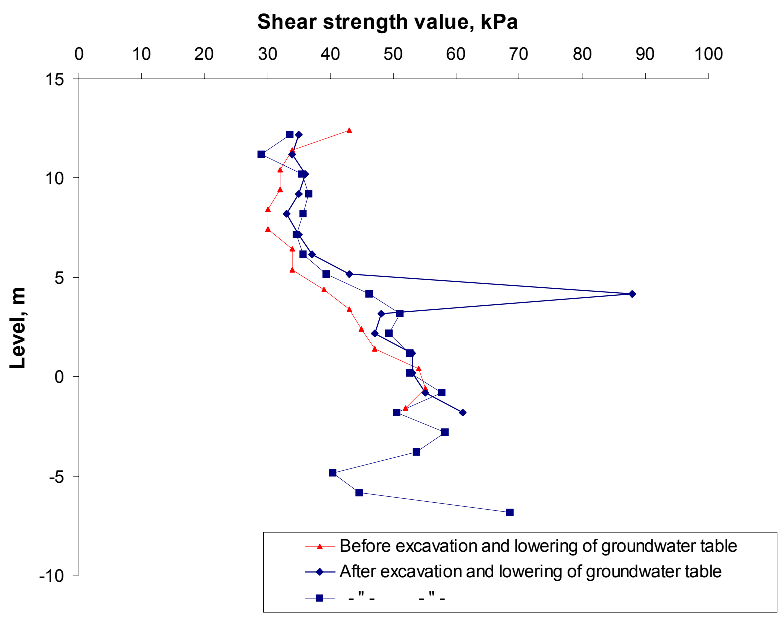


Fig. 53. **Comparison between shear strength values measured by field vane tests before and after excavation at Point 3 in Section A.**

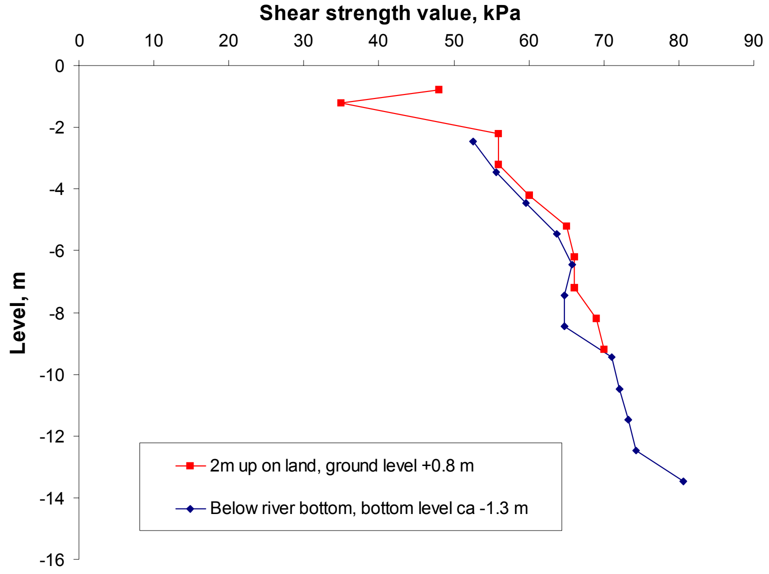


Fig. 54. Comparison between shear strength values measured by field vane tests below the riverbank and the river bottom in Section A.

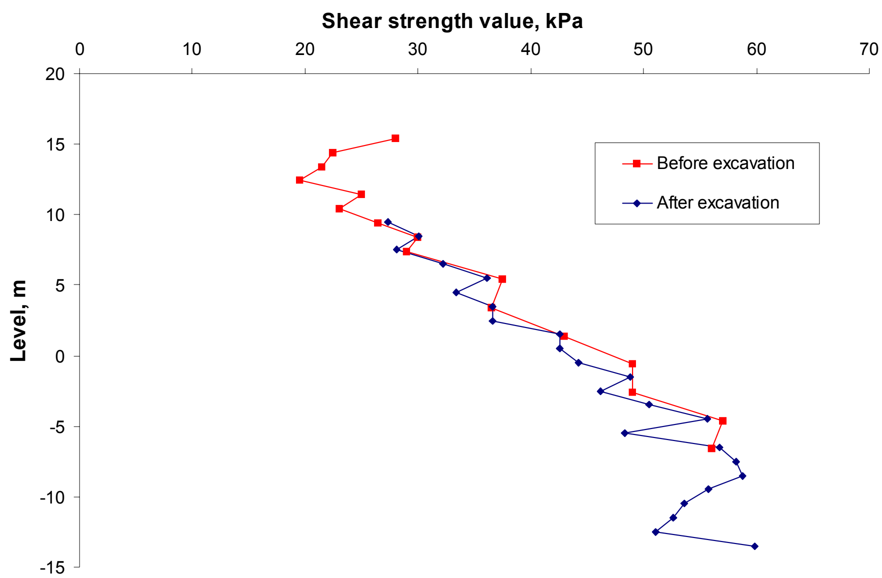


Fig. 55. Comparison between shear strength values measured by field vane tests before and after excavation at Point 22 in the northern part of the area.

Point S9 in Section C is located on the upper excavated terrace where about 6.5 metres of soil were excavated. A draining base layer was then laid out and covered and the resulting difference in level before and after is about 6 metres. A clear difference in shear strength values between the tests before and after the stabilisation works is measured at this point, Fig. 56. The effect reaches down to about 6.5 metres below the present ground surface. The comparison is rendered somewhat difficult since the difference measured is located fully in the upper soil layer consisting of silty clay with infusions of sand and organic matter.

These comparative field vane tests do not show without doubt that a significant change in the shear strength has occurred as a result of the excavation. There are a number of indications that this has happened, but these are relatively weak and subjected to possible sources of error. At one point, there were no detectable changes at all. The uncertainty is partly due to the fact that field vane tests had only been made in single boreholes at each point in the previous investigations and no comparison of mean values and standard deviations can be made.

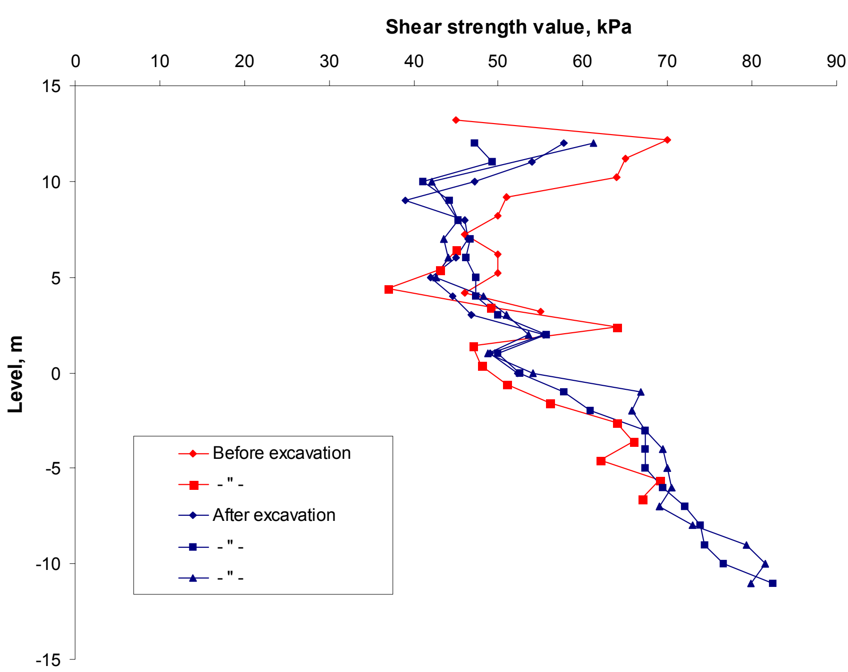


Fig. 56. **Comparison between shear strength values measured by field vane tests before and after excavation at Point 9 in Section C.**

The excavations were fairly large, but the depth interval within which a measurable effect can be expected is still limited, among other things because of the adjoining lowering of the groundwater level. The soil also consists mainly of clayey silt and very silty clay with different kinds of infusions on those levels where a significant change ought to have occurred, which makes the interpretation more difficult. However, together with all other comparisons between measured shear strengths at different points in the slope and with different methods, it can be considered established that a reduction in shear strength occurs when the soil is unloaded, particularly in superficial layers. The size of the measured reduction depends on what test method and what interpretation is used. The reason why different shear strengths are measured by different methods, particularly in the lower parts of slopes, is being investigated at present in a separate joint research project between the Swedish Rescue Services Agency, Chalmers University of Technology and SGI (Löfroth 2002).

2.9 STABILITY CALCULATIONS

2.9.1 Previous calculations

The stability calculations before the excavations had resulted in minimum safety factors varying between 0.86 and 0.78 for Sections A and C respectively. The calculations were made with assumptions of drained shear strength in the upper sand layers and undrained shear strength in the silt and clay below. The critical slip surfaces calculated in this way started 10 to 15 metres behind the crest and ended at the toe of the slope. Almost all soil masses involved were located above the river bottom. There were therefore no large passive zones, but the major part of the slip surfaces were located in relatively steep active zones. The assumption that the calculated safety factor would be about 1.0 if the shear strength anisotropy was taken into account is thus reasonable.

Two alternatives were given for the stabilising measures, one for a 30 % increase in calculated safety factor and one for a 50 % increase. In both cases, the calculations were made with the same assumptions of drained shear strength in the upper sand layer and isotropic shear strength in the silt and clay. No possible reduction in shear strength because of the unloading was considered.

In Section A, the first alternative required an excavation down to the level +15 metres, i.e. a lowering of the ground surface by 3–4 metres, within a distance of 30 metres from the crest. The second alternative required an excavation down to the level +11 metres within 50 metres of the crest. In Section C, where the excavation was to be made in two steps, the first alternative entailed excavations down to the levels +11 and +15 metres for the two terraces. These levels had to be lowered to +9.5 and +13 metres in the second alternative. The real excavations fell somewhere between these two alternatives except in Section B, located about midway between the investigated sections. The operation of the then existing cement works entailed that the stabilising measures here were limited to a 30% increase in stability. It was then considered that the industry was only in operation during the daytime (Bergdahl 2002).

The calculated safety factors after the final design of the excavations using the same assumptions as before and without considering shear strength anisotropy were about 1.3. This value was found both for slip surfaces with the same form and location as the previously calculated slip surfaces and for deeper slip surfaces, which reached behind the new upper crest and thus involved larger soil masses. The latter slip surfaces were restricted in all sections by the assumed firm bottom.

2.9.2 New calculations

New calculations have been performed in order to study the influences of the new and more comprehensive shear strength determinations, the shear strength anisotropy in real calculations, the use of combined analyses in which parts of the silt and clay layers can also behave in a drained manner and the real contour of the firm bottom being different from what was previously assumed. The calculations have been made using both a "self-seeking" programme for circular sip surfaces and Janbu's general procedure of slices for slip surfaces of arbitrary shapes (Janbu 1954).

The first type of calculation has been made with the computer program SLOPE/W, (Geo-Slope International 1994) using Spencer's method for a rigorous analysis (Spencer 1967). The shear strength in the sand and the coarser silt has been assumed as drained with a friction angle of 32°. These calculations have been made using both combined and undrained analyses and with the assumption of isotropic shear strength in the clay. The shear strength in the silt has mainly been modelled for a combined analysis, but in some calculations it has been restricted to undrained shear strength in order to facilitate studies of possible critical slip surfaces located at greater depths.

Approximately the same critical slip surface and safety factor as in the previous calculations were obtained in the new calculations using isotropic undrained shear strength for the conditions before the excavation in Section A, Fig. 57.

Insertion of a roughly modelled anisotropic shear strength increased the calculated safety factor to somewhat over 1.0. On the other hand, a control calculation with combined analyses showed that the critical clip surface moved out towards the slope and became shallower and that the drained shear strength in general became dimensioning. The calculated safety factor thereby became very close to 1.0 again, Fig. 58.

Control calculations for deeper slip surfaces showed that the most critical slip surfaces involving large soil volumes went deeper than the previously assumed contour of the firm bottom.

The possibility of a detailed modelling of the soil properties is generally limited in the "self-seeking" programmes. Supplementary calculations were therefore performed with Janbu's general procedure of slices in which the modelling of soil properties and shapes of slip surfaces is unrestricted. The search for the critical slip surfaces was started by the former calculations. The shape of the slip surface was then modified to follow possible weaker planes and layers and to be more kinematically correct considering a drained failure mode at its ends.

The calculations for a superficial slip surface similar to the critical surface obtained in the SLOPE/W calculations also resulted in a safety factor of 1.0. The difference between the calculations was primarily that the shear strength anisotropy could be better modelled in the latter calculations. However, this was of little importance since the drained shear strength was lower and governing along most of this slip surface.

A larger slip surface involving a large soil volume was also modelled. This slip surface was given a shape that started as an active failure zone in its upper part. It then connected to a straight part following the contour of the firm bottom but located a couple of meters up in the clay. About midway under the steep lower part of the slope it transformed to a passive failure zone ending up below the river. It is fairly common that a lower shear strength is measured in the transition zone between clay and firmer soil layers below. This was also the case in this section. Whether this reflects a real weaker zone or is a result of an increased disturbance in the normally silty and layered transition zones is difficult to estimate, but in this way the possibility of such a weaker zone could be taken into account. The calculated safety factor for this large slip surface was 1.17 with a combined analysis and isotropic shear strength. It increased to 1.25 when the anisotropy of the undrained shear strength was considered.

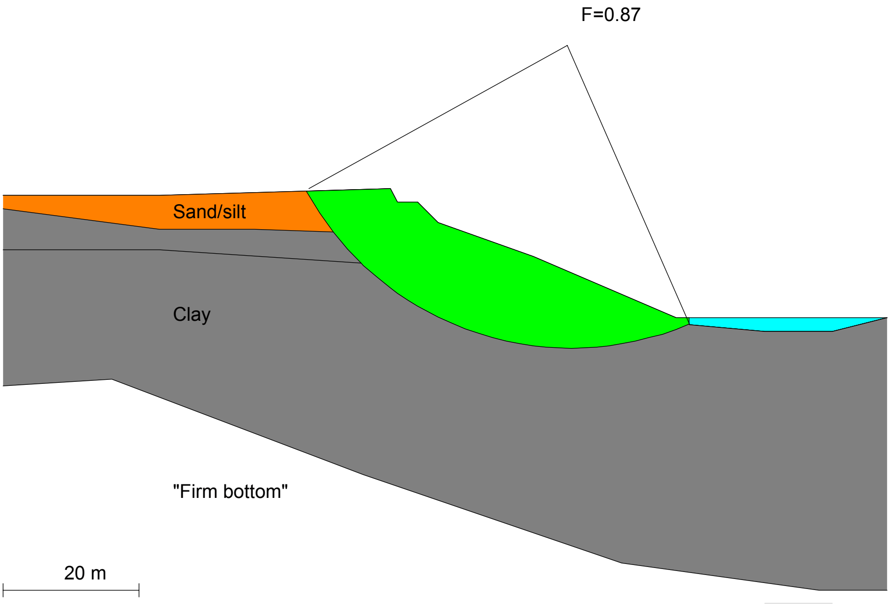


Fig. 57. Calculated critical slip surface in Section A before excavation using SLOPE/W, Spencer's method and undrained isotropic shear strength in the clay.

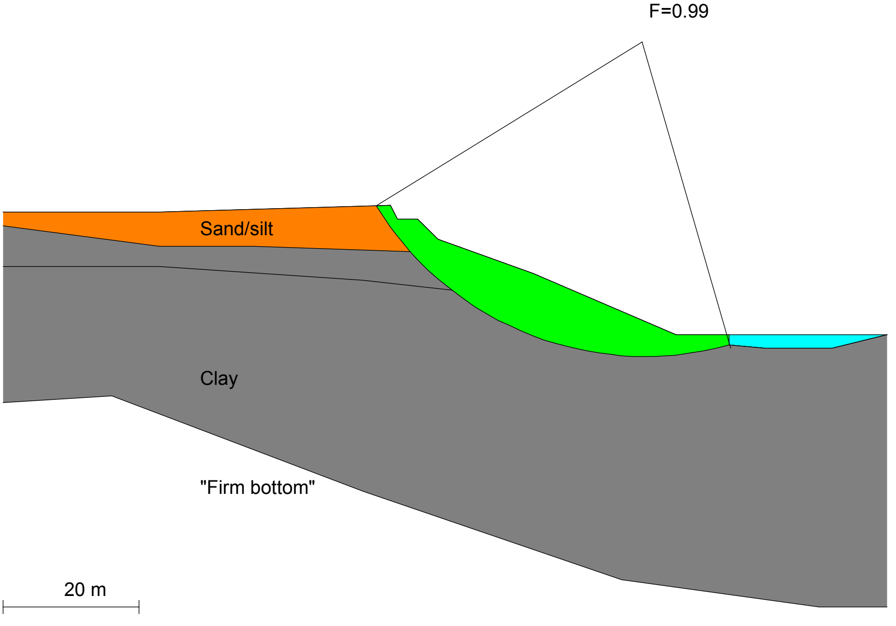


Fig. 58. Calculated critical slip surface in Section A before excavation using SLOPE/W, Spencer's method, combined analysis and anisotropic undrained shear strength in the clay.

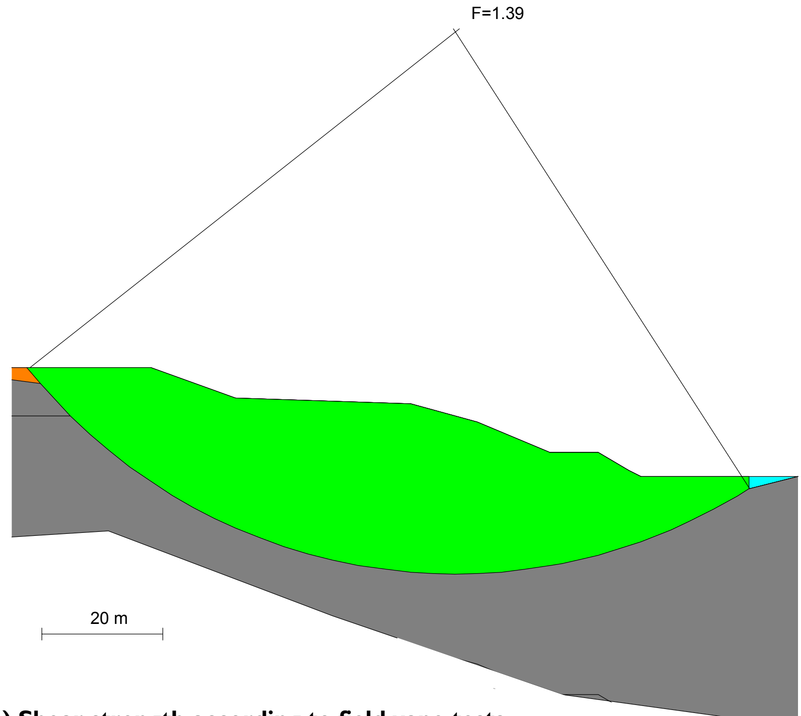
The safety factor in Section A before the stabilisation works can thus be estimated to have been close to 1.0 for relatively shallow slip surfaces close to the slope and gradually have increased to about 1.25 for very large slip surfaces reaching more than 50 metres behind the crest at that time and down to the transition zone between clay and firm coarse soil on top of the bedrock.

Calculations for the conditions after the excavation show that the safety factors have increased, but not in all respects as much as intended. The first calculations using SLOPE/W and isotropic undrained shear strength resulted in a lowest calculated safety factor around 1.4. The location of this calculated critical slip surface depends to some extent on what is assumed about the distribution of the shear strength. The field vane tests showed somewhat higher shear strength in superficial layers but lower shear strength increase with depth than the other methods. Use of the shear strength values from the field vane tests thereby resulted in a calculated critical slip surface reaching far behind the crest and in principle comprising the whole soil volume down to firm bottom. Use of the shear strength obtained by the other methods resulted in a slip surface comprising about half of the excavated area and reaching a limited depth, Fig. 59.

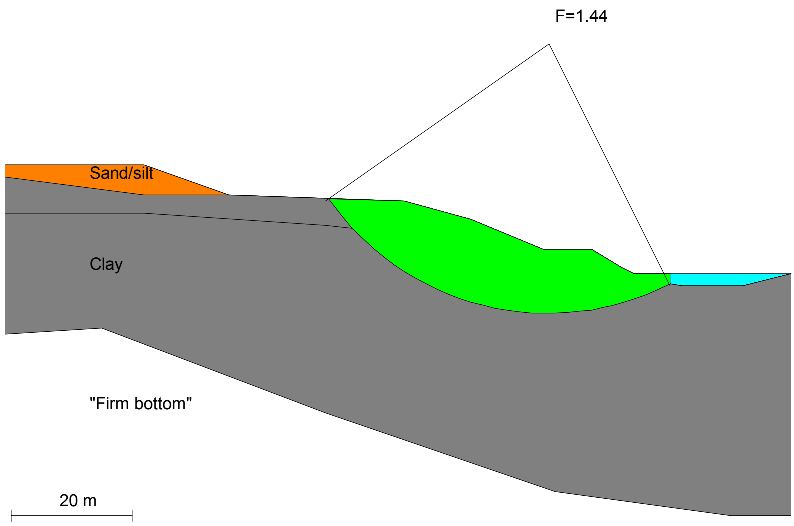
The corresponding calculations with combined analyses resulted in shallower slip surfaces with safety factors of about 1.1 (1.06 to 1.12 depending on the isotropic undrained shear strength determination that was used), Fig. 60.

The succeeding calculations using the general procedure of slices and anisotropic undrained shear strength increased the calculated safety for a large slip surface reaching behind the excavated area to 1.5. The calculated safety factor with combined analyses for slip surfaces starting on the excavated area was increased to 1.16 when the anisotropy was taken into account. At the same time, the critical slip surface was moved somewhat further out towards the slope and thus become shallower and the drained shear strength had become governing to a higher degree than when isotropic shear strength was used.

The excavation has thus resulted in an increased calculated safety factor for large slip surfaces up to about 1.5. However, the safety factor decreases gradually for smaller slip surfaces and is only about 1.16 for relatively shallow slip surfaces, although these may involve about one third of the excavated area. In these more superficial slip surfaces, the drained shear strength is largely governing and the increase in safely factor is primarily related to the decrease in average slope inclination resulting from removal of the very steep upper part of the previous slope and the construction of the erosion protection at the toe of the slope.



a) Shear strength according to field vane tests



b) Shear strength according to other test methods (primarily direct simple shear tests).

Fig. 59. **Calculated critical slip surface in Section A after excavation according to undrained analysis with isotropic strength (SLOPE/W, Spencer's method).**

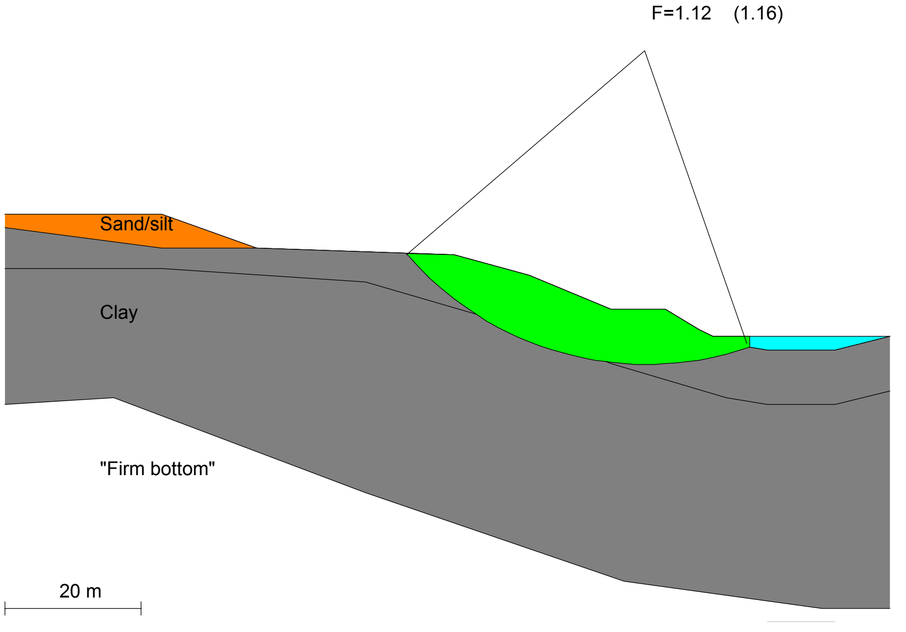


Fig. 60. **Calculated critical slip surface in Section A after excavation according to combined analysis and isotropic shear strength (SLOPE/W, Spencer's method).**
(Values in brackets refer to safety factors calculated with combined analyses, anisotropic undrained shear strength and Janbu's general procedure of slices.)

The assessment of the stability conditions is influenced by the fact that the required safety factor for combined analyses is lower than for undrained analyses. It is also influenced by the fact that if a slide occurs in a relatively small and shallow slip surface and the masses end up in the river and are transported away by this, then a large part of the stabilising forces for larger slip surfaces is also taken away. The safety of these larger slip surfaces is thus not necessarily greater than that of the shallow slip surfaces.

In Section C, the lower slope towards the river was very steep, with an average inclination of just over 30°. The erosion process had also created a small cavity at the toe of the slope. At this state, the superficial part of the slope was barely stable according to a drained analysis. Very small increases in the assumed pore pressures would lower the calculated safety factor to values below 1.0, and it may be assumed that effects of the vegetation in the slope had contributed and enabled the slope to stand in this way.

An undrained analysis results in a critical slip surface with approximately the same shape and location as in the previous section. It thus starts 10–15 metres in behind the crest and ends at the toe of the slope. Without consideration for the shear strength anisotropy it becomes about 0.81, which is very close to the previous estimate made before the excavation, Fig. 61. Control calculations with the general procedure of slices using anisotropic shear strength and combined analyses show that the safety factor is close to 1.0 for all of the outer part of the slope and that the drained shear strength is governing for all parts of the shallow slip surfaces.

The calculated safety factors increased for larger slip surfaces. For very large such surfaces reaching 20 metres behind the area involved in the subsequent excavation, the calculated safety factor taking anisotropy into account becomes about 1.2. This slip surface reached a depth of about 40 metres below the original ground surface. A control of even larger slip surfaces involving parts of the carpentry factory and reaching down to the coarser draining layer, where a possible weaker zone is indicated by the CPT test results, gives a safety factor of 1.26 increasing to 1.39 when the shear strength anisotropy is considered, Fig. 62.

The excavation and the other stabilising measures brought moderate changes in the outer slope. The erosion cavity at the toe of the slope was filled in and then covered by the erosion protection. A minor excavation of sand and silt was also performed above the steepest part, but the major part of the steep lower slope remained unchanged. The resulting increase in calculated safety factor according to a drained

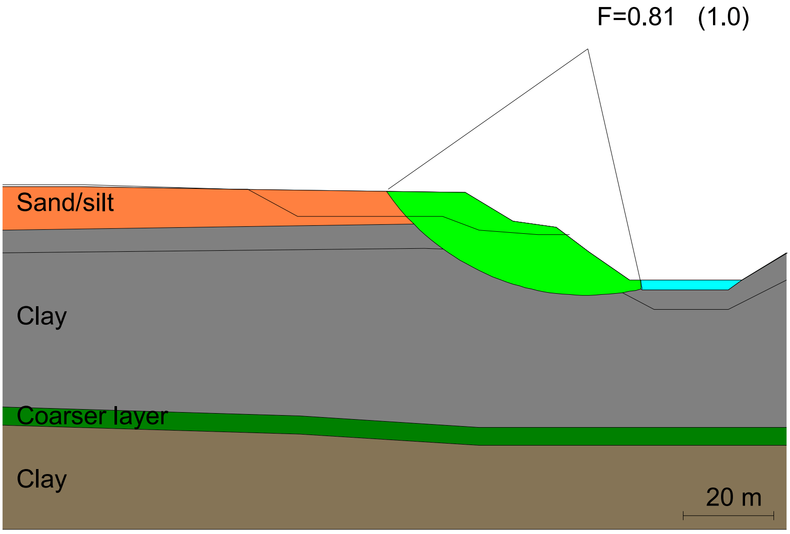


Fig. 61. **Calculated critical slip surface in Section C before the excavation according to undrained analysis with isotropic shear strength, SLOPE/W, Spencer's method.**
(Values in brackets refer to safety factors calculated with combined analyses, anisotropic undrained shear strength and Janbu's general procedure of slices.)

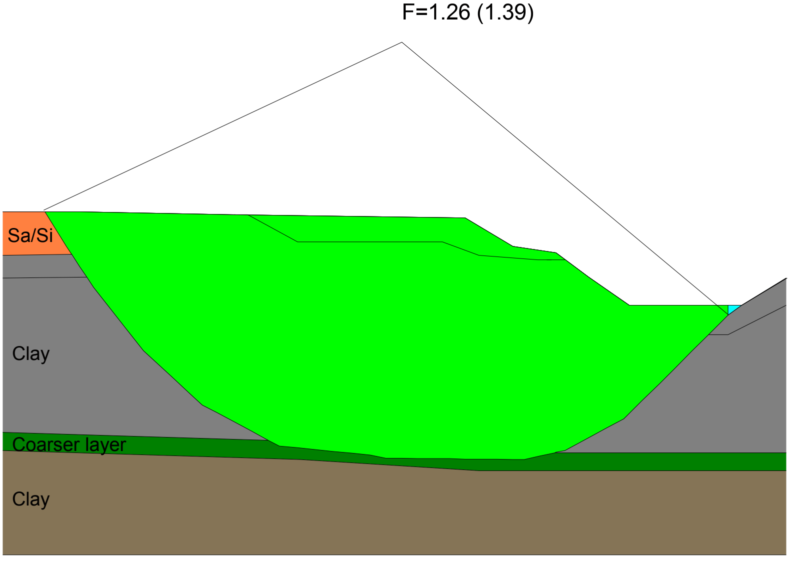


Fig. 62. **Calculated safety factor for a selected slip surface reaching down to the coarser soil layer at great depth. (Combined analysis, isotropic shear strength, SLOPE/W, Spencer's method.)**
(Values in brackets refer to safety factors calculated with combined analyses, anisotropic undrained shear strength and Janbu's general procedure of slices.)

analysis is only about 15 % for superficial slip surfaces, Fig. 63. However, for the critical slip surface according to undrained analysis before the excavation the calculated safety factor has increased much more. This slip surface now starts about 10 metres in on the upper excavated terrace and the calculated safety factor is 1.32.

The stabilising effect decreases for larger slip surfaces since the stabilising forces for these have also been reduced because of the excavation. It is thus found that the calculated safety factor for the large slip surface reaching 20 metres behind the upper crest has only increased to about 1.3, i.e. an increase by only about 8 %, and that this surface is now the critical slip surface according to undrained analysis. For the very large slip surface reaching down to the possible weaker zone, the calculated safety factor has been reduced from 1.39 to 1. 36, Fig. 63. The values given are based on control calculations using the general procedure of slices with modified slip surfaces and more detailed descriptions of the shear strength.

The effect of the excavation and the construction of the erosion protection in Section C has thus been that the calculated safety factor has increased to about 1.15 for shallow slip surfaces in which the drained shear strength is governing. This calculated safety factor is very sensitive to what is assumed about the pore pressure conditions in this part, which have not been examined in detail. For slip surfaces reaching further into the slope, the calculated safety factor fairly rapidly rises to about 1.3 and then remains about constant independent of the size of the slip surface. In these larger slip surfaces, the undrained shear strength in the clay governs the size of the calculated safety factor.

As in Section A, the stability of large slip surfaces is dependent on the outer masses in the slope remaining stable in their current state. If a slide occurred in the outer part and the soil masses were swept away from the toe of the slope, then the safety factor for the next slip surface in the slope would be lowered. If this part failed too, the stability for the next slip surface would be jeopardised, and so on. How far such a successive development could reach can only be speculated on. Fortunately, there is no quick clay right beside the possible initial slides, so a rapid and dramatic development is not plausible.

The excavations performed have entailed that in both sections the calculated safety factors have fulfilled the demands for what is classified as “uninhabited area with less important constructions”, which corresponds to the affected areas after demolition of the dwelling houses and closure of the cement works. The present and previous topography shows that previous slides along this part of the river have been relatively shallow, even if the deepest scar reaches about 30 metres in from

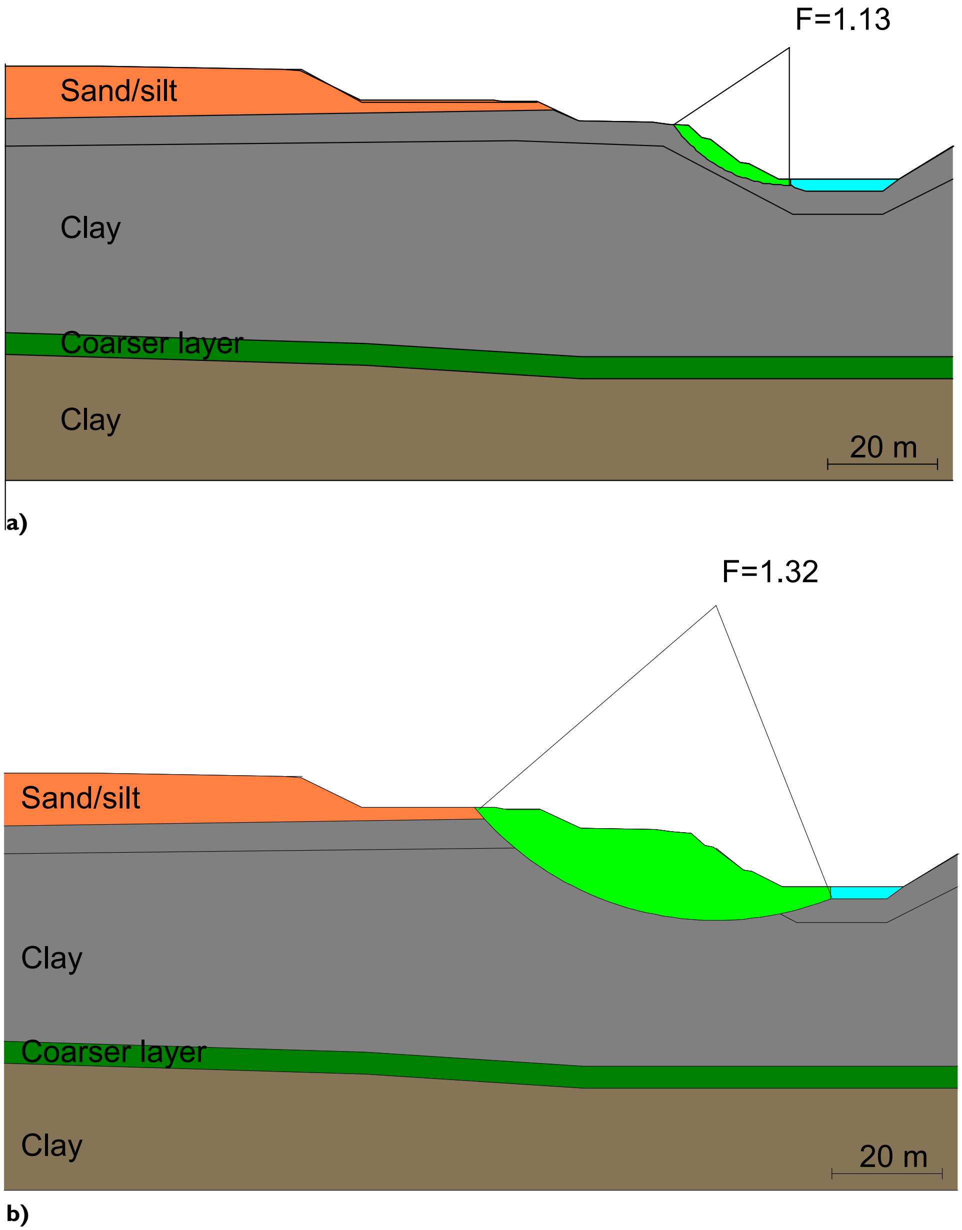
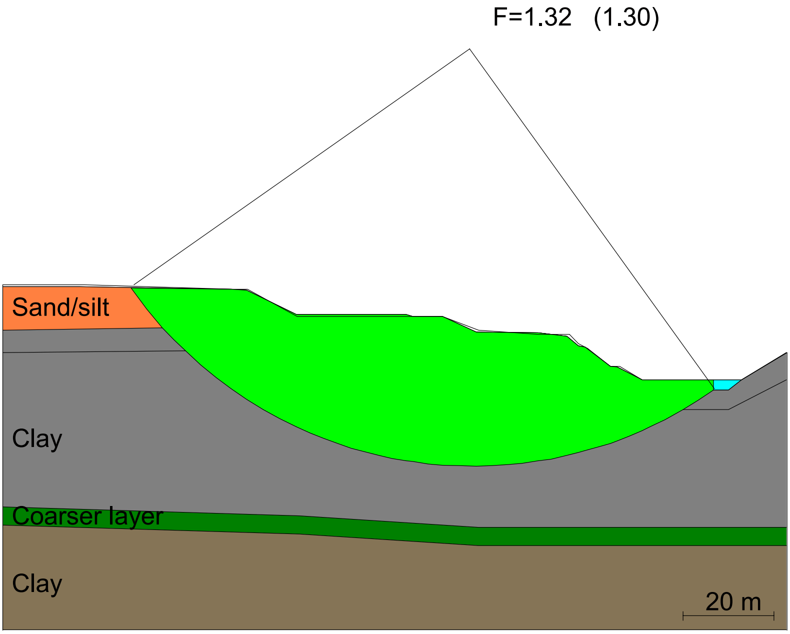
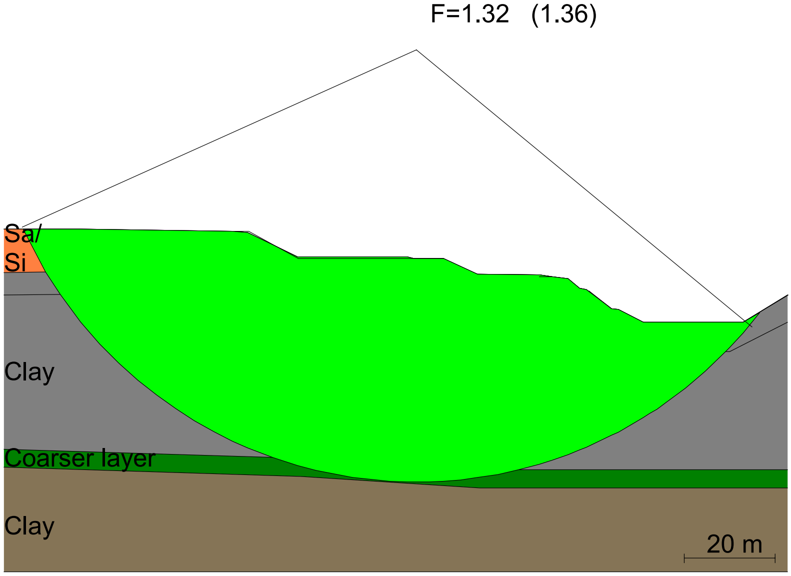


Fig. 63. **Calculated safety factors for different slip surfaces in Section C after the excavation, SLOPE/W, Spencer's method.**
a) superficial slip surface in the slope
b) critical slip surface in undrained analysis before the excavation



c) critical slip surface in undrained analysis after the excavation



d) slip surface reaching an assumed weaker zone in the silty soil at great depth

Fig. 63. Calculated safety factors for different slip surfaces in Section C after the excavation, SLOPE/W, Spencer's method. (Values in brackets refer to safety factors calculated with combined analyses, anisotropic undrained shear strength and Janbu's general procedure of slices.)

the previous crest. It is not known whether this slide has developed as a single slide or a number of retrogressive slides. In spite of its depth, there is still a fairly long distance between the backscarp of the largest slide and the area with quick clay. The probability of a development that would rapidly reach into quick clay and result in a major landslide is therefore small.

According to common rules and recommendations, the current buildings and activities in the area could remain provided that they are not extended and that no actions are taken which make the stability conditions worse. This was also the goal of the stabilising measures performed. The remaining insecurity is mainly related to the stability of the outer parts of the slope down towards the river. According to the calculations, this is low and very sensitive to assumptions about the pore pressure conditions. On the other hand, the stability conditions have been improved by the excavation and particularly by the construction of the erosion protection and the accompanying fill at the toe of the slope. Provided that no great changes occur, such as undermining and deterioration of the erosion protection or forest fire or other destruction of existing vegetation, the risk of a slide in the outer part of the slope should thereby be small.

Further considerations regarding the required safety factor and use of the area are outside the scope of this project.

Chapter 3.
Strandbacken, Lilla Edet

3.1 DESCRIPTION OF THE AREA

The Strandbacken area in the municipality of Lilla Edet is located on the western side of the river Göta-älv about 1 km south of the locks at Ström and the Lilla Edet hydroelectric power plant, Fig. 64. The buildings in the area today comprise about ten single-family houses located between the river and the main road between Lilla Edet and the city of Kungälv, which runs parallel to the river at a distance of about 250 metres. The area was previously built up with a larger number of houses located almost all the way out to the then existing crest of the steep slope towards the river. The old houses close to the river in the now excavated area were demolished in connection with the stabilisation works, and the closest building is now located about 100 metres from the river.

The built-up area on this side of the main road lies as an isolated enclave surrounded by farmland. However, new houses have recently been built on the other side of the road all the way up to the rocky hills behind. Erosion ravines created by small brooks running towards the river border the area to the north and to the south. There is also a small ravine at the centre of the area. This is partly created by human activity and has through the years, among other things, been used as an access road for the local fire brigade to fetch water from the river. On the riverbank there have also been constructions in the form of small landing-stages and piers. These have later been replaced by an erosion protection of rock-fill.

A first excavation of the slope crest was made in 1959 in order to counteract an estimated risk of imminent slope failure. This excavation had a limited size and reached only about 10 metres behind the crest at the most. No buildings were demolished at this stage, but the design of the excavation was adjusted in order to let them remain, Fig. 65. The then existing erosion protection was also extended further southwards. However, the calculated stability was still low for the whole area and the requirements for minimum safety factors that have been established later were far from fulfilled. After an investigation in 1988, it was therefore decided to ensure the stability of the area by a large excavation over a length of about 250

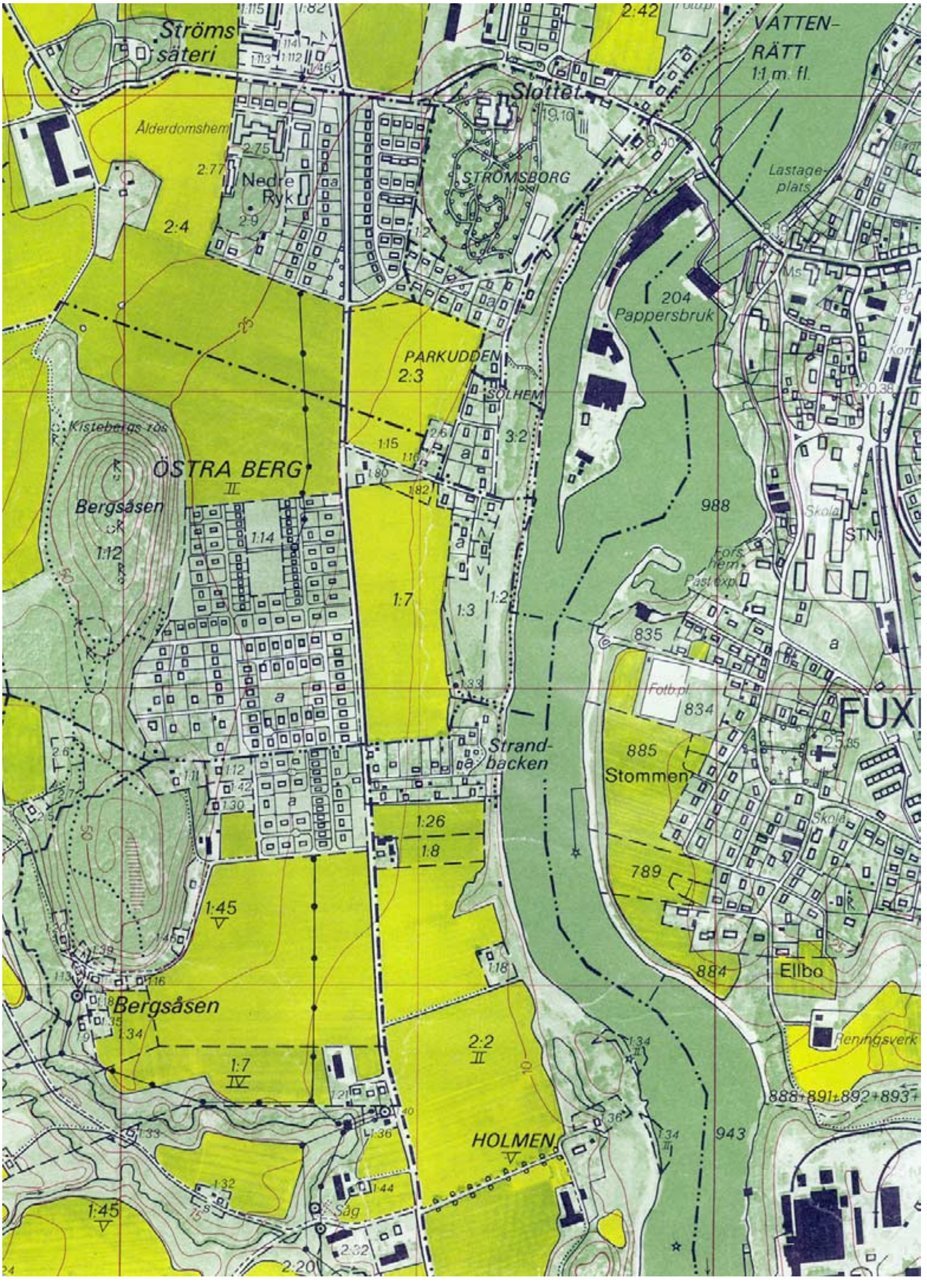


Fig. 64. **Strandbacken at Lilla Edet. Map from 1977.** Copyright Lantmäteriverket 2003. From The Property Map reference no. M2003/5268. Valid to 2007-09-30.

Fig. 65. Strandbacken after the first excavation.
a) Photo of the excavation from 1962 taken by Å. Paulsson.

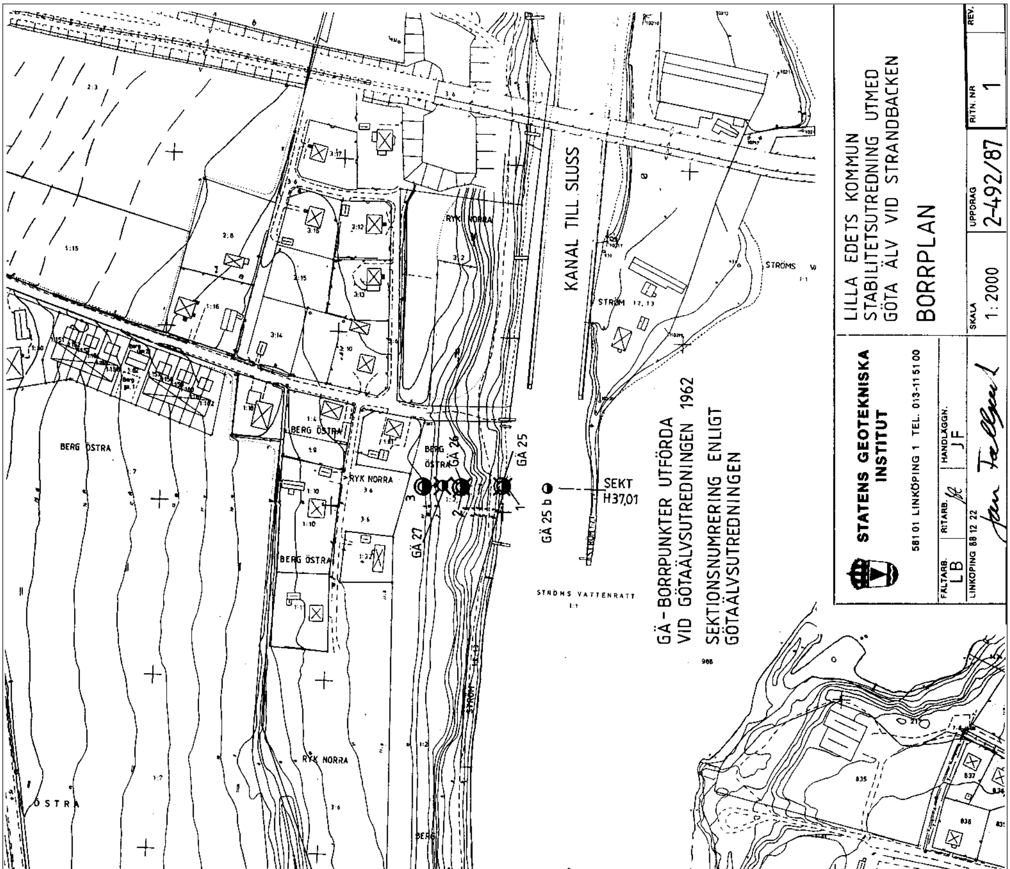


Fig. 65. **Strandbacken after the first excavation.**
b) Map of the area from the investigation by SGI in 1988.

metres, i.e. the whole distance between the surrounding ravines, reaching 50 metres behind the original crest. All buildings within this area were bought up and demolished. The excavated area in front of the remaining buildings has thereafter only been used for walks and airing of domestic dogs. The slope has been stable, but smaller slope failures have occurred in recent times both to the north and to the south of the excavated area. However, these slides have not affected any buildings or constructions.

3.2 GEOLOGY

A comprehensive investigation concerning the geology of the Göta-älv valley was made in connection with the large Göta-älv investigation (SOU 1962:48). Some modifications of the general geological description have been made thereafter, mainly concerning the detailed dating of different geological phases. The description given below is mainly taken from the Göta-älv investigation but has been modified according to the chronology given in the National Atlas of Sweden – Geology (Fredén 1994).

Thick deposits of clay started to be deposited in the Göta-älv valley in connection with the melting of the inland ice. The retreating ice front passed the Lilla Edet area about 12,000 years ago. Fine-grained particles in the meltwater then started to sediment directly on top of the bedrock or on a layer of till which had been deposited below the ice cover. The sea level in the area at that time was about 125 metres higher in relation to the bedrock than today, and only a few peaks protruded as rocky islets. When the ice had retreated further, the area at first constituted a branch of the sea connecting it to the basin of Vänern. Later on, when the ice front passed the province of Närke, the ice dam that until then had locked in the present Baltic Sea ceased to exist. The previously trapped water masses could then flow out into the sea together with the fine-grained soil particles they brought. Because of the land-elevation, the sound at Närke was again shut off about 9,000 years ago and the Vänern basin returned to be a bay. Since then, the area has successively become shallower and with time turned into dry land.

The clay layers in the area were deposited in seawater in which the salt content has varied during different periods, both with depth and with distance from the coastline. The glacial clay gradually becomes more fine-grained upwards in the soil profile. The late-glacial layers have contents of ironsulphide, which can occur as distinct black-coloured bands or as a blotchy sulphide colouring. In this clay, traces of earlier organisms that have lived in the mud on the sea bottom are also found. These traces are similar to worm holes. Thick deposits of post-glacial clay have been deposited on top of the glacial layers. At the end of the period during which the Lilla Edet area was submerged, a river had already been created in the area to the north. The river brought sand and silt particles that were deposited at the riveroutlet into the shallow water, which probably had the shape of a delta. In this way, the clay in the central parts of the valley was overlain by mixed layers of clay and silt, which gradually turned into silt and loose sand towards the ground surface. These layers contain a considerable amount of organic matter that mainly originates from reeds and other plants growing at the bottom of the shallow bay. The thickness of the upper sand and silt layers amounted to about 7 metres in the investigated section in the excavated area at Strandbacken. However, it varies considerably along and perpendicular to the river and rapidly becomes thinner behind the excavated area in the investigated section. The exact thickness is also difficult to determine because of the gradual transition to more fine-grained soil. Since the land rose above sea level, the river has eroded its channel down through the soil layers. The location of the channel is probably governed by the more easily eroded sand and silt layers. Also the locations of the brook ravines to the north and south of the area may be governed by local variations in the composition of the upper soil layers.

Anomalies and variations in the soil layering are common in the Göta-älv valley. The retreat of the ice front was not a continuous process; several momentary stops and retreats occurred. The latter sometimes entailed that coarser layers were created within the clay deposits (Stevens 1987). Such layers have been found at several locations in and around the Göta-älv valley, and a probable layer of this kind is also found in the Strandbacken area at a depth of 36 metres below the original ground surface. Both during the periods of deposition and in later times, a large number of slides have occurred, which have created remoulding and irregular stratification. There is also a certain variation in grain size distribution within the separate soil layers between the soil near the valley sides and that in the central parts of the valley. The borders between the different layers are generally somewhat inclined. This is partly attributed to the fact that, even if the borders have once been horizontal, the subsequent consolidation has meant that the total compression has been largest in the parts where the clay layers are thickest. The borders between the layers have then become inclined in a manner that reflects the underlying contour of the bedrock of firm bottom layers. The location of the thickest layers does not necessarily coincide with the location of the river channel.

The sediments in the Göta-älv valley are mainly deposited in seawater. During the time thereafter a considerable leaching of the salt in the pore water has occurred, particularly in the northern part of the valley. This has had the result that the liquid limit of the clay, and sometimes also its remoulded shear strength, has been considerably reduced in many places and quick clay is often found. The clay deposits to the north of Lilla Edet are generally completely leached. South of the central parts of the municipality, the degree of leaching varies across the valley in such way that the clay close to the valley sides in general is more leached than the clay at the centre of the valley. The Strandbacken area is located close to this border. Only about 700 metres to the north there are profiles that have been classified as completely leached, whereas considerable salt contents remain in the vicinity of the river at Strandbacken. In this area, it is mainly superficial layers and layers close to draining layers that have been substantially leached. The degree of leaching increases further away towards the valley side and becomes almost total at a distance of about 300 metres from the river. The clays at Lilla Edet in sections some distance to the north of Strandbacken and at the same and larger distances from the river are known as extreme quick clays and have been used for research and demonstration purposes on several occasions.

The natural ground level at Strandbacken is today located about 10 metres above the mean water level in the river. Within the river channel, the erosion has entailed that the water depth gradually increases to about 5 metres over a varying distance from the riverbank. The depth then rapidly increases to about 10 metres in the deep navigable channel, which is the fairway between Lake Vänern and the sea. The shallower inclined terrace is triangular in shape and reaches almost out to the middle of the river (about 50 metres), in the southern part but only about 10 metres in the northern part where the fairway leads in towards the lock on the western side of the river.

The erosion conditions have been altered during the last century because of the construction of the Lilla Edet hydroelectric power plant and the lock at Ström and also because of the increased ship traffic. The raised water level above the lock and the power plant has meant increased erosion at the riverbanks, which has been remedied by erosion protection structures. On the other hand, the bottom erosion has practically ceased in these parts. The water level south of the lock has not been greatly affected. Erosion protection has been constructed along the riverbanks and no significant shore erosion is going on, except possibly in the side ravines. How the increased ship traffic and the forceful and altered water streams from the outlet of the power plant have affected the bottom erosion is more uncertain. The river appears to have eroded through the clay layers from the lock down to a distance of about 600 metres to the north of Strandbacken. The bottom here consists of firm till. South of this point, the bottom consists of medium-stiff, very fine-grained clay, which by common criteria should be considered as very resistant against erosion. In earlier times, there was continuous erosion, which created a natural compensation for the ongoing land-elevation, but the latter has practically ceased in the area. Whether any significant bottom erosion has to be anticipated is thus uncertain, and no regular inspection has been performed, except for the regular check that the water depth in the fairway is sufficient for the ship traffic.

3.3 THE GÖTA-ÄLV COMMITTEE

The low stability at Strandbacken was observed already in 1951, when SGI investigated the stability of the area along the western side of Göta-älv south of the lock at Ström on the commission of the Inland paper-mill. The stability was then found to be low all along the investigated 1 500 metre long distance and an excavation at the slope crest was recommended. This was considered to be particularly important at Strandbacken, where there were buildings located near the crest. The question was circulated between different departments and authorities, but none of these was willing to cover the costs.

After the major landslides at Surte and Göta, the Göta-älv Committee was commissioned by the Swedish government in 1958 to investigate the stability conditions along the Göta-älv valley (SOU 1962:48) and the question was brought up again. This new investigation found that the previous assessment of low stability in the Strandbacken area and the recommendation of an immediate excavation at the crest remained valid. The question once again went through protracted and difficult negotiations between different authorities until the situation became acute in February 1959, when cracks indicating an imminent failure were observed. The speed of the negotiations rapidly increased until the government (the King in Council), in May decided that the stabilisation work should be carried out and paid for by the Crown. This was then done during the summer and early autumn. This excavation only comprised a fairly narrow strip along the crest and the existing buildings could thereby remain. The extent of the measure was enough to remedy the risk of imminent failure but not the long-term stability problems.

The Göta-älv investigation showed that the stability was very unsatisfactory also in other parts of the area south of the lock at Ström and recommendations were made for excavations in other locations here. These measures were not considered acute in the same way and their execution was delayed. New investigations have then been made in different contexts, mainly in connection with development and construction in the area. These have given the same results and excavations have successively been performed in other parts along the river. In 1988, SGI was commissioned by the municipality of Lilla Edet to reinvestigate the Strandbacken area. The stability was once again found to be clearly unsatisfactory (F_c = 1.0–1.1), and a combination of a fill at the toe of the slope in the river and an enlarged excavation at the crest was recommended. The buildings near the crest of the slope were evacuated directly after the stability calculations had been performed. The Vattenfall Company was opposed to a fill in the river since this would be made a short distance downstream from the Lilla Edet hydroelectric power plant and affect the power production. The stabilising measures were therefore confined to an excavation at the crest of the slope, which became all the more extensive. An excavation with a lowering of the ground surface by about 4 metres over a width of about 50 metres along a distance of over 200 metres was therefore performed during 1990–1991 after the evacuated buildings had been bought up and demolished. The stated dimensions of the excavation include the first excavation performed in 1959.

3.4 PREVIOUS INVESTIGATIONS

The work by the Göta-älv Committee was very comprehensive, and no less than 18 sections were investigated within the distance from the lock at Ström to the end of the Strandbacken area alone, Fig. 66. The investigations were made with the equipment available and according to the practice at that time. Some new equipment was also introduced and tested during the course of the investigation. The investigations comprised field vane tests, iskymeter tests, A-sounding tests, weight sounding tests, machine sounding tests, soundings with the salt probe and sampling and laboratory tests. Undisturbed samples were taken by piston sampler and the Swedish sampler for continuous cores encased in metal foils. The pore water pressure was also measured at a number of points. The investigations were performed in sections with boreholes distributed from locations out in the river to far up towards the side or the valley.

Many of the investigation methods employed have later been abandoned and interpretations of their results are uncertain. Of the field test methods, only the field vane test and the weight sounding test remain unaltered. Sampling is performed by a piston sampler today as well, but it is uncertain whether the same type of sampler was used in all or part of the Göta-älv investigation. This investigation started in 1957 and ended in 1961, and the piston sampler used in Sweden today did not become standard until 1961.

The reliability of the results of the laboratory tests thereby varies depending on how sensitive the measured property is to the sample quality. The testing routines and interpretation methods were also partly different from those employed today. The correction factors for the undrained shear strengths determined by field vane tests

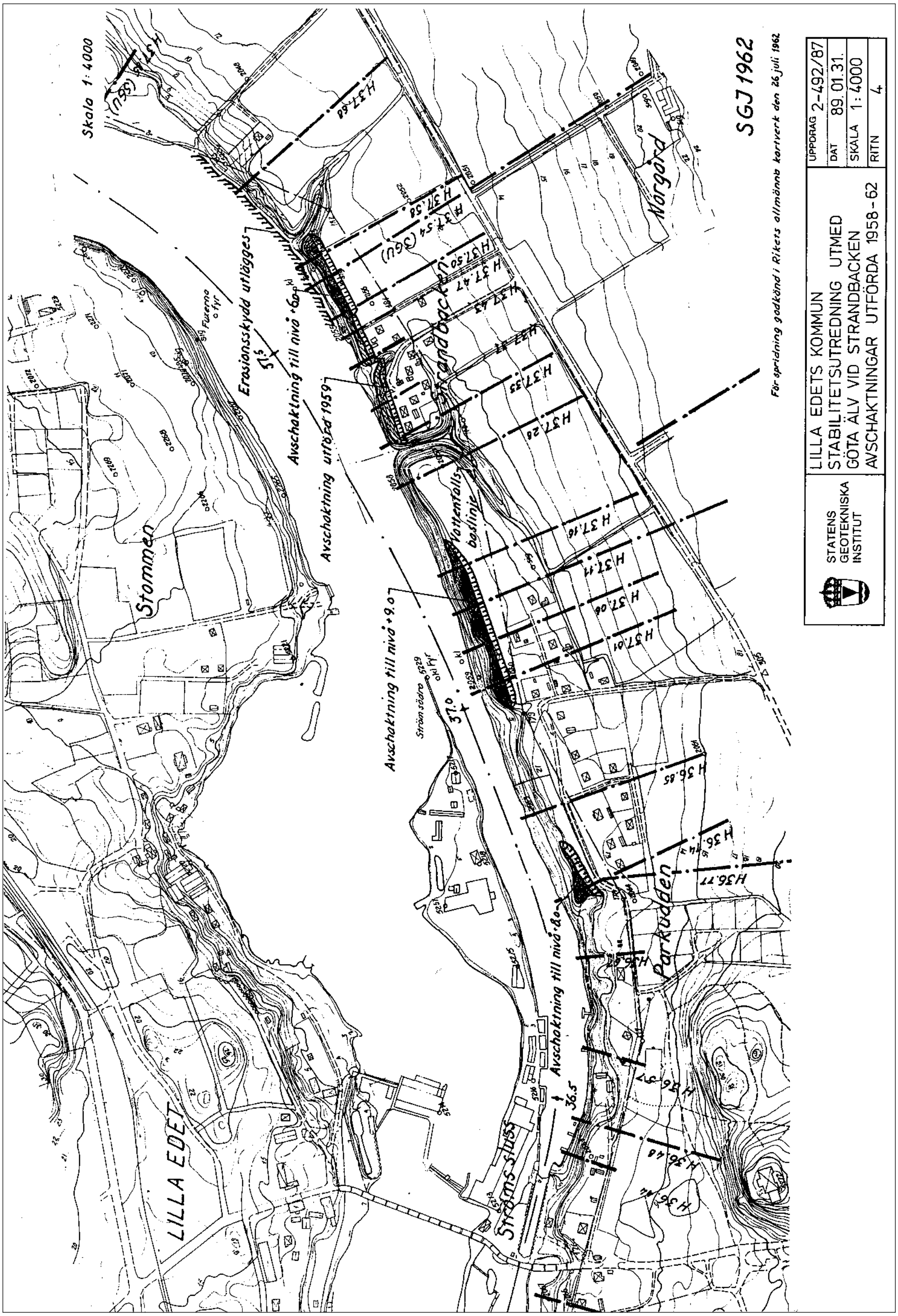


Fig. 66. **Investigated sections in and around the Strandbacken area in the Göta-älv investigation.**

and fall-cone tests were also different from those that are used nowadays. The investigations were therefore not coupled in such way that field vane tests and sampling were performed at the same test points; instead the different investigations were spread out along the sections with relatively long distances between the test points. No samples at all were taken in the investigations from a raft out in the river. This entails that the interpretation of these tests is somewhat uncertain and that a compilation of the results taking the local variations in soil composition into account cannot be made.

In the investigation by SGI in 1988–1989, the previous investigations in the Strandbacken area were supplemented by an ocular inspection and investigations in two boreholes in Section 37.33, which is located close to the ravine at the northern end of the area, Figs. 65b and 66. Two pore pressure measuring systems were also installed in a station located 10 metres behind the new upper crest since the excavation in 1959 and the pore pressure variations were observed for some time.

The ocular inspection revealed clear indications of ongoing movements in the slope. These indications consisted of leaning and bent trees in the slope, a leaning boat-shed at the toe of the slope and rolls and patches with bare soil on the surface of the slope. The 10–12 metre high slope had an average inclination of about 1:3. However, the inclination varied with steeper parts at the erosion protection, in the lower slope up to the excavated area and in the excavated slope up to natural ground level. More level parts were found on the excavated terrace and on top of the erosion protection. The river bottom was sounded by the fairway authority, Kanalverket, and was in this section found have a shallow inclination from about 2 metres' depth at the toe of the erosion protection to about 4 metres' depth about 15 metres further out. The depth rapidly increased to about 8 metres and then remained fairly constant to a distance of 40 metres from the riverbank, from which it slowly increased to a depth of 9–10 metres in the fairway. All depths refer to the lowest water level in the river.

The investigations consisted of CPT tests, field vane tests and standard piston sampling. The CPT tests were stopped at 20–25 metres' depth and field vane tests and sampling were performed at every third or fourth metre depth down to somewhat deeper levels. The purpose of these tests was only to confirm the previous results from the Göta-älv investigation, and to make possible a correction of the field vane test results with compatible liquid limits. A few oedometer tests were also performed in order to check the assumed stress history.

3.5 VARIATIONS IN SOIL CONDITIONS AND PROPERTIES

The results of the previous investigations showed that the slope towards the river was fairly steep and about 10 metres high within a distance of 1500 metres southwards from the lock at Ström. In the Strandbacken area, i.e. between sections 37.01 and 37.68, the slope was between 8 and 13 metres high, with the lowest heights in the southern end. Within the area that was involved in the last excavation, the slope was between 10 and 12 metres high, with an average value of just over 10 metres. The ground behind the crest rose gradually with a very small inclination towards the side of the valley. The soil consisted of clay with varying thickness underlain by sandy, gravelly material which was assumed to rest on till. The clay was covered by a layer of sand and silt with an estimated thickness of 2 to 5 metres. Thicker layers of sand and silt have been found in the present investigation. The difference may be due to the fact that the layer according to the old classification system was designated as sand and "mo", (coarse silt), whereas the "mjäla" (fine silt), was probably regarded as part of the clay layer. In the supplementary investigations in 1988, it was also found that the upper layers had significant organic contents down to about the zero-level, i.e. down to about 10 metres' depth. The zero-level roughly corresponds to the water level in the river when no water is let out through the power plant. The thickness of the soft soil layers was estimated to vary from a few metres at the lock to almost 50 metres at the southern end of the investigated area. Erosion protection had been constructed for the larger part of the distance but not in the southern part where erosion was going on and superficial slides occurred from time to time.

Within the area in and close to Strandbacken, the weight sounding tests had been stopped at levels of mainly –25 to –30 metres at the deepest. A few deeper levels were reached in the tests furthest to the south. In all cases, the weight sounding tests were stopped after a very large and gradually increasing amount of rotation had been required for penetration over a fairly large depth interval. The so-called machine soundings penetrated a few metres further and stopped at a level of about –35 metres, i.e. about 45 metres below the original ground surface. This stop level is fairly constant within a wide strip from the slope to about a hundred metres behind it. At larger distances from the river, the stop depths first increase somewhat and then decrease towards the valley side. The stop levels below the river are somewhat higher than on land. The clay was reported to contain some layers or infusions of sand and shells. However, these did not appear to be continuous but to have limited extents or occur as lenses. From the test results it is difficult to estimate exactly where the border between clay and underlying coarser soil lies. Samplings and field vane tests have generally been stopped somewhat above the stop levels for the weight sounding tests and the samples have consisted of clay down to these depths. At a few points, the samples from the deepest levels have contained single thin silt layers and in one point in Section 37.11, to the north of the Strandbacken area, there was sand and gravel in the lowest sample tube from the level –30 metres.

The water content in the clay was reported to vary between 60 and 100 % in the upper layers and decrease to between 30 and 50% at the bottom. The water content appears to be generally somewhat higher further away from the river than at and near the riverbank. The liquid limit is reported to vary in a similar way, but the maximum values only reach 90% at the most. The sensitivity varies from normal to very high values. Quick clay has been found at several locations in the investigated area, partly as thick layers. A closer study of the test results shows that the quick clays in this area generally occur at a relatively large distance from the river. However, quick clay was found all the way down to the slope at Parkudden, 200 metres to the north of Strandbacken in the investigations in 1988. The quick clay mainly occurs at distances of 200 metres and more from the river, i.e. on the opposite side of the main road. In Section 37.58, which is located within the Strandbacken area, the first sensitivity values corresponding to quick clay are found at a distance of 160 metres from the river. This layer is superficial and is only about 2 metres thick. On the other side of the road, about 230 metres away from the river, there is quick clay with sensitivity values of over 400 in a roughly 10 metre thick layer located deeper down in the profile. The conditions are similar in Section 37.11, located to the north of the area. The relation between the sensitivity and a quasi liquidity index expressed by the relation between water content and liquid limit, i.e. the quotient w_N/w_L, has been investigated for different areas along the Göta-älv valley, Fig. 67. In general, this quotient was found to be greater than 1.1 in the quick clays.

The shear strength properties in the sections perpendicular to the river were reported to be uniform when related to the level, apart from a softer superficial layer directly below the river bottom. As already mentioned, a closer comparison is rendered difficult due to the fact that sampling and shear strength testing has not been performed at the same points. The comparison is also obstructed by the fact that the thickness of the upper sand and silt layer and also the ground level vary along and across the river, and that the area contains a number of eroded ravines running perpendicular to the river. The latter have probably entailed that the groundwater level and the pore pressures have been lowered locally to varying amounts, resulting in further consolidation and increased stiffness in the clay in their vicinity. The same is valid for a strip along the river located near the crest of the slope. Buildings and constructions and connected drainage systems have also

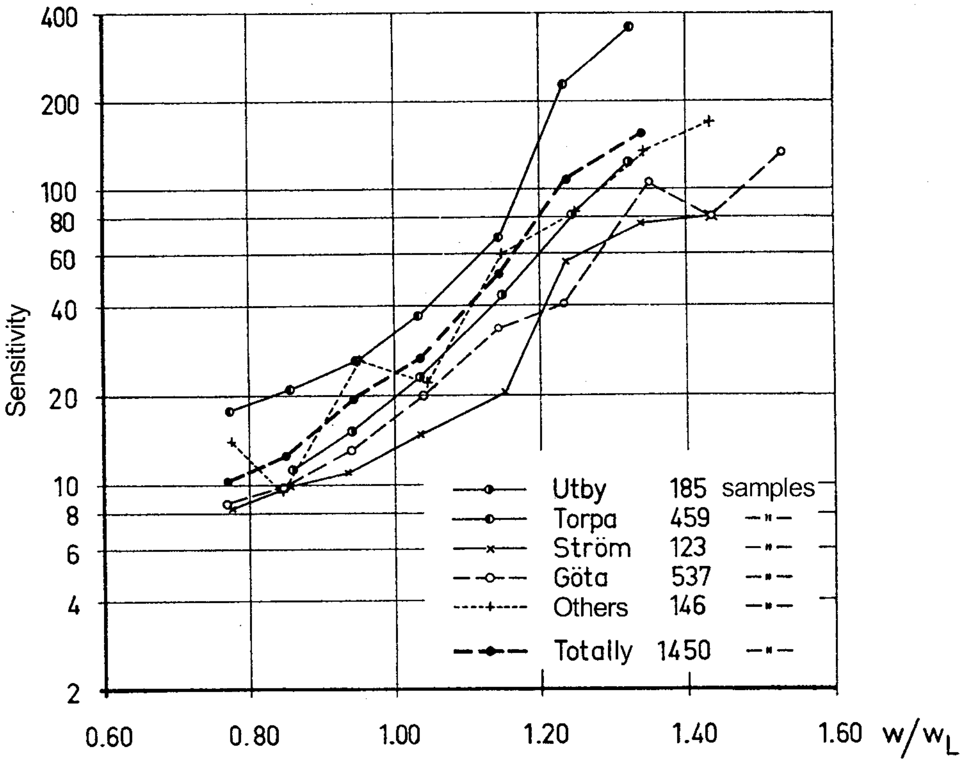


Fig. 67. **Measured relation between sensitivity and the quotient w_N/w_L for clays in the Göta-älv valley, (The Göta-älv Committee 1962.)**

existed in some sections. The soil at the riverbank constitutes a special problem since the stratification here can be assumed to be influenced by slides that have occurred in the slope. The amount of consolidation here may also be affected by previous constructions along the riverbank. Altogether, this results in a fairly wide spread in the test results. This is the case also when only the results in sections within a 750 metre long distance that covers the Strandbacken area and its closest surroundings are considered.

The tests on samples in the laboratory showed that the natural water contents were similar at each level but varied significantly from section to section and particularly with distance from the river within the sections, Fig. 68. No samples were taken below the river.

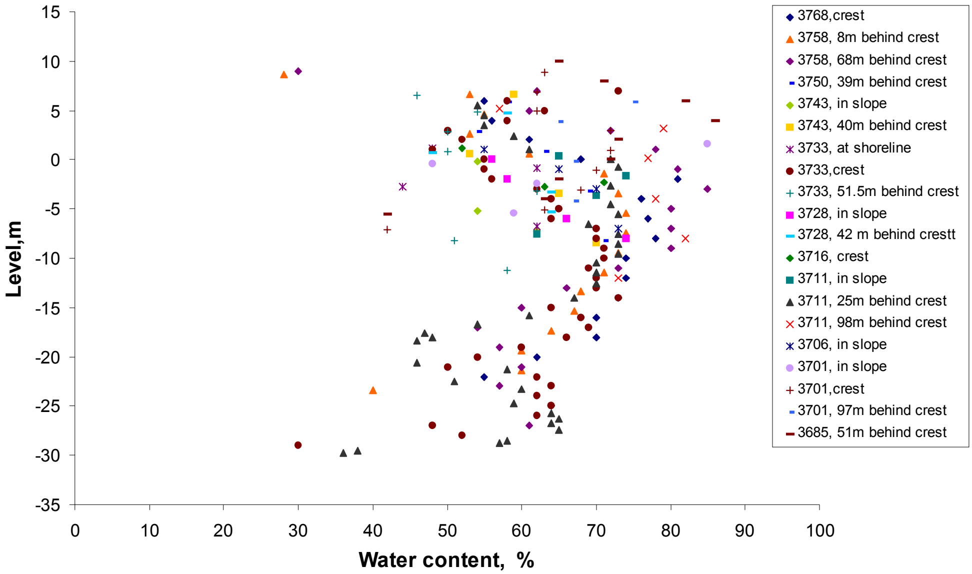
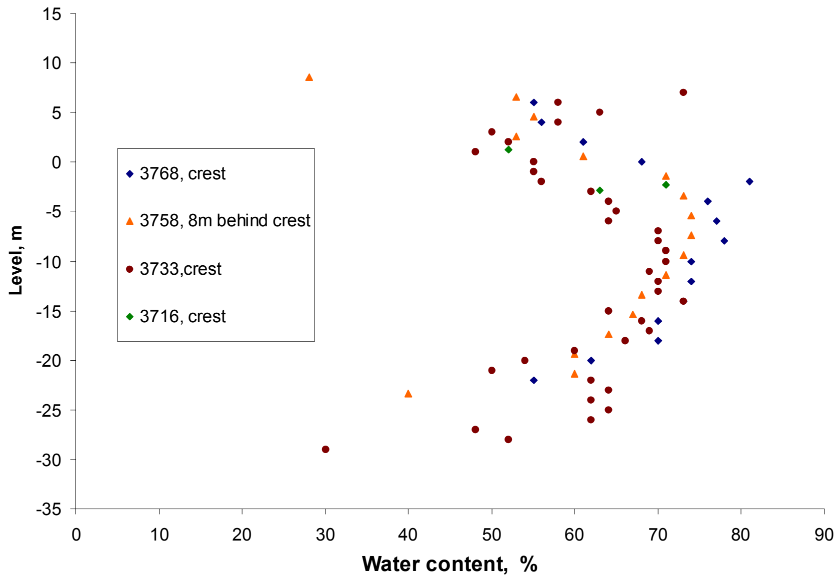


Fig. 68. **Compilation of measured natural water contents in and around the Strandbacken area.**

The spread in results is considerably reduced when the area is limited further and when the part close to the river is separated from the part far away from the river, Figs. 69a and b. It is then clear that the properties at the same distances from the river are more similar and that the water content is generally higher far away from the river. It can also be inferred that the maximum water content is found at a higher level at greater distance from the river.

The determinations of the consistency limits are fewer. A compilation of all determinations of liquid limit shows that it varies in approximately the same way as the natural water content and that the spread between different points is about the same. However, it is not possible to make the same separation of the data as for the water content and study the variation in different parts of the area. There is possibly an indication of higher liquid limits at some distance from the river, Fig. 70.



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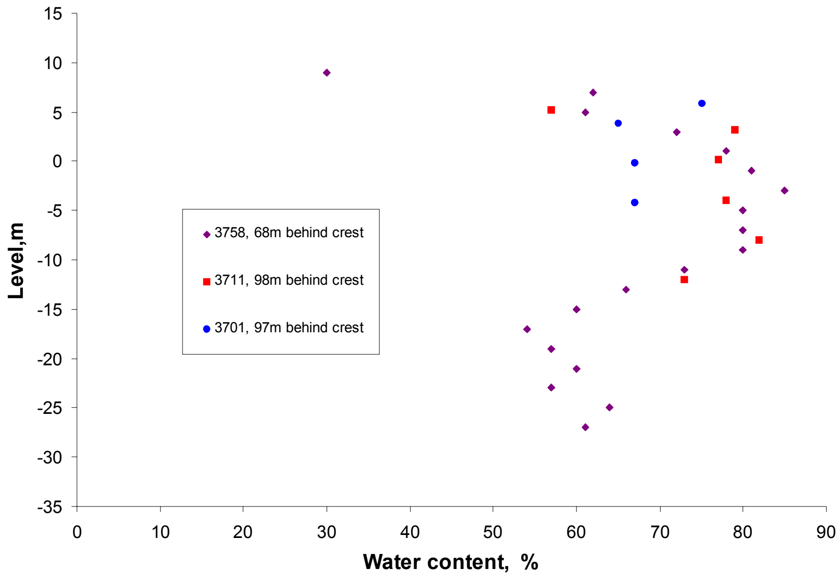


b)

Fig. 69. **Variation in natural water content in the soil profiles in the Strandbacken area.**
a) Close to the crest
b) At large distances from the river

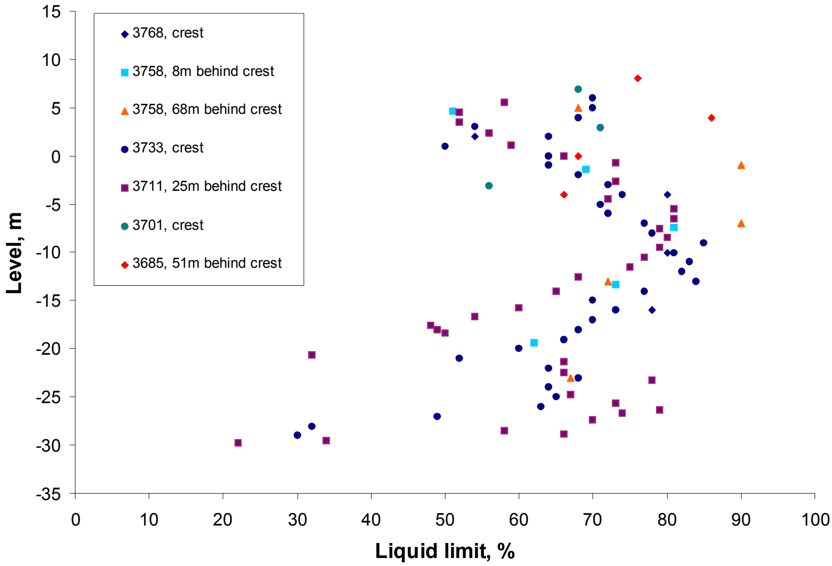


Fig. 70. **Variation in liquid limit in the profiles in and around the Strandbacken area.**

In the Göta-älv investigation, the plastic limit and the plasticity index were also determined, which is not common practice in Sweden. These determinations enable a plotting of a plasticity chart which provides another indication that the clay at larger distances from the river has different properties from that close to it even if it is not necessarily quick. A compilation of the results from all tests in the investigations south of the lock at Ström has been made, Fig. 71. It shows that in clay from areas with normal sensitivity values, the relation between liquid limit and plasticity index follows the normal trend of $I_P = 0.8(w_L - 0.18)$ (Karlsson and Hansbo 1984). Exceptions are some values in low-plastic soils, which relate to layered clay and silt. However, in those profiles where clay with considerably higher sensitivities and quick clay are found, the relation is quite different. Here it becomes $I_P = 0.66(w_L - 0.18)$ as an average. This relation corresponds to the lower borderline for Swedish inorganic clays and lies somewhat below the Casagrande A-line (Casagrande 1947). The latter is often used internationally as a borderline between inorganic clays and organic and/or silty soil.

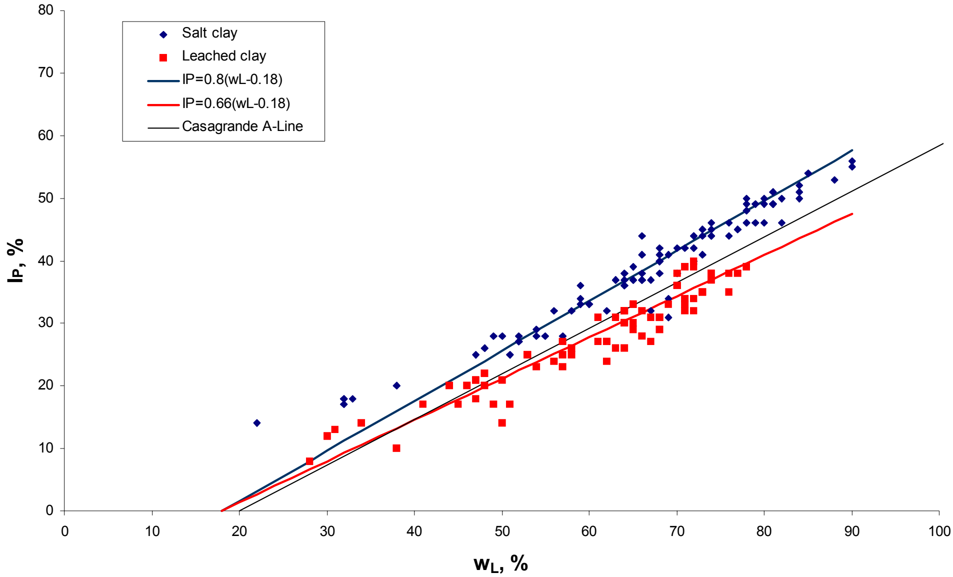


Fig. 71. **Measured correlation between liquid limit and plasticity index in the area south of the lock at Ström.**

The sensitivity in clays originally deposited in seawater has been found to be related to the leaching of salts from the pore water. Leaching of salt is known to reduce the liquid limit. From the results presented above, it also appears to change the relations between the consistency limits in such way that these indicate a more organic and/or silty soil than before.

A compilation of all field vane tests in the area shows that the general trend for the results is similar but the scatter is very large, Fig. 72. The results shown in the figure have been divided into groups, with red curves representing tests performed under the river bottom, blue curves representing tests performed in the lower parts of the slope at the riverbanks and green curves representing tests performed behind the crest of the slope. From this presentation, it is obvious that significantly lower shear strength values are measured below the river than in the slope and behind the crest. This is the case not only for superficial layers, as was stated in the report by the Göta-älv Committee, but for all of the investigated 20 metres thick clay layer. There is also a general tendency for lower shear strength values in the lower part of the slope compared to those measured behind the crest. As previously stated, it is not possible to correct these shear strength values with regard to the liquid limit.

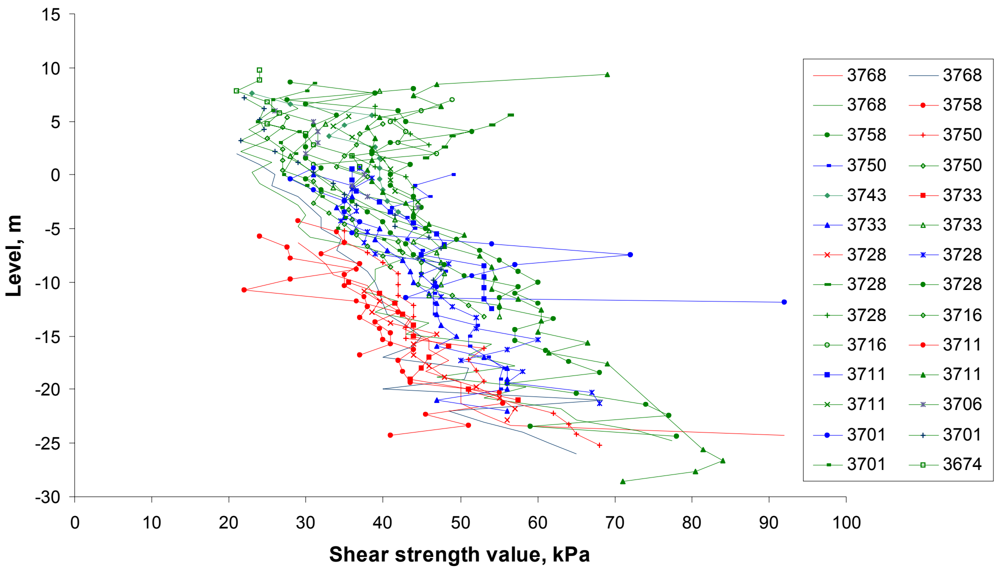
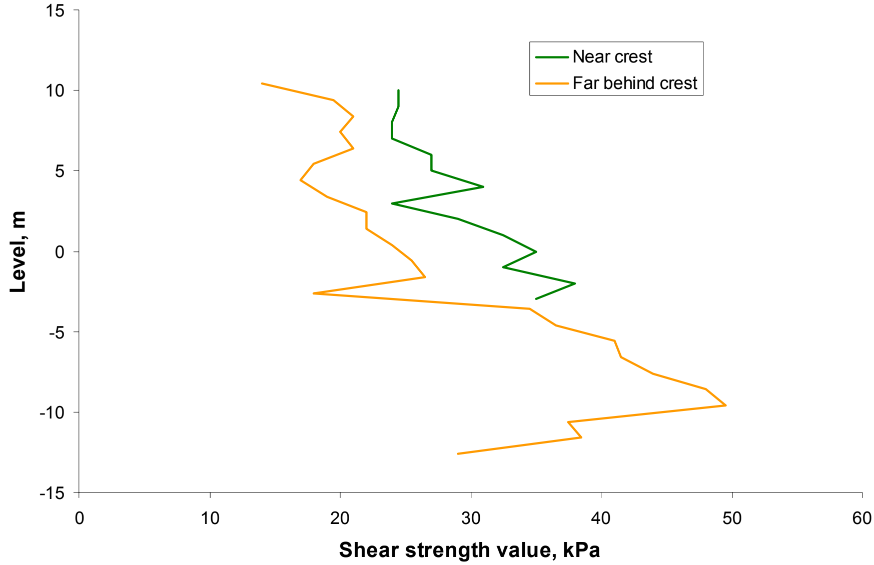


Fig. 72. **Compilation of shear strength values measured by field vane tests in the Strandbacken area. (Red curves represent tests below the river bottom, blue curves represent tests at the riverbank and green curves represent tests behind the slope crest.)**

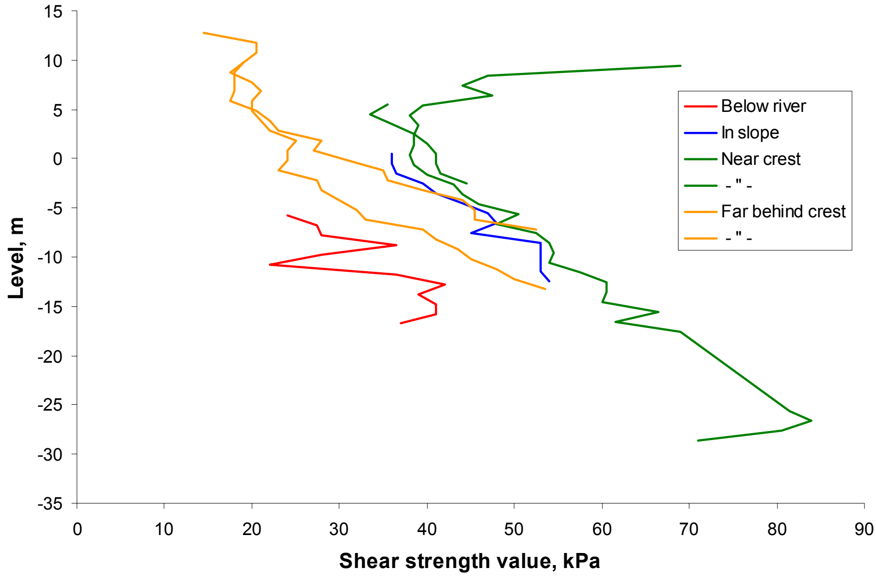
A presentation of the measured shear strengths section by section in most cases shows a shear strength that is lowest below the river, increases below the slope, is highest in the area just behind the slope crest and then decreases again at a larger distance from the river, Fig. 73. The pattern is clear in many sections but is less pronounced in others.

The shear strength values can also be presented in groups containing tests below the river, at the riverbank and close behind the crest. This has been done for the results that are most relevant for the Strandbacken area, Fig. 74. The results obtained from tests in 2000 within this project have also been included for comparison.

A trend for each group of values versus the depth below the slope crest has been roughly estimated and it is obvious that the results of the shear strength tests are affected by where in the section they have been performed. It is also obvious that the effect lasts throughout the investigated depth interval, although it decreases with depth both relatively and in absolute numbers.

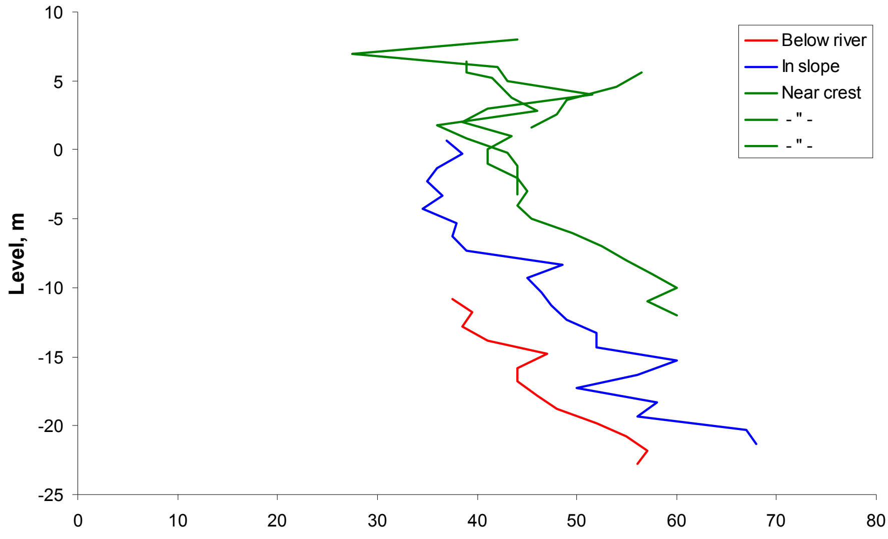


a)

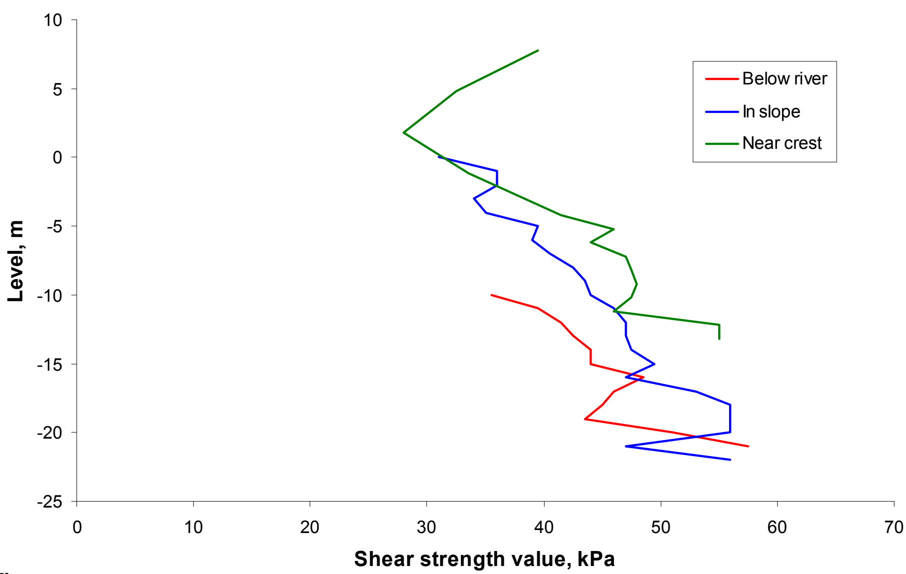


b)

Fig. 73. **Measured shear strength values in field vane tests in sections in the Strandbacken area.**
a) Section 3685
b) Section 3711

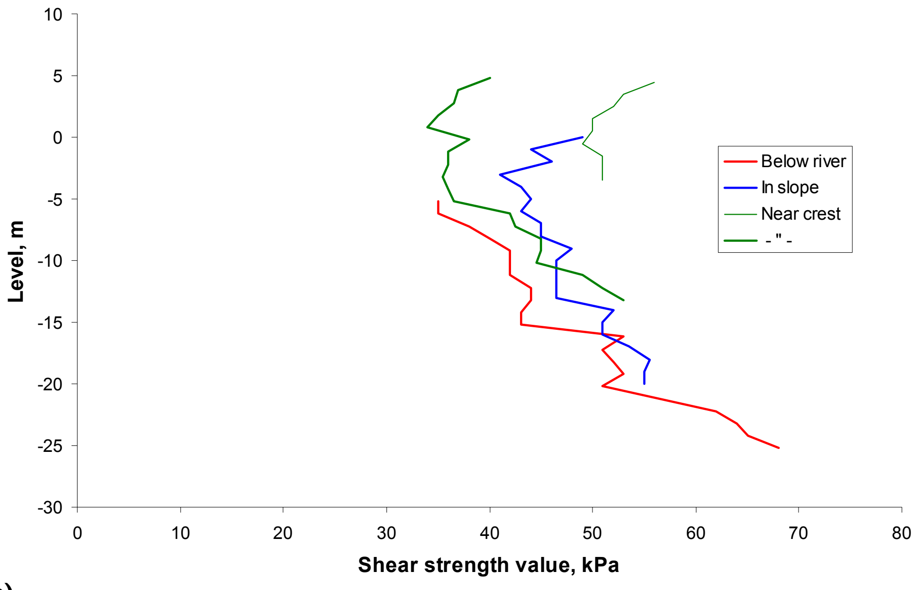


c)



d)

Fig. 73. **Measured shear strength values in field vane tests in sections in the Strandbacken area.**
c) Section 3728
d) Section 3733



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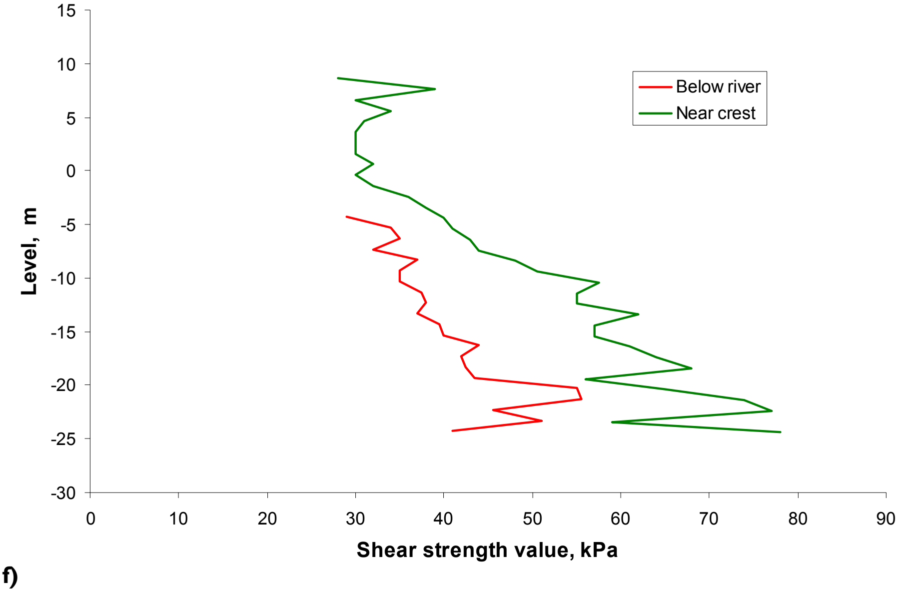
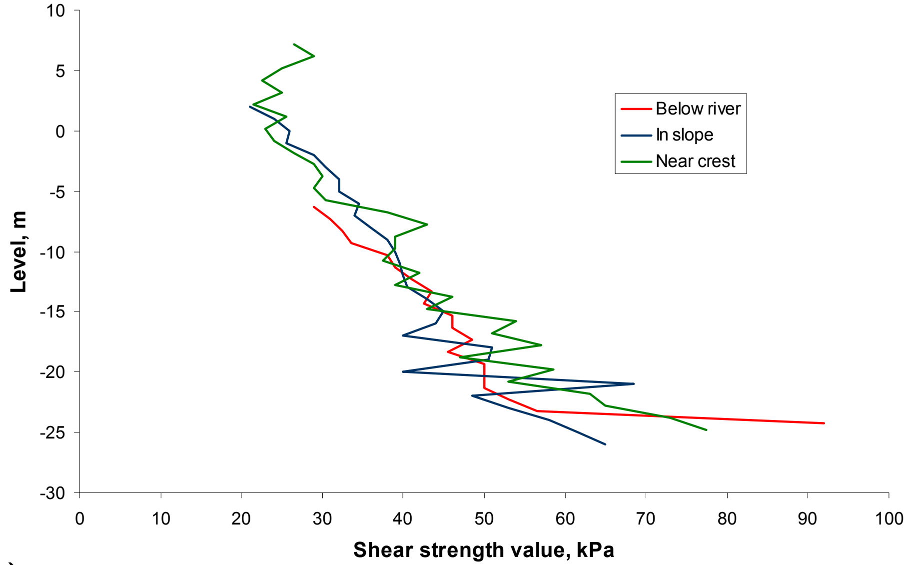


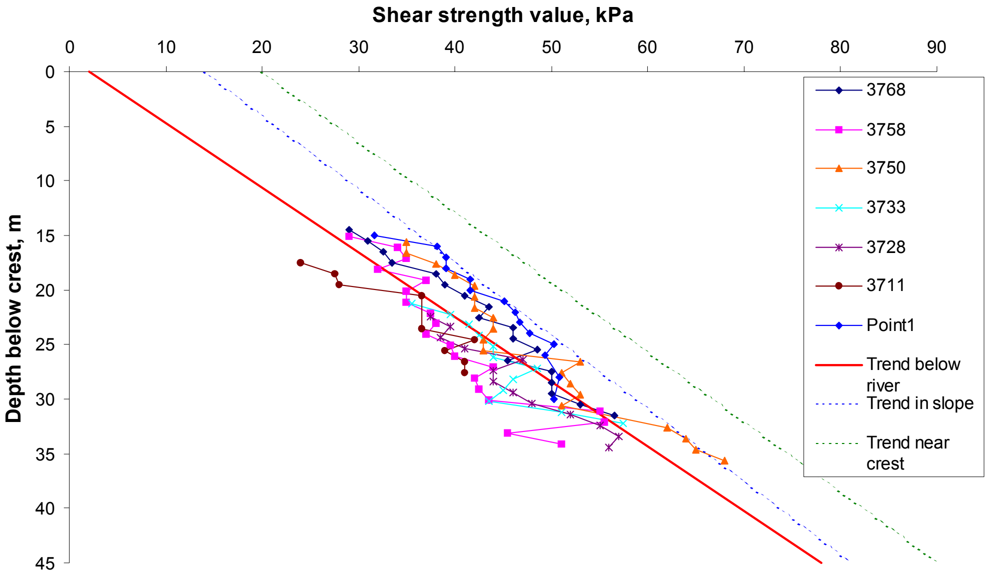
f)

Fig. 73. **Measured shear strength values in field vane tests in sections in the Strandbacken area.**
e) Section 3750
f) Section 3758



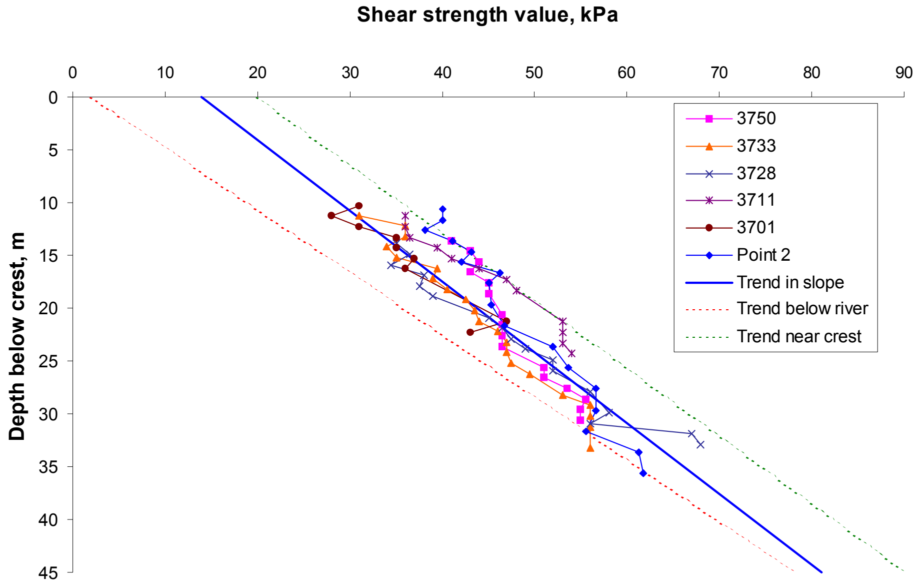
g)

Fig. 73. **Measured shear strength values in field vane tests in sections in the Strandbacken area.**
g) Section 3768



a)

Fig. 74. **Shear strength values obtained by field vane tests in different parts of sections perpendicular to the river in the Strandbacken area.**
a) below the river



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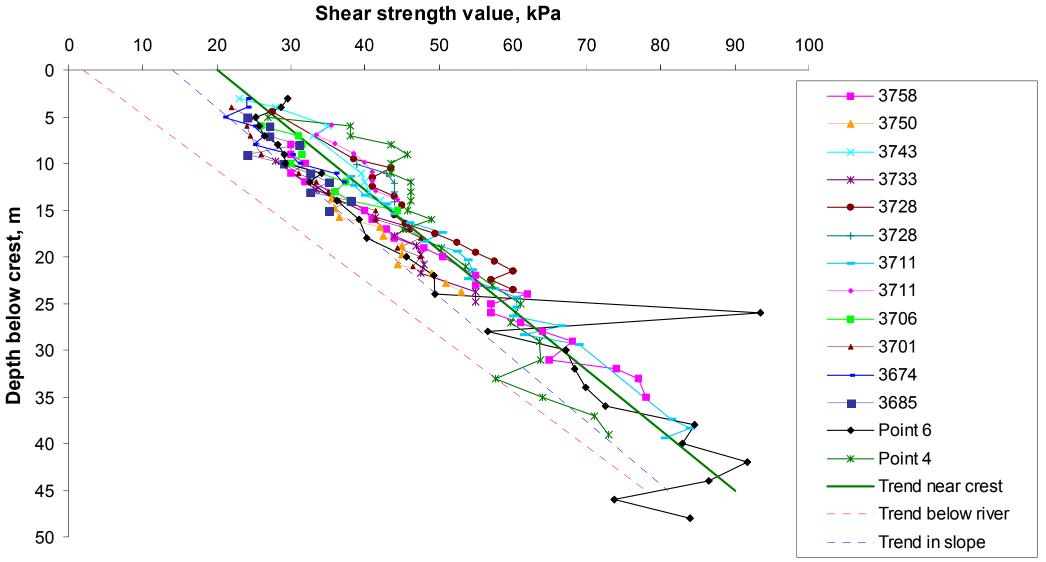


c)

Fig. 74. **Shear strength values obtained by field vane tests in different parts of sections perpendicular to the river in the Strandbacken area.**
b) in the riverbank
c) close behind the crest

The pore water pressures were measured in open systems in two sections in and adjacent to the Strandbacken area during the Göta-älv investigation. In systems that were installed far behind the slope crest, the pressure level in superficially located filter tips was found to be just below the ground surface and there was then a downward hydraulic gradient. In systems located at medium distances from the crest, the pressure levels in superficial filter tips were still just below the ground surface, which was here at a level a few metres lower. The pressures at depth here were described as almost hydrostatic from the upper groundwater level. Close to the crest, the pore pressures were lower in the upper soil layers and the free ground water level here was located a few metres below the ground surface, depending on the thickness of the upper sand and silt layers. The ground surface was also generally at a somewhat lower level than further away from the slope. The pore pressure gradient here had shifted to become slightly upward. The pore pressures down at the riverbank were described as slightly artesian. The majority of the observations were made during July 1961, which can be assumed to correspond to a relatively dry period.

The pore pressure measurements during the later investigation in 1988-89 were made in a point located about 10 metres behind the upper crest behind the terrace of the first excavation. One filter tip here was installed close to the lower boundary of the upper sand and silt layer and another tip was placed in the clay 11 metres further down. A longer observation period showed that the pore pressures in the upper layer varied a lot and corresponded to a free groundwater level from a half to about three and a half metres below the ground surface. The measured pore pressures in the clay varied in the same way but with a smaller amplitude.

The measured pore pressures are plotted versus level in Fig. 75. A direct comparison is obstructed by the fact that the ground level and the thickness of the sand and silt layer vary somewhat between the measuring sections, which is probably also the case for the underlying bottom layers of coarser and permeable soil. The trend lines have been drawn using guidance also from the observations of the pore pressures in the bottom layers in the present investigation.

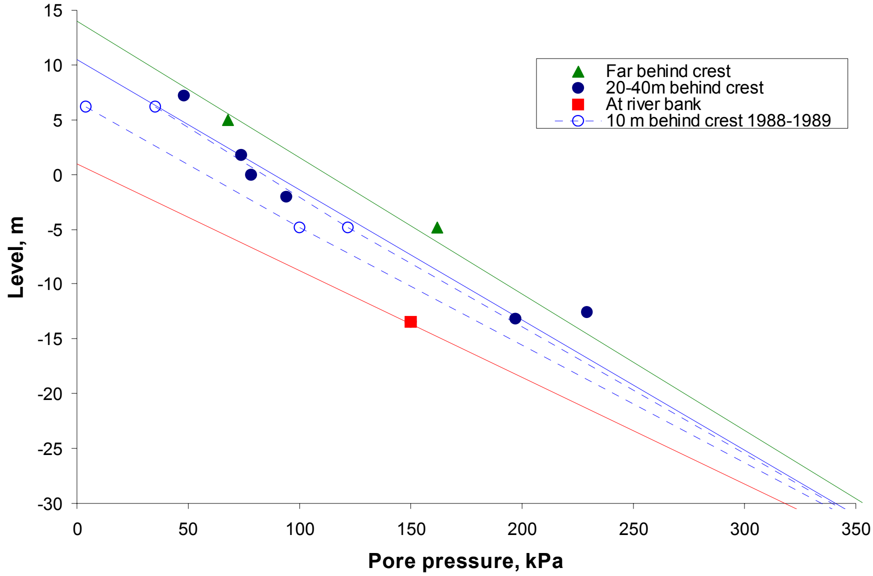


Fig. 75. Pore pressure observations in previous investigations in the Strandbacken area.

3.6 STABILISING MEASURES

The large stabilising works in the Strandbacken area were performed directly after the investigation in 1989. They finally comprised an excavation at the crest of the slope, which was about 250 metres long and totally about 50 metres wide and entailed a lowering of the ground surface within this area of between 4 and 5 metres. In 1959, 30 years earlier, a smaller excavation had been made with a width of about 10 metres to remedy an acute risk of slope failure. At the same time, the erosion protection along the riverbank had been extended southwards to cover the whole distance past the Strandbacken area.

The works during 1989 meant that a large number of properties were bought up and the buildings demolished. All buildings within a distance of 40 metres from the upper crest after the first excavation were involved. This was done already during the course of the investigation, when the results of the stability analyses were available. The excavation works then commenced. These were performed in such a way that a single terrace with an almost flat and about 50 metre wide surface was created with a ground surface at a level of about +6 metres. According to the measurements performed in the present investigation, the inclination is very small, only about 1:80, from a level of about +6.3 metres at the rear to about +5.7 metres at the crest of the slope down to the river. This is even less than the already small inclination of 1:50 that was proposed in the report from 1989. The excavation was made in the upper sand and silt layer. Most of this was taken away, but part of it still remains. No replanting of the area has been mentioned and none seems to have been attempted.

The excavation was designed with the aim that the factor of safety according to an undrained analysis should be at least 1.5 for the remaining built-up areas. This was also the aim for smaller potential slip surfaces in the outer slope. For larger slip surfaces in areas where no buildings were involved, a safety factor of 1.3 according to undrained analysis was aimed for.

3.7 NEW INVESTIGATIONS

3.7.1 Observations

An ocular inspection of the area was made before the new field tests started and the condition of the area was documented by photographs. Further observations have been made during the course of the investigations. The area is large and a full view can only be obtained from the other side of the river. From this side and from passing ships in the river, at normal water levels one can mainly see a solid erosion protection structure of rock-fill at the riverbank and the lower slope, which is overgrown by trees and bushes. The front of the built-up area has been moved from just behind the riverbank to a distance of about 80 metres away from the river. From this side the houses as well as the excavated terrace and the upper slope at its rear can mainly be seen during wintertime. In summertime they are largely obscured by the trees in the lower slope and at its crest, Fig. 76.

The erosion protection constructed of rock-fill starts some distance out in the water and then covers the riverbank to a height of about 2 metres above the zero level, Fig. 77. Scattered stones are also found all over the bottom shelf that slopes gently from about a metre or so depth at the riverbank to 4–5 metres' depth before it gets a steeper inclination down to the deep channel in the fairway. The fairway and the channel into the lock should have a minimum depth of 5.95 metres at the lowest water level in the river. This is regularly checked. The depth in the fairway in the river is generally larger, but the information is somewhat uncertain. In the sections in the Göta-älv investigation 1962, this depth varied mainly between 8 and 11 metres, and measurements in the northern part of the Strandbacken area showed depths between 8 and 10 metres. However, according to the map from the fairway authorities based on soundings in 1975, the water depths in the fairway in the area

a)

b)

Fig. 76. **The Strandbacken area seen from the opposite side of the river in late October 2000.**
a) The visible part of the area. The river and a landing pier on its eastern side are seen in the foreground. The erosion protection and the lower part of the slope can be clearly observed, the excavated terrace can be discerned and the remaining buildings can be seen in the background.
b) A close-up of a section where the different levels can be better observed.

Fig. 77. **Riverbank and erosion protection.**
a) The riverbank towards the south. A bollard can seen in the river with a marker for the western side of the fairway.

Fig. 77. **Riverbank and erosion protection.**
b) The riverbank towards the north. The protruding point at the old access road to the riverbank obscures parts of the bank. The bridge at Lilla Edet, the entrance to the lock at Ström, the Inland paper mill and the hydroelectric power plant at Lilla Edet can be seen in the background.

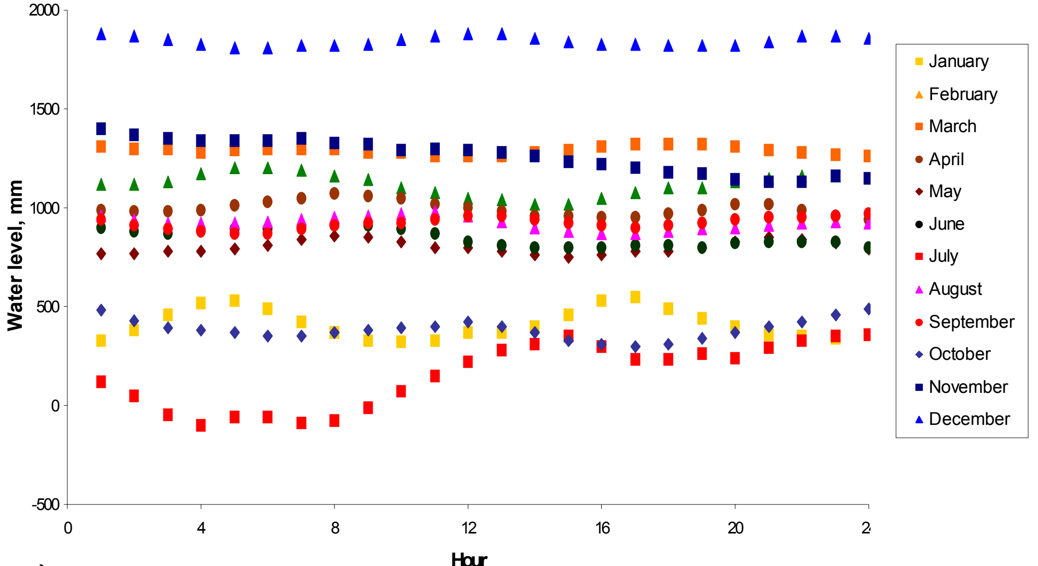
mainly vary between 6 and 8 metres, with a deeper part with a bottom level at about –8.2 metres at Strandbacken.

The lowest water level in the river is given as –0.25 metres. The water level in the river varies strongly also during the day. It is somewhat affected by the tide and the wind direction since there are no further locks downstream towards the sea, but it is mainly influenced by the power production and outlet of water at the power plant 1.5 kilometres upstream. It normally appears to vary between the levels 0 and +1.4 metres. However, during the extremely wet winter in 2000–2001, when the areas around Lake Vänern became flooded and emergency tapping of water through the river was necessary, the water level was constantly close to the top of the erosion protection, i.e. at about +2 metres.

The variations can be illustrated by the measurements of water level and water outlet performed by the fairway authorities during year 2000. The variations in water level and water flow during one of the first days in each month are shown in Fig. 78. It can here be seen how the water level, like the tide, varies regularly with two peaks and two lows each day. The water flows during the same periods were almost constant and thus cannot have affected the variation except for the selected day in July. On that day, the water outlet increased significantly after about half the day, which directly affected the water level in the river. The average water levels during the selected days can also be seen to be related to the corresponding water flows.

The effect of the water flow is also illustrated in Fig. 79, where total water flow and mean water level for each day during the year are plotted. Apart from raising the water level, an increased water flow also entails that the width and section area of the river and the rate of the water flow in it increase. The cubic root of the water flow has therefore been selected as a relevant value for comparison with the water level in the river. The figure shows clearly that the water flow is the main factor that affects the water level in the river and that the water level decreases rapidly when the power production is reduced during long holidays and the vacation period or for some other reason. The effect of the emergency tapping at the end of the year to reduce the effects of the flooding around Lake Vänern is also clearly illustrated.

At the riverbank in the Strandbacken area, there is a small hut above the erosion protection, which is used by the local fishermen. A few simple wooden jetties have also been constructed along the riverbank.



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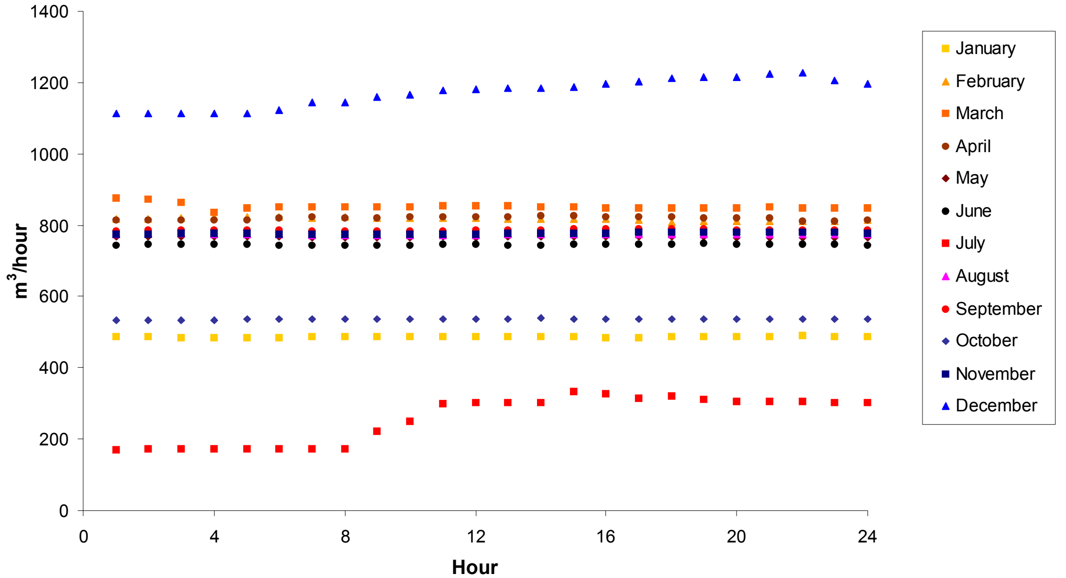


b)

Fig. 78. **Water level in the river on the downstream side of the lock at Ström and water outlet at the Lilla Edet power plant during one of the first days in each month in 2000.**
a) Water level
b) Water flow

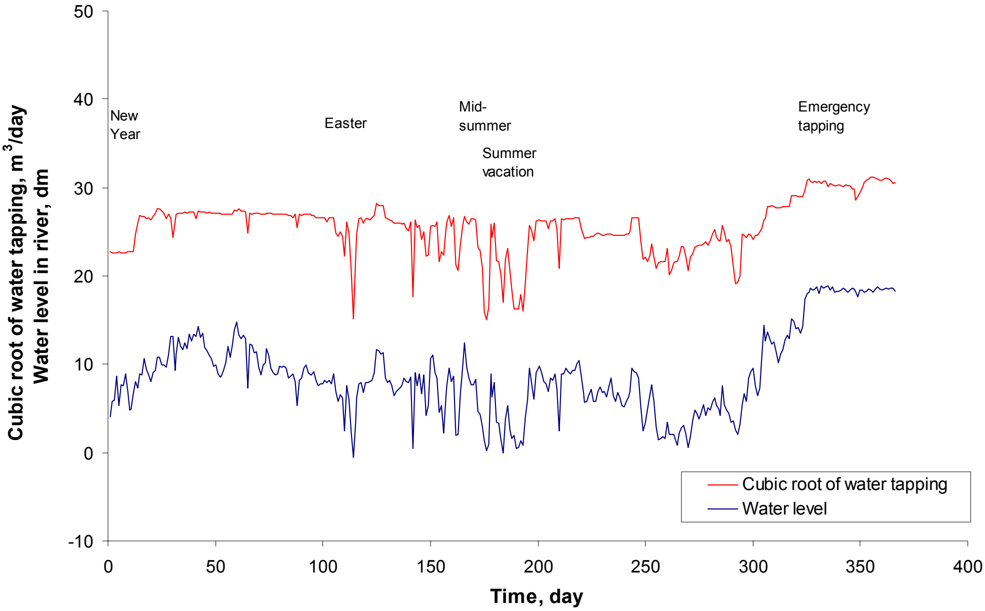


Fig. 79. Mean water level and water flow in the river during 2000.

Parts of the lower slope are fairly steep with inclinations larger than 1:1.5. However, the excavation has reduced the height of this slope and the construction of the erosion protection has entailed a considerable reduction in the average inclination.

A walking trail runs along the western side of the river. To the north of the Strandbacken area, it runs on top of the erosion protection or just inside this. The erosion protection has created a terrace here, which is generally about 2 metres wide but which widens to up to about 10 metres in places where slides have occurred in recent times. After passing a small bridge across the ravine that constitutes the northern border of the Strandbacken area, the trail runs on the excavated terrace along the crest of the lower slope where the ground is relatively dry and firm, Fig. 80. In the middle of the area, where the old access road to the river ran, there is a simple access road with a thin embankment, which in dry periods can be used by light vehicles. This road turns southwards at the crest of the lower slope and coincides with the walking trail. The road and the trail are mainly used for walks and airing of domestic dogs.

Fig. 80. **The trail along the lower crest in the southern part of the area where it can also be used by light vehicles.**

The most significant feature of the excavated terrace is its marshy nature. The ground surface is normally very wet. This is the case throughout the year at the rear end of the terrace and for most of the year in all other parts except for a strip close to the lower crest. The vegetation consists mainly of reeds, which grow in large clumps, sedges and a few birch trees. In large spots there is no vegetation at all. Here the grey sandy silt lies open and no vegetation has managed to become rooted during the more than 10 years that have elapsed since the excavation works. Birches grow at the edges of the terrace towards the ravine in the north and the slope towards the river, where the ground conditions are drier, Fig. 81.

The area is somewhat drier and the vegetation is somewhat lusher in summertime, but the marshy nature is still evident, Fig. 82.

At the rear end of the excavated terrace, the groundwater level is located at the ground surface all year through since water is seeping out of the upper excavated slope in which the soil consists of sand and silt. As a result of this seepage, internal erosion occurs at places in this slope, Fig. 83.

The remaining buildings are located behind the upper slope. Some property reaches all the way to the crest of this slope, but the nearest building is located about 10 metres away from this, Fig. 84.

Fig. 81. **The excavated area in late October in year 2000.**
a) Reeds and sedges on the northern part of the terrace. The birches in the background stand at the crest above the ravine.

Fig. 81. **The excavated area in late October in year 2000.**
b) Birches at the crest of the slope towards the river and reeds on the southern part of the excavated terrace.

Fig. 81. **The excavated area in late October in year 2000.**
c) The southern part of the excavated terrace with reeds and sedges and a large spot with bare sand and silt. The upper excavated slope is seen in the background together with birches growing on this slope too.

Fig. 81. The excavated area in late October in year 2000.
d) Winter-green sedges at the border of a bare spot

Fig. 81. **The excavated area in late October in year 2000.**
e) View of most of the excavated area looking towards the north with the river and the built-up centre of Lilla Edet in the background. Bare spots with sand and silt can be seen through the trees.

a)

b)

Fig. 82. **The excavated terrace in June 2001.**
a) View of the northern part from the upper crest
b) Nature in the southern part.

Fig. 83. Example of internal erosion in sand and silt in the upper slope.

Fig. 84. The crest of the upper slope and remaining buildings. The truck in the background is partly backed down on the access road to the excavated terrace.

North and south of the remaining buildings in the area there is open farmland up to the main road. Behind this, there are new buildings reaching all the way up to the rocky hills at the border of the valley, Fig. 85.

Requests for permission to build on the land between the upper crest and the main road have been made. However, the authorities have been restrictive and such requests have only been granted in areas very close to the main road.

3.7.2 Location of the new investigations

The new investigations in this project have been located in a section through the central parts of the excavated area. They have been performed at six points, at four of which detailed investigations with field tests and sampling were performed whereas only pore pressures were measured at two. The four points with detailed investigations were placed in such way that one is located between 20 and 25 metres out in the river, one is located in the riverbank, one is located at the centre of the excavated terrace and one is located in original ground about 25 metres behind the new upper crest. The desire was to find points with as well defined conditions as possible, which at the same time were easily accessible and did not require intrusion on private properties or obstruction to the ship traffic in the fairway. It was therefore not possible to place the points in a straight line perpendicular to the river. Instead, the points on land were placed in a line with an oblique angle towards the river. At the riverbank, this line bent backwards about 90° towards the investigation point in the river, Fig. 86.

The points were numbered from 1 to 6. Point 1 is in reality a short line about 20–25 metres out in the river. A raft was borrowed from the fairway authorities and was moved along this line in such way that it did not interfere with the fairway in the deep channel. The raft was fixed in position along the river using cramp irons in the protruding point of the erosion protection at the old access road and the bollard holding the marker for the side of the deep channel. A large number of bottom anchors and ropes to firm objects ashore were also used to fix the raft sideways, Fig. 87.

Point 2 is located slightly inside the erosion protection at the former access road, which has been mentioned earlier. The shoreline here protrudes into a small cape and the point is located approximately in the line of a straightened-out riverbank, Fig. 88. All test holes at this point had to be pre-drilled about 2 metres to ensure that no stones or old fill material would damage the test and sampling equipment.

Fig 85. **The area south of the remaining buildings in the Strandbacken area. The rocky hills, which constitute the western boundary of the Göta-älv valley, are seen in the background.**
a) Open farmland up to the main road and new houses on the other side of the road. The border of the remaining built-up area in Strandbacken is seen at the right side of the picture.

Fig 85. The area south of the remaining buildings in the Strandbacken area. The rocky hills, which constitute the western boundary of the Göta-älv valley, are seen in the background.
b) The upper crest and open farmland towards the south.

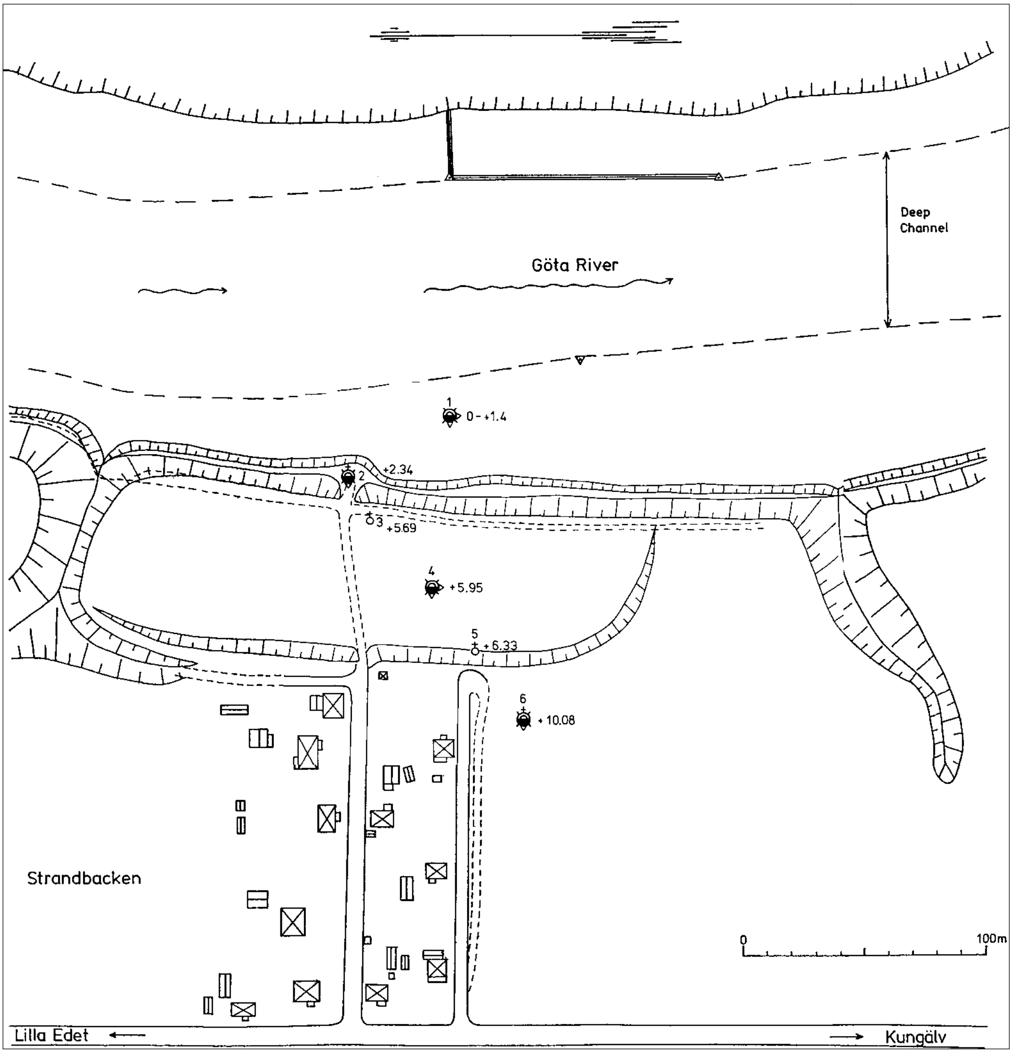


Fig. 86. Plan of Strandbacken with the present excavated terrace, remaining buildings and investigation points.

Fig. 87. The raft and drill rig in position in Point 1 out in the river.
a) View from the western riverbank. The bollard used for positioning is seen to the right in the picture.

Fig. 87. **The raft and drill rig in position in Point 1 out in the river.**
b) Preparations for a dilatometer test with automatic recording of test data in point 1. The bridge at Lilla Edet and the entrance to the lock at Ström can be seen in the background to the left of the drill rig.

Fig. 88. **Point 2 at the riverbank. The pipes for the pore pressure measurement systems are seen on the left hand side of the area with flattened reeds. The other investigations have been performed to the right of these pipes.**

Point 3 is located about 4 metres inside the crest of the lower slope. Only pore pressures are measured at this point. The location is selected to place the systems close to the crest but at the same time in a protected position where they do not interfere with the road and trail. The location is shown in Fig. 81b, where the top of the pipes can be seen among the birch trees to the right of the road.

Point 4 is located at the middle of the flat excavated terrace. It is placed in one of the bare spots where no vegetation has been established. The spot is seen at the right side in Fig. 81c.

Point 5 is located at the toe of the upper slope from the rear of the excavated terrace to the natural ground surface. The position is shown in Fig. 89.

Fig. 89. The rear end of the excavated terrace with the pipes of the pore pressure measuring systems in Point 5 placed at the toe of the upper slope.

Point 6, finally, is located about 25 metres behind the crest of the upper slope in what until recently has been cultivated farmland. The only change that has occurred in this area is that a contractor has bought the land and that a few decimetres of topsoil have been scraped off and sold to gardeners. The pipes for the pore pressure measuring systems are seen in Fig. 85a near the side of the road, where they are out of the way. The other investigations at this point were carried out further out on the open field.

The points in this section are located between sections 37.43 and 37.50 in the Göta-älv investigation.

3.7.3 Field tests

CPT tests

CPT tests have been performed at Points 1, 2, 4 and 6 in the new investigation. The tests were performed with a clay probe, which is a probe with a high sensitivity and resolution but a load capacity limited to 5 MPa in cone resistance. Pre-drilling has therefore been performed through any upper layers of fill or dry crust. No such layers existed at Point 1 and at Points 4 and 6 the crust effects and stiffer upper layers were limited to about 1 metre. At Point 2, which was located at the erosion protection and old access road, pre-drilling down to 2.9 metres had to be done to ensure that all coarse cobbles, stones and remains of old fills had been passed.

The drill rig was anchored by screw augers in the tests on land. The tests here were stopped when the maximum load capacity of the probe was reached in stiff bottom layers. In the test below the river, the drill rig could not be anchored. This test was stopped when the front of the drill rig lifted, which almost coincided with reaching the maximum probe capacity. The stop levels are deeper than those reached in the weight sounding tests and correspond approximately to those reached in "machine sounding tests" in the previous investigations. It is thus unlikely that significantly greater depths would have been reached if a coarser probe and more penetration force had been used.

The results indicate that the soil below the upper layers of sand and silt or fill material consists of relatively homogeneous clay. However, according to the trends in pore pressure generation and tip resistance with depth, which show a number of significant breaks, the character of the clay varies with depth. At greater depths, there appears to be a certain content of shells or other coarser material, and indications of coarse objects or layers are also found. A rather thin embedded stiffer layer is found at a relatively great depth at all points. The layer is located at a level of –27 metres at Points 4 and 6, and below the slope and under the river it rises to a level of about –22 metres.

The pore pressure registrations indicate that this layer may be draining. The CPT tests have passed through the layer and below there is another layer of stiff clay with a somewhat different character. Further down, stop in penetration occurs in coarse permeable soil, Fig. 90.

The tip resistance in Point 6 was found to be rather low in relation to the overburden pressure and the results at the other points. The test was therefore repeated, but the results of the two tests at this point proved to be almost identical, Fig. 91.

Dilatometer tests

Dilatometer tests were performed at Point 1 out in the river and Point 4 on the excavated terrace. The purpose of these tests was to obtain a supplementary classification, alternative determinations of the undrained shear strength and the overconsolidation ratio and an estimate of the in situ horizontal stresses. The test from the raft in the river reached a depth of only about 19 metres below the water level before the front of the drill rig lifted. This is 8 metres less than the CPT test and is probably a result of the side friction against the coarser drill rods in the stiff clay. The test at Point 4 reached 39 metres depth, which is the same as for the CPT test.

The results from the dilatometer tests confirmed the information obtained from the CPT tests, but fewer details of layers could be observed, Fig. 92.

Field vane tests

Field vane tests were performed at Points 1, 2, 4 and 6 using equipment type SGI and the normal vane size. The test depths were selected with guidance from the CPT test results and in general comprised the whole profile. The exception was the tests from the raft in the river, where friction against the large protecting tubes in combination with the unanchored drill rig meant that the tests had to be stopped at the same level as the dilatometer tests.

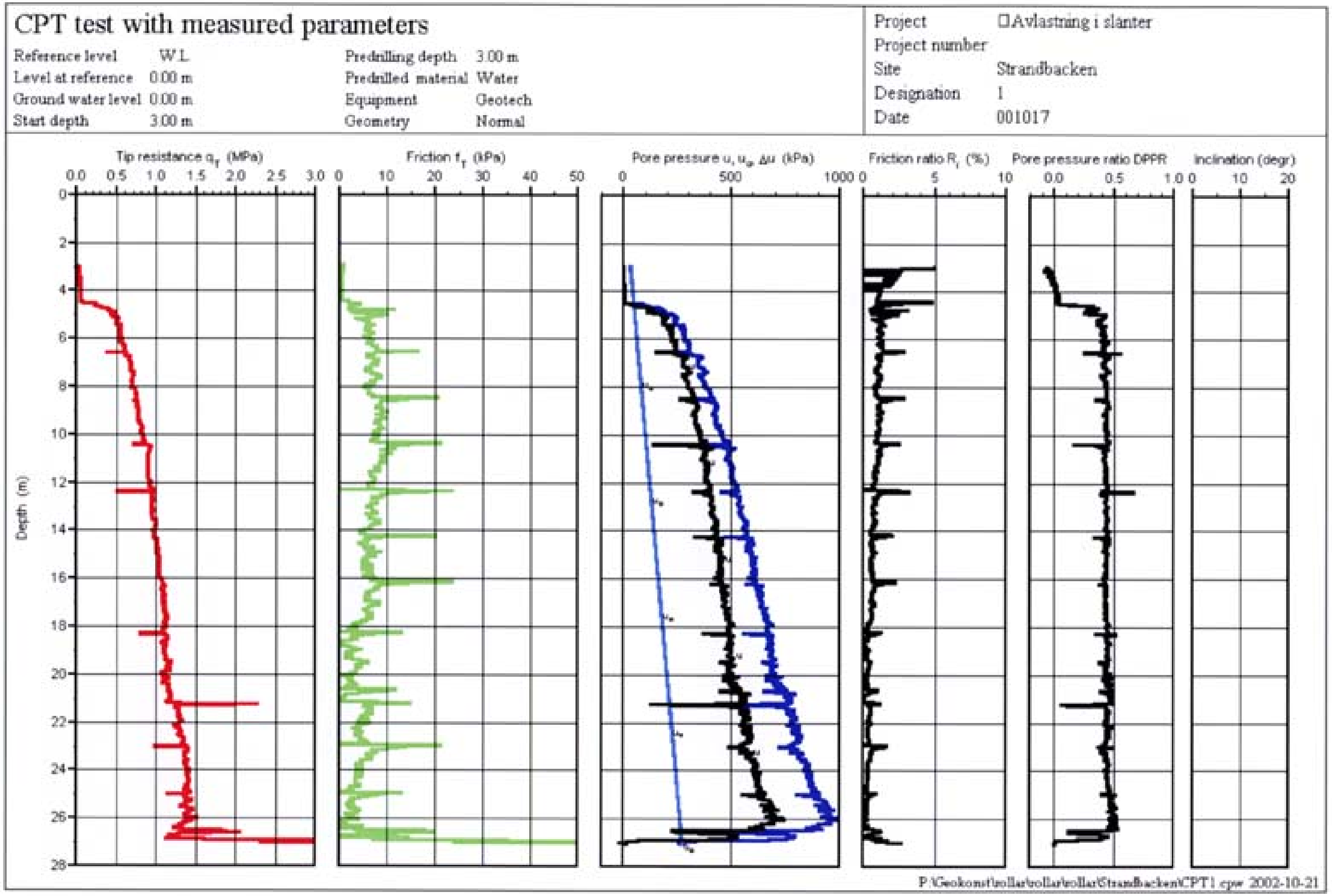


Fig. 90. **Results of the CPT test in Strandbacken presented using the program CONRAD. a) Point1**

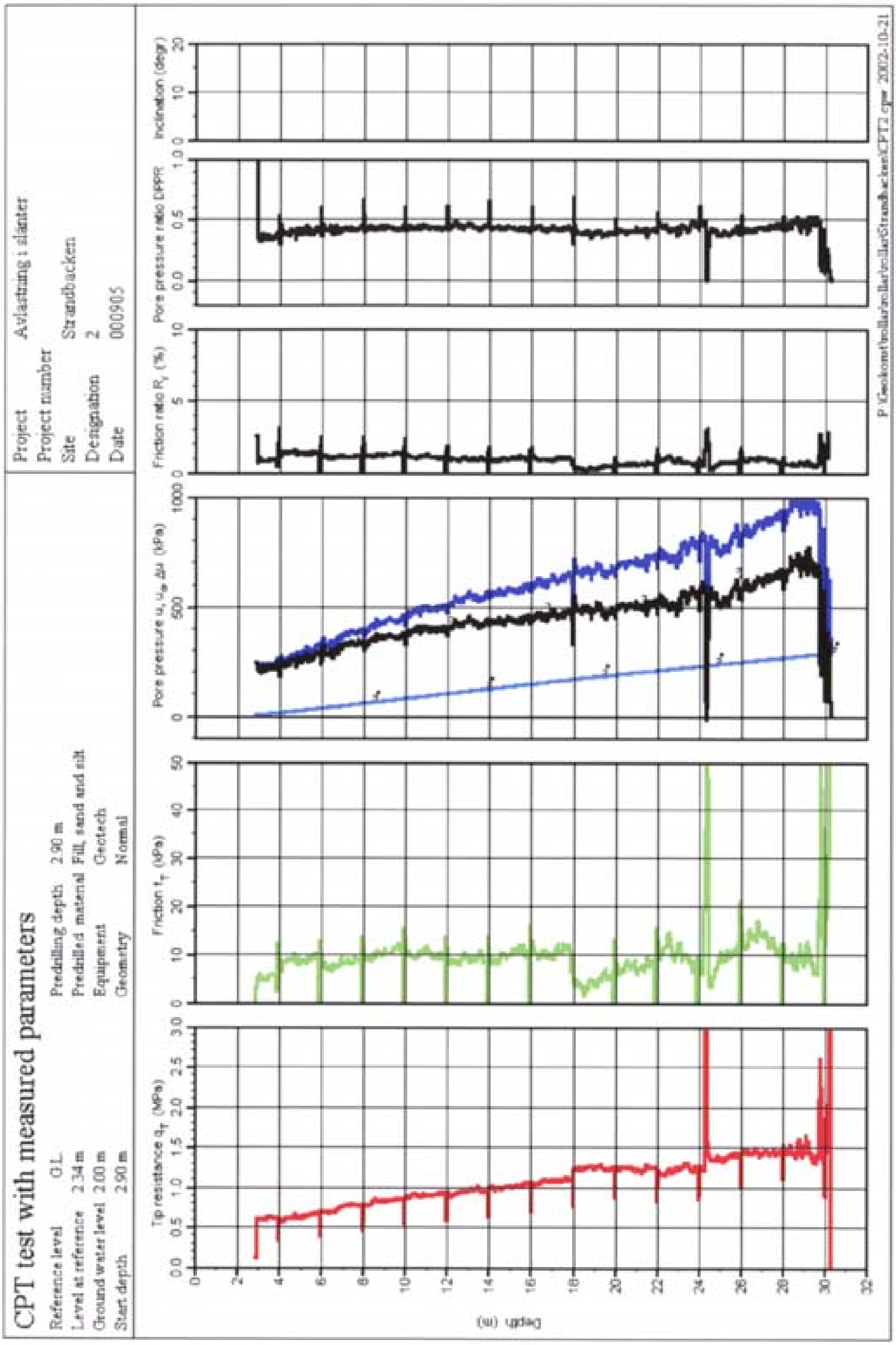


Fig. 90. **Results of the CPT test in Strandbacken presented using the program CONRAD. b) Point 2**

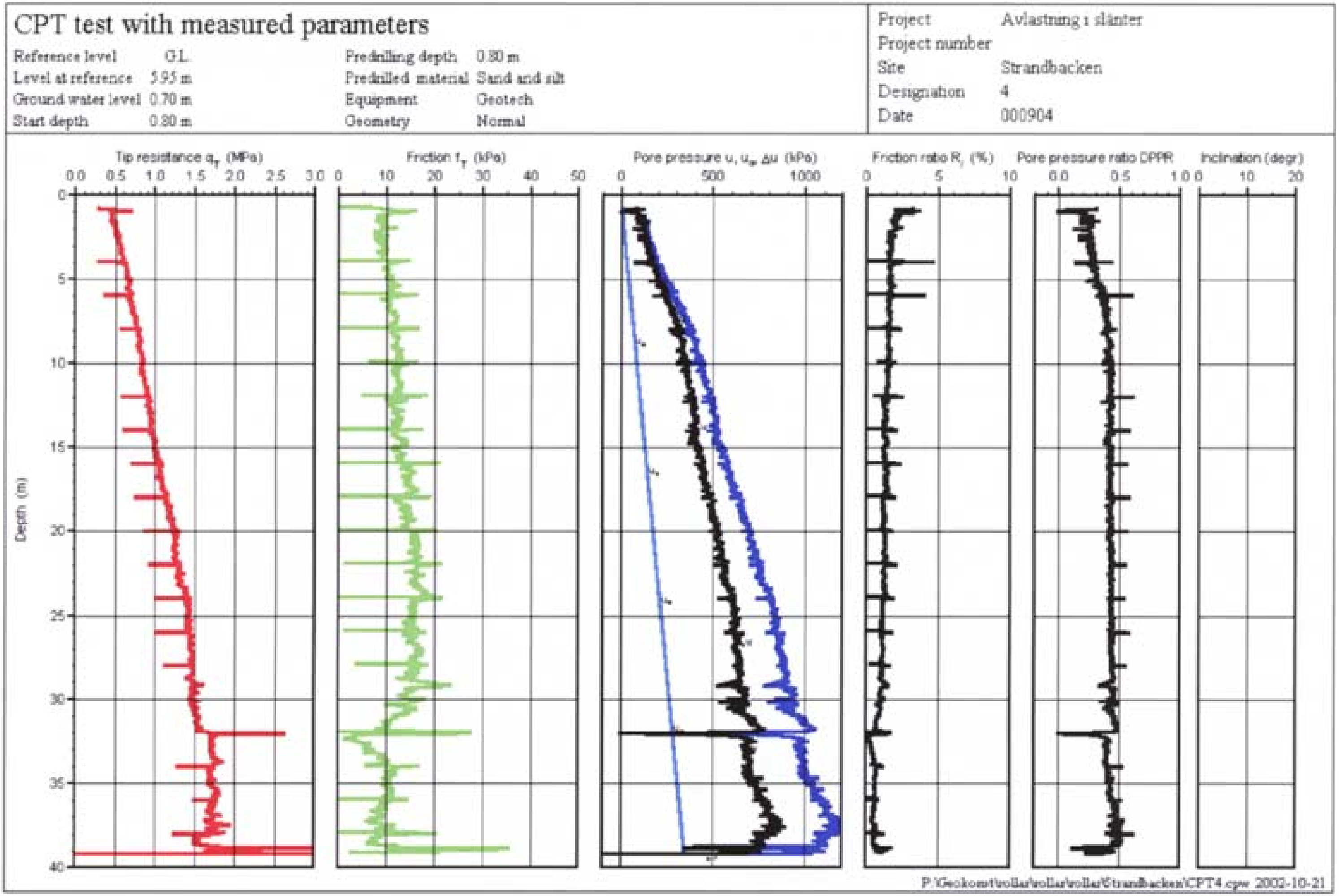


Fig. 90. Results of the CPT test in Strandbacken presented using the program CONRAD. c) Point 4

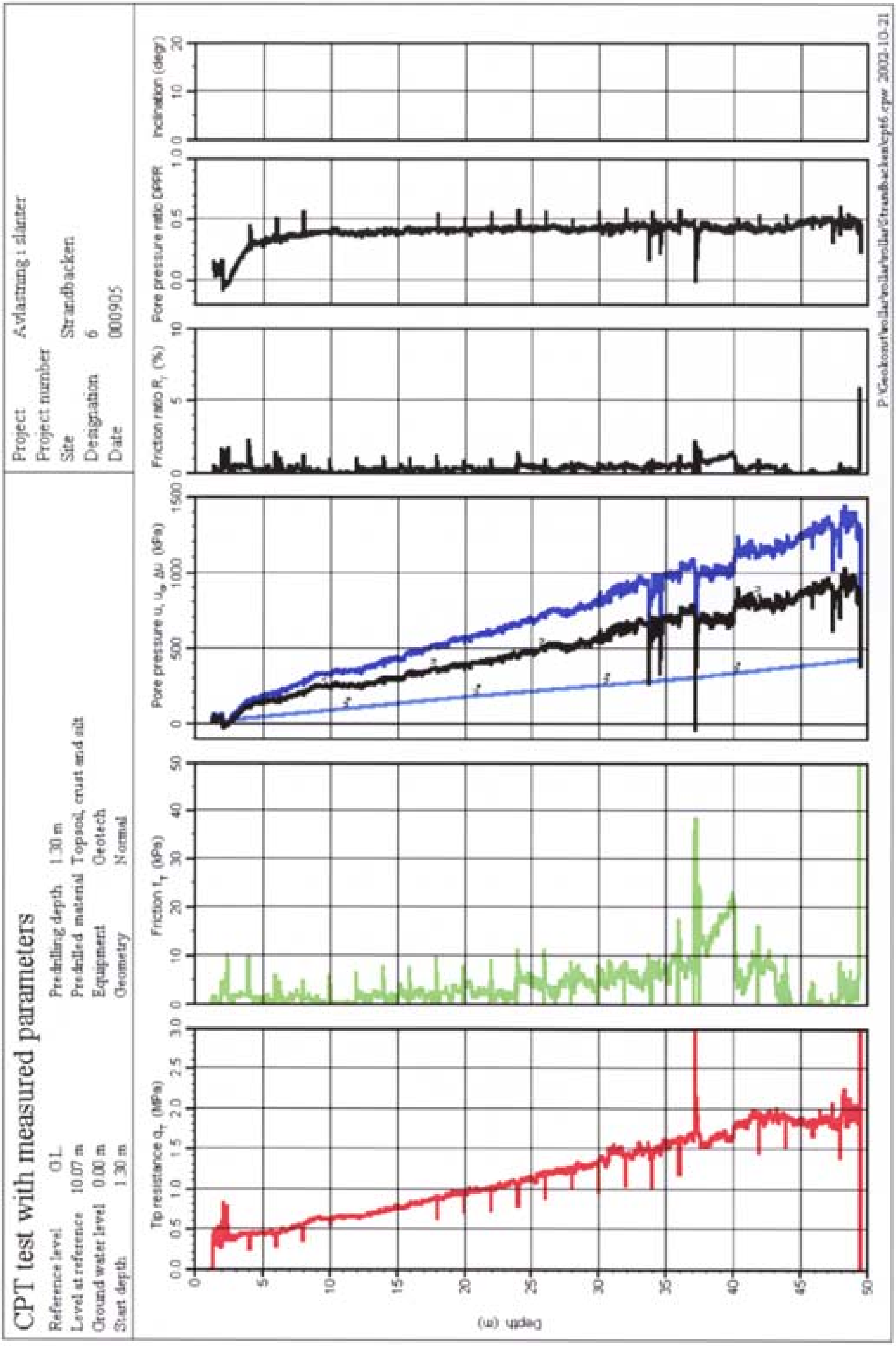


Fig. 90. Results of the CPT test in Strandbacken presented using the program CONRAD. d) Point 6

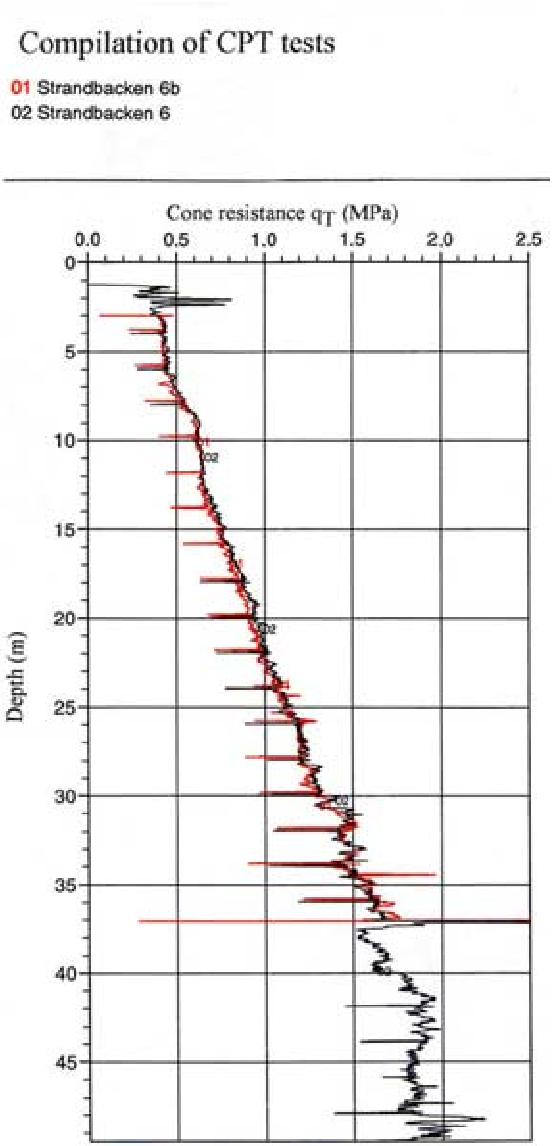


Fig. 91. Comparison between the results of the parallel CPT tests at Point 6.

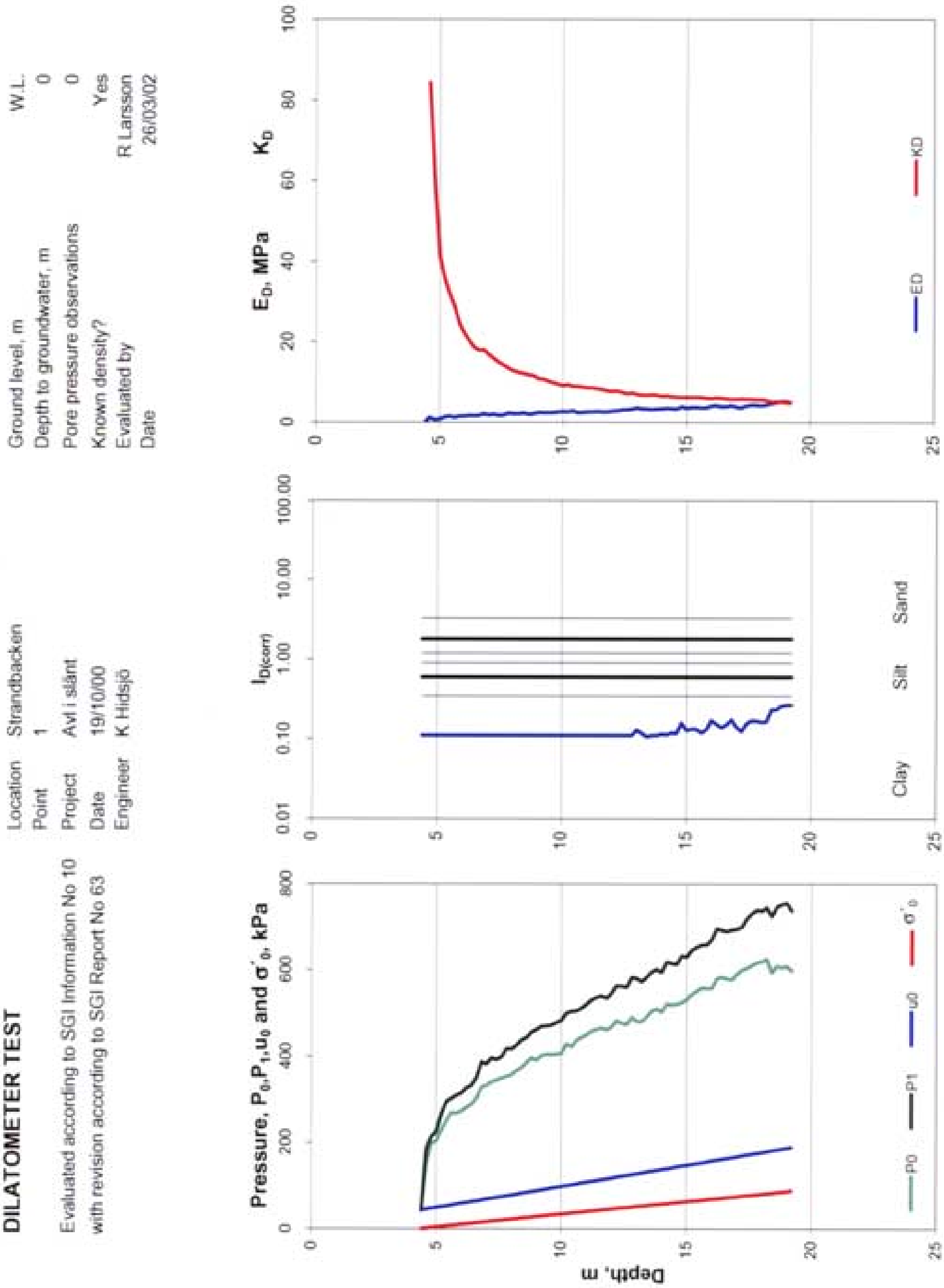


Fig. 92. **Results of the dilatometer tests in Strandbacken.**
a) Results of dilatometer tests at Point 1, base data

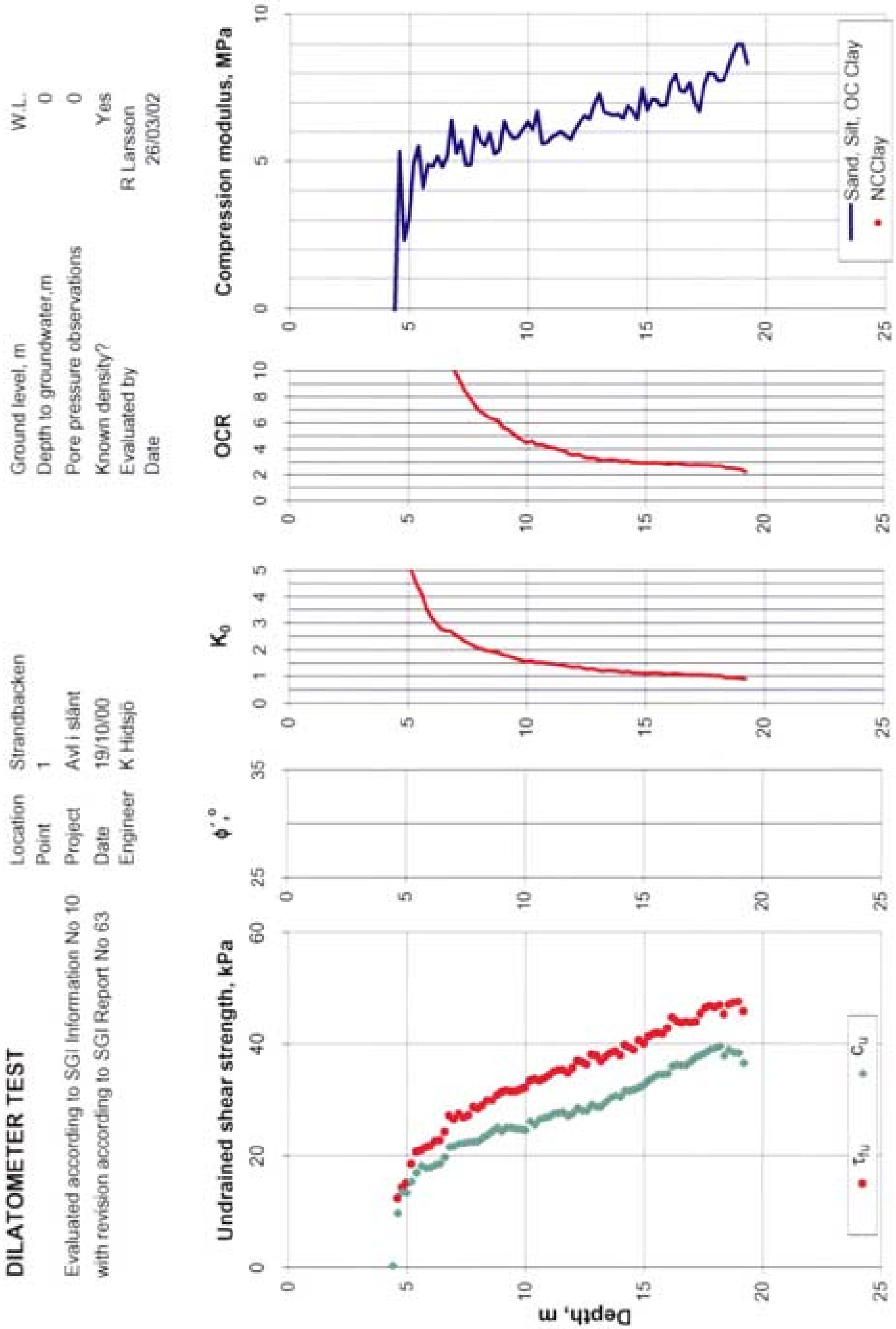


Fig. 92. **Results of the dilatometer tests in Strandbacken.**
b) Results of dilatometer tests at Point 1, evaluated properties (τ_{fu} = undrained shear strength evaluated according to SGI Information No. 10, c_u = undrained shear strength evaluated according to the alternative method).

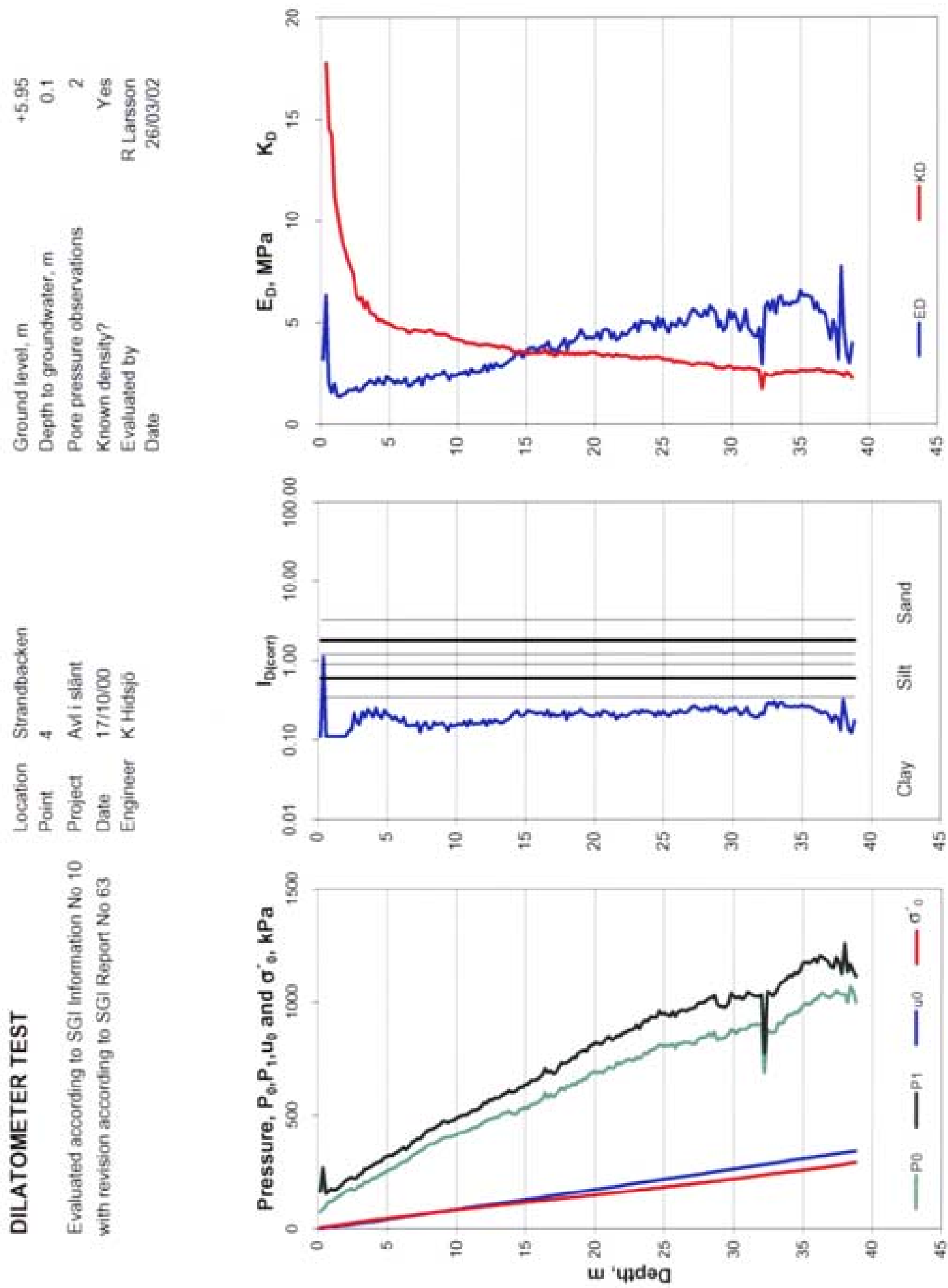


Fig. 92. **Results of the dilatometer tests in Strandbacken.**
c) Results of dilatometer tests at Point 4, base data

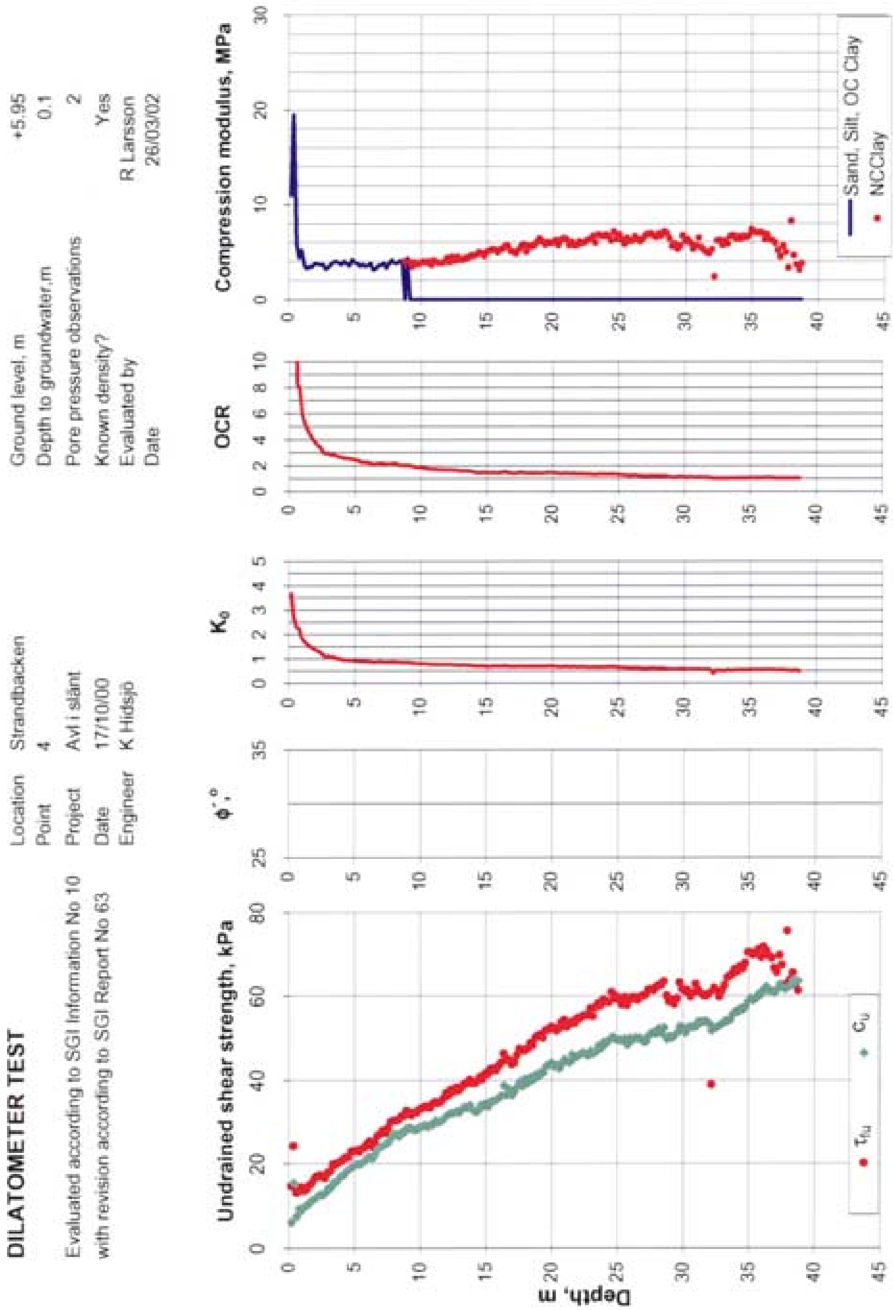


Fig. 92. **Results of the dilatometer tests in Strandbacken.**
d) Results of dilatometer tests at Point 4, evaluated properties (τ_{fu} = undrained shear strength evaluated according to SGI Information No. 10, c_u = undrained shear strength evaluated according to the alternative method).

The field vane tests produced fairly even results although the curves versus depth are slightly serrated because of the content of shells and other coarse material in the clay. The tests in Point 4 were doubled in other to check the repeatability of the results. The test series in each point was started with tests at every metre depth. The sequence was then thinned out to tests at every second metre at larger depths. In the doubled test series in Point 4, the deeper test levels were selected in such way that one test series was performed at every second even metre depth and the other at every second odd metre. In this way a more continuous profile was obtained.

Pore pressure measurements

Closed pore pressure measuring systems of BAT type were installed at Points 2, 3, 5 and 6. The stabilised pore pressures in the bottom layers were also read off in the CPT tests. Three systems were placed in each station. The filter tips in each station were placed in such a way that the uppermost tip was located fairly close to the ground surface but safely below the upper free groundwater level. This was intended to enable a monitoring of the variation of this groundwater level. The lowest tips at Points 5 and 6 were placed at a level of about –27 metres, where according to the CPT test results there might be a continuous permeable layer. The lowest tips at Points 2 and 3 were placed at the same level, which here corresponds to the transition between the lower clay layer and the coarse and firm soil below. The third tip in each station was placed in the central part of the upper clay layer at levels selected from indications in changes in soil properties from the CPT tests.

The pore pressure measuring systems have been read off more or less regularly with the purpose of monitoring seasonal variation and catching extreme values during wet and dry seasons. During the course of the measurements, problems with development of gas have been encountered in some of the tips. These tips have then had to be re-saturated at each measurement after the problem started.

3.7.4 Sampling

Undisturbed samples have been taken with Swedish standard piston sampler type St II at Points 1, 2, 4 and 6. The sampling levels have been selected with guidance from the results of the CPT tests and have in principle comprised the whole penetrated profiles. The material brought up in the cutting edges and spacers has been examined in the field in order to obtain as detailed information about the stratification as possible. Extreme care has been taken when taking, transporting and handling the samples. Disturbed sampling has also been performed to shallow depths at a point at the crest of the lower slope in order to investigate the thickness of the remaining sand and silt layer at this point.

3.7.5 Surveying and levelling

The investigated part of the area has been surveyed and the investigation points have been introduced in the coordinate system used by Lilla Edet municipality. All investigation points and all points where the inclination of the ground surface changes have been levelled in relation to a reference point provided by the municipality. A reference point for the water level in the river has also been selected and levelled.

3.7.6 Laboratory tests

The soil samples have been investigated by routine tests comprising classification, density, water content, liquid limit and undrained shear strength and sensitivity according to fall-cone tests. A large number of CRS oedometer tests have also been performed in order to determine the preconsolidation pressure and the permeability of the soil.

Supplementary determinations of the undrained shear strength have then been made by direct simple shear tests on specimen from selected levels at all sampling points. For two levels at Point 4, test series on specimens with different degrees of unloading have also been performed in order to study how an unloading affects the undrained shear strength according to this type of test.

A number of triaxial tests have also been performed in order to check the validity of the empirical relations for shear strength in clay for this particular soil and to determine the shear strength parameters in the upper sand and silt layer.

3.8 TEST RESULTS

3.8.1 Soil conditions – stratigraphy and variations over the area

The results of the field and laboratory tests showed that the soil conditions within the investigated area at Strandbacken varied with depth and in certain respects also across the area, i.e. perpendicular to the river. The upper soil layers below the original ground surface have consisted in a thin layer of topsoil overlying sand and silt. At Point 6, about 25 metres behind the upper crest, these upper layers reach only about 2.5 metres below the ground surface. At the upper slope, this thickness has increased to at least 4 metres and the entire upper slope consists of this type of soil. The thickness has increased further at the centre of the excavated terrace where the upper 3 metres of the remaining soil is classified as silt. This means that the sand and silt layer here had a thickness of about 7 metres before the excavation. The disturbed samples taken at the crest of the lower slope indicate similar conditions at this point. However, it is not a straightforward task to determine the exact thickness of these layers since the transitions from sand to silt and silty clay with a decreasing silt content are gradual and thin layers of coarser material are found embedded in the clay, particularly in the uppermost part. The soil properties are also affected by the organic content, which is found in the form of embedded plant remains, thin layers of plant remains and more evenly distributed finer particles. The upper soil layers are thereby designated as organic. The organic content is significant within a relatively large depth interval, and infusions of visible plant remains have been found down to 12 metres' depth below the original ground surface. The organic content varies between 2.2 and 3 % within the whole layer of clayey silt and silty clay down to this depth. The upper soil layers also contain various amounts of shells.

At about the same level, about 12 metres below the original ground surface, the character of the soil changes according to both field and laboratory tests. The generation of tip resistance and pore pressure in the CPT tests and the evaluated material index in the dilatometer test indicate that more homogeneous clay is encountered. According to the laboratory tests, it is a high-plastic sulphide spotted clay with some content of shells, which are absent in certain depth intervals and at certain levels form thin layers or lenses of shells. At greater depth, about 30 metres below the original ground surface, the soil becomes coarser and lenses and thin layers of coarse soil start to appear. At the level of about –27 metres, i.e. 37 metres below the original ground surface, an apparently continuous layer of coarser soil is found. At Points 1 and 2, this layer is found at a level of about –22 metres. The tests have penetrated further in all points and samples have also been taken at deeper levels. At those points where the sampling operations have caught the embedded layer, it has been found to consist of a number of alternating thin layers of fine sand and plant remnants.

Below this layer, the clay continues for a number of metres before stop in the soundings is obtained in sand or coarser soil. This clay at the bottom of the profile is coarser and contains single small channels, which have been stated to originate from small organisms that once lived in the bottom mud in the sea. In the deepest clay layers, there is a transition from grey, sulphide spotted clay to brown-grey varved clay with an increasing content of thin silt and sand layers. The results of the CPT tests indicate that the thickness of the clay layers is smaller below the river than in the area behind the riverbank. This could have been a result of the limited penetration force from the unanchored drill rig on the raft, but it is verified by results from earlier experience, where the stop levels for primarily the "machine sounding tests" showed the same bottom configuration. The established borders for different layers and clay types in the profiles also support these conditions, Fig. 93.

Certain anomalies with embedded layers of coarser soil were found in the upper metres of the profile at Point 2. This is assumed to be a result of the previously continuous process of erosion and superficial slides where the superficial soil layers at the toe of the slope consists of slide debris originating from higher levels in the soil profile.

The liquid limit is a measure of the composition and character of the soil, and a compilation of the measured values shows that the soil is very similar throughout the area, Fig. 94, apart from the fact that the thickness of the clay layer decreases below the riverbank and the river and the thickness of the sand and silt layer decreases between Points 4 and 6, i.e. with distance away from the river. The measured values correspond well to those measured in earlier investigations in the area, see Fig. 70.

A corresponding compilation of the measured natural water content in the test points shows a very good correlation between the results at Points 1–4 when the varying thickness of the layers is considered, Fig. 95. However, a considerable difference was obtained at Point 6, where the water contents are considerably higher than in the other points, particularly in the upper soil layers. This is also reflected in the measured bulk densities, which are lower at Point 6, Fig. 96.

The relation between the water content and the liquid limit in a soil is a measure of its state of consolidation and also affects properties such as remoulded shear strength and sensitivity. The quasi liquidity index that is represented by the quotient w_N/w_L shows clearly that the upper part of the soil profile at Point 6 differs from the soil at the other points in this respect, Fig. 97.

That the undrained shear strength was lower at Point 6 had been shown already in the CPT tests. A closer study of the measured sensitivities in the laboratory showed that these were significantly higher in the upper part of the soil profile at Point 6, Fig. 98.

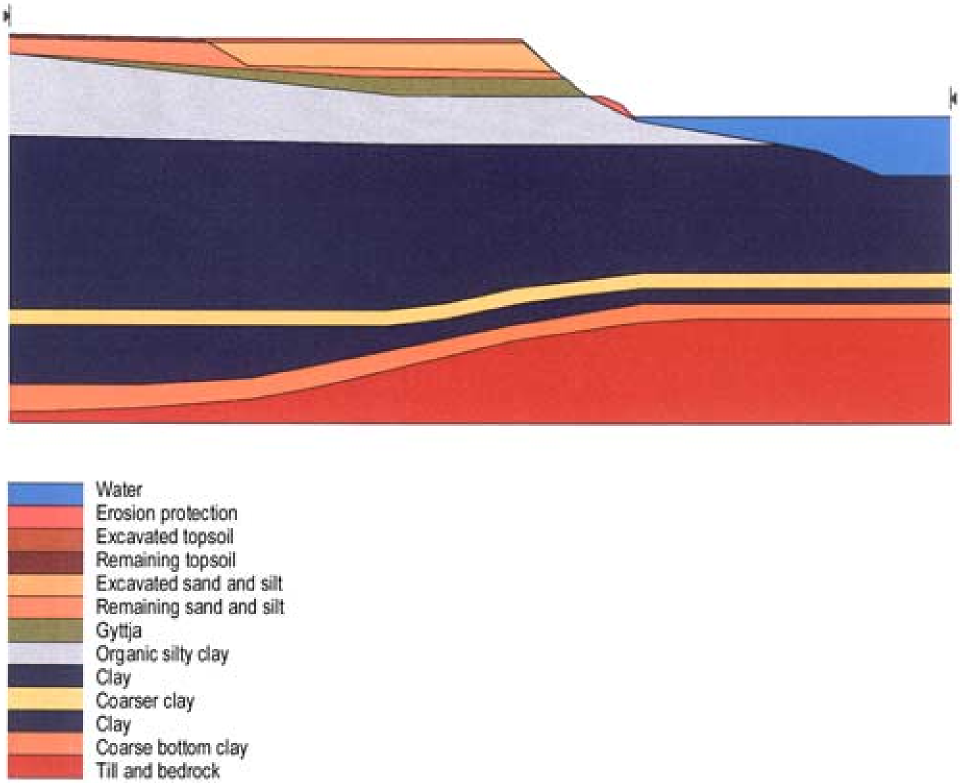


Fig. 93. Variation in stratigraphy at Strandbacken with distance from the river. (The different layers are described in more detail in the text. Different colours have been used in the upper layers of topsoil, sand and silt to illustrate the extent of the excavation.)

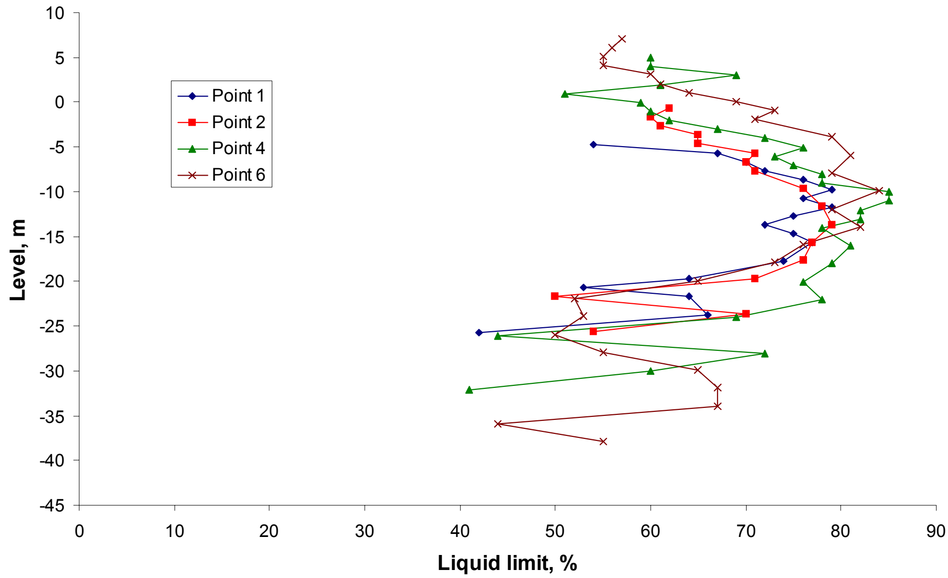


Fig. 94. **Compilation of measured liquid limits in the new test points at Strandbacken.**

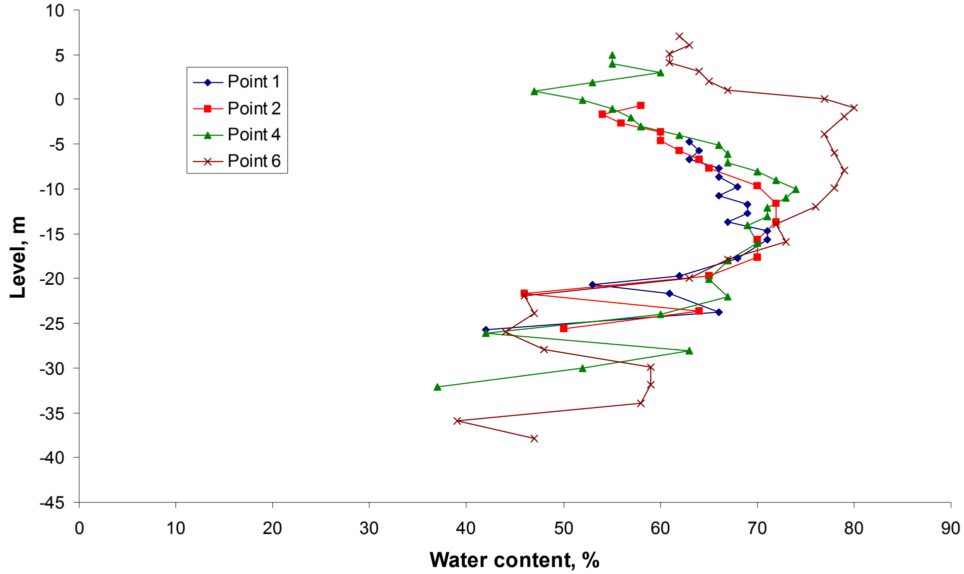


Fig. 95. **Compilation of measured water contents at the new test points at Strandbacken.**

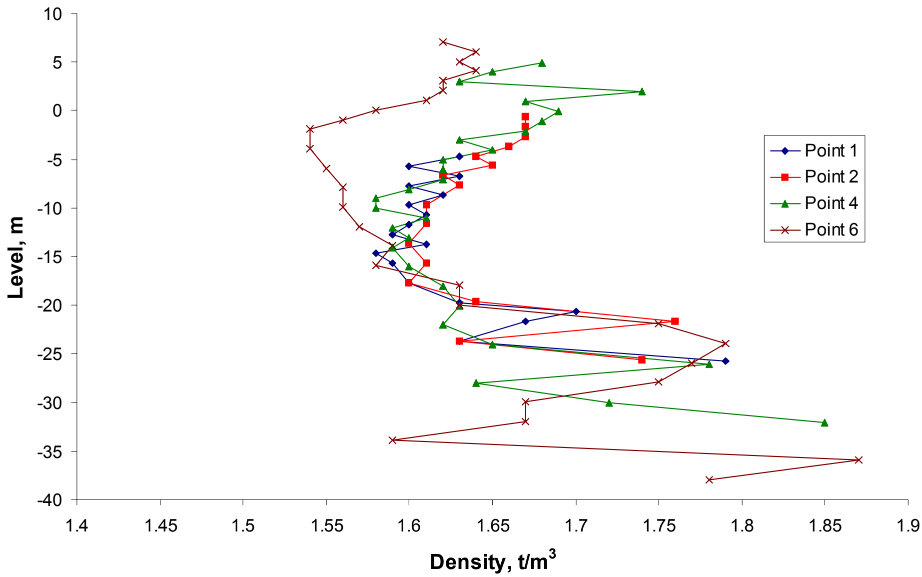


Fig. 96. **Compilation of measured bulk densities at the new test points at Strandbacken.**

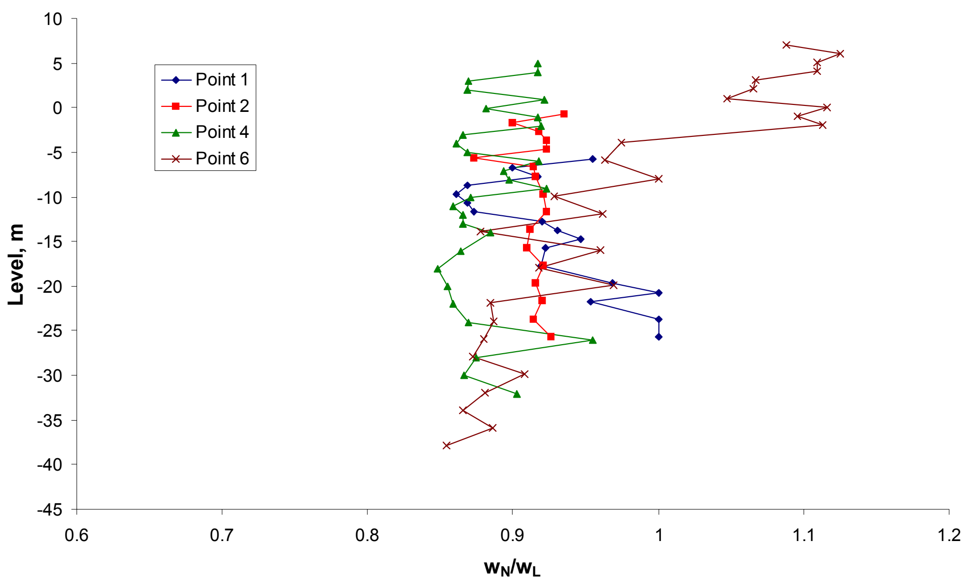


Fig. 97. **Quasi liquidity index at the new test points at Strandbacken.**

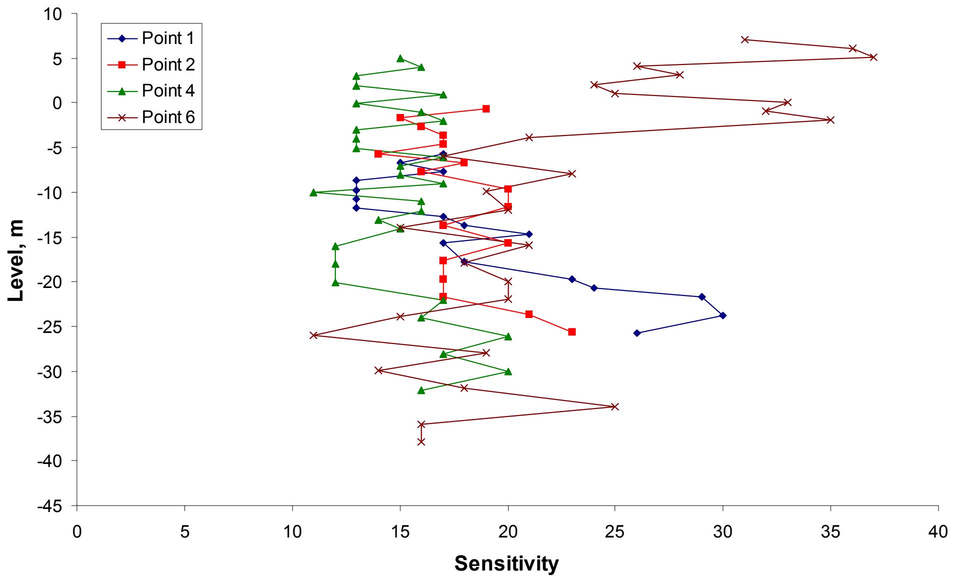


Fig. 98. **Compilation of measured sensitivities at the new test points at Strandbacken.**

Higher quasi liquidity indices and sensitivities were also measured in the bottom layers at Point 1, but this can be assumed to be related to the fact that these determinations have been made in the coarser varved and layered soil just above the underlying coarse bottom layers.

The relation between quasi liquidity index and sensitivity in principle follows the relation that was found in the Göta-älv investigation, Fig. 99. The sensitivity is thereby high in the upper soil layers at Point 6. The maximum quasi liquidity index was 1.1 and there was no quick clay within the investigated area. However, the previous investigations have shown that the clay becomes quick at further distance from the river.

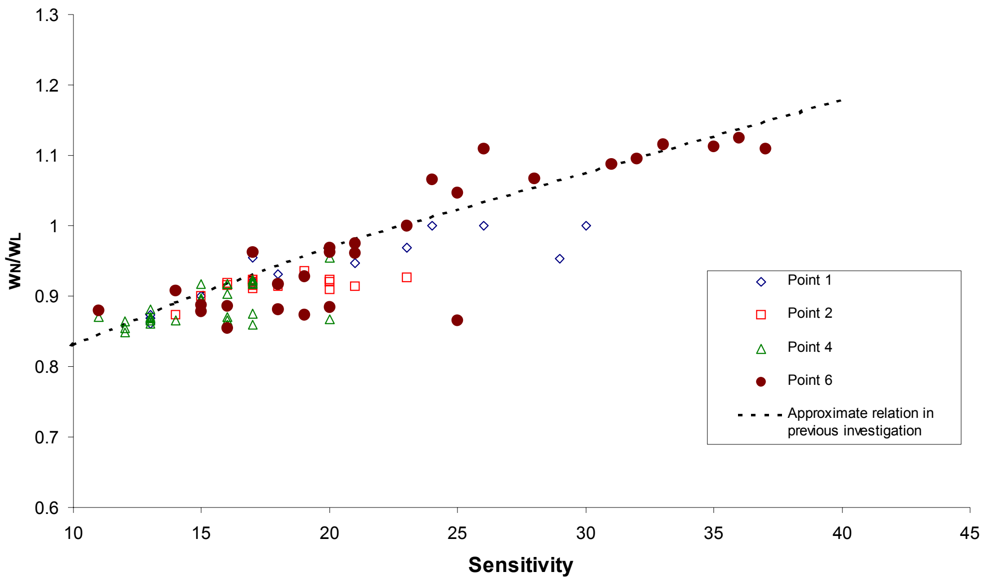


Fig. 99. Relation between quasi liquidity index and sensitivity at the new test points at Strandbacken.

3.8.2 Permeability and pore water pressure

The permeability of the soil has been evaluated from the results of the CRS oedometer tests, Fig, 100. The results scatter, particularly on those levels where there are embedded layers of different types of soils, but there is a clear trend for the variation. The permeability is thus high in the upper sand and silt layers and decreases gradually with depth down to a level of about –4 metres. No tests have been performed in the layers consisting solely of sand and silt, and in the layers with clayey silt and silty clay the tests have been run on specimens from the more fine-grained parts. The permeabilities measured in this way decrease from about $2 \cdot 10^{-9}$ to about $3.3 \cdot 10^{-10}$ at a level of –4 metres. The permeability then remains more or less constant down to the embedded coarser layer at level –27 metres (–22 metres below the river), and then drops to about $2 \cdot 10^{-10}$ in the clay below. The higher water content in the upper layers at Point 6 would have been expected to entail a higher permeability than at the other points, but no such effect can be detected in the measured values.

The pore water pressures have been measured during different seasons and have been found to vary somewhat, Fig. 101. In general, the variation is moderate. The pore pressure systems at the riverbank are installed 4, 18 and 30 metres below the

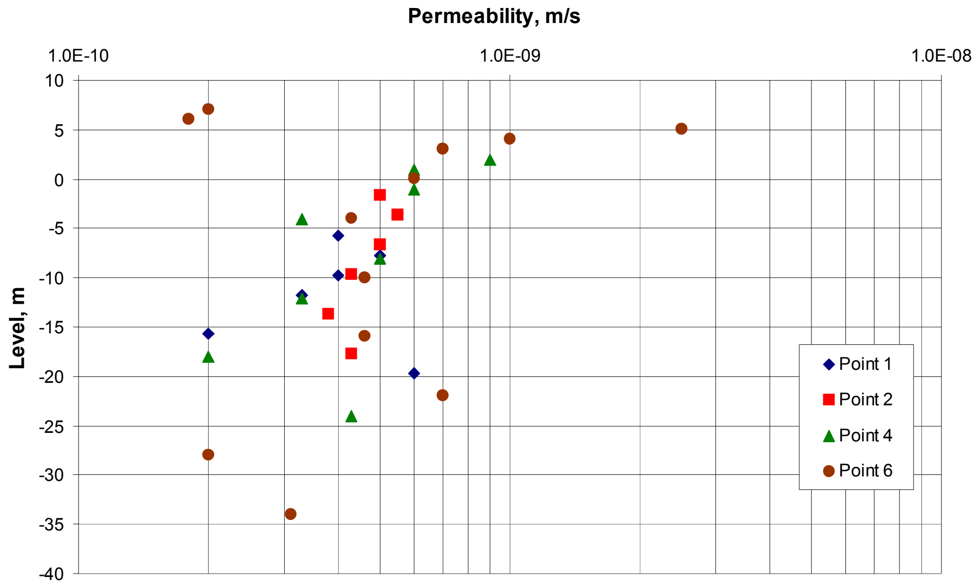


Fig. 100. **Evaluated permeability from CRS oedometer tests.**

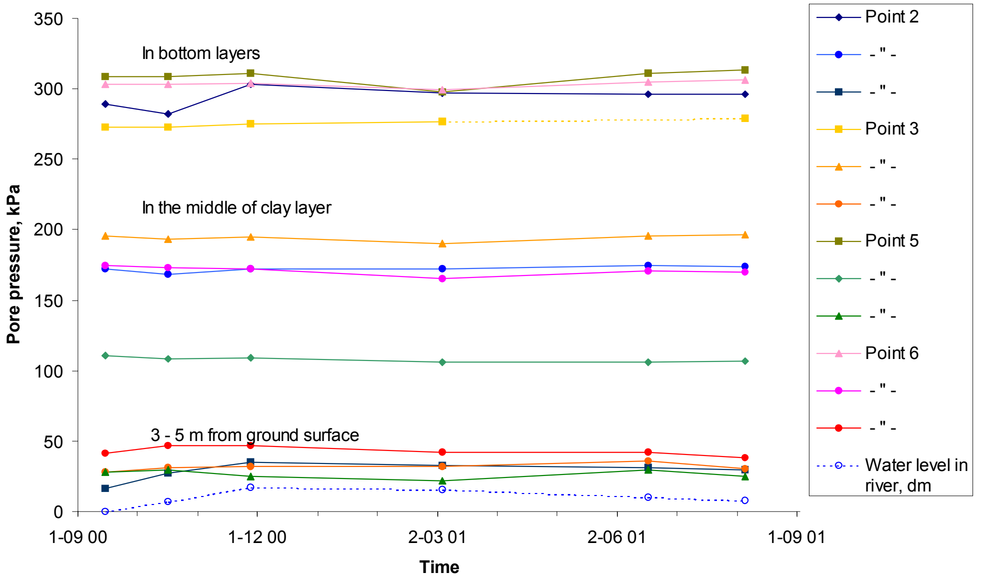


Fig. 101. **Measured variations in pore pressure and corresponding water levels in the river.**

ground surface. The largest measured variations have occurred at this point, which is probably because both the upper filter tip and the lowest tip installed in the coarse bottom layer appear to be in almost direct contact with the water level in the river through coarser soil. The contact for the upper tip is probably created by the erosion protection, coarser fill on top and upper layers of sand and silt that have slid down from the crest. The contact for the lower filter tip is probably created through the coarser bottom layer, which a few hundred metres to the north constitutes the river bottom and is thus in direct contact with the river water there. The variations in these tips consequently almost directly followed the variations in the water level in the river. For the tip at the centre of the clay layer, there are corresponding variations but the response is slower and the amplitudes are smaller. The average pressure at this point corresponds roughly to a hydrostatic pressure from the mean water level in the river, Fig. 102.

In the measuring station behind the crest of the lower slope, i.e. Point 3, the pore pressure in the upper filter tip corresponded to a free groundwater level at a depth of 1.5–2 metres below the ground surface. The lowest filter tip is placed at about the same level as the lowest tip in Point 2 and the measured pressure is approximately the same. Whether or not the variation corresponds directly to the variations in

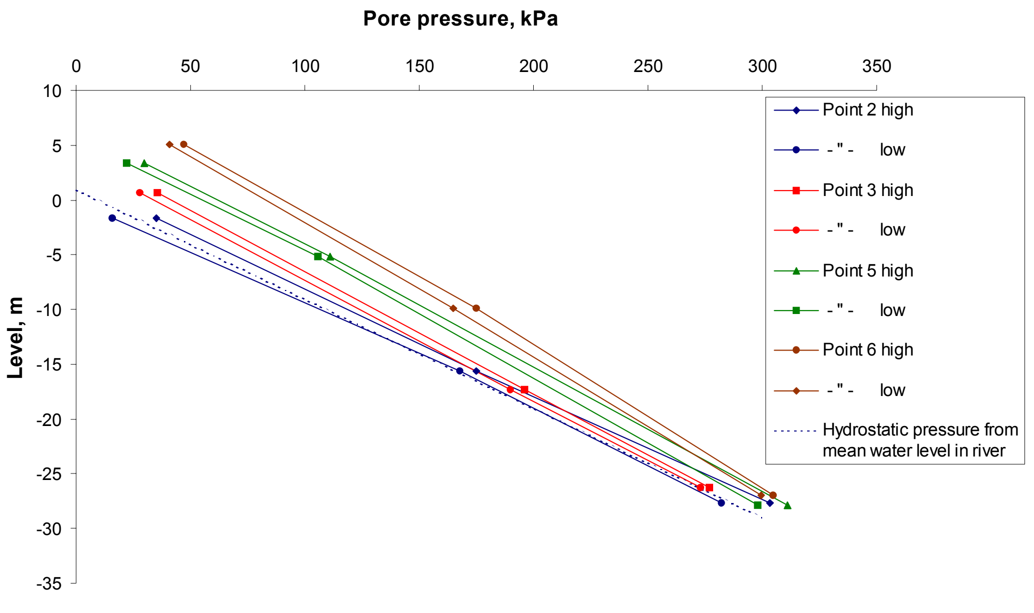


Fig. 102. **Measured pore pressure distributions at the different measuring points. a) pore water pressure**

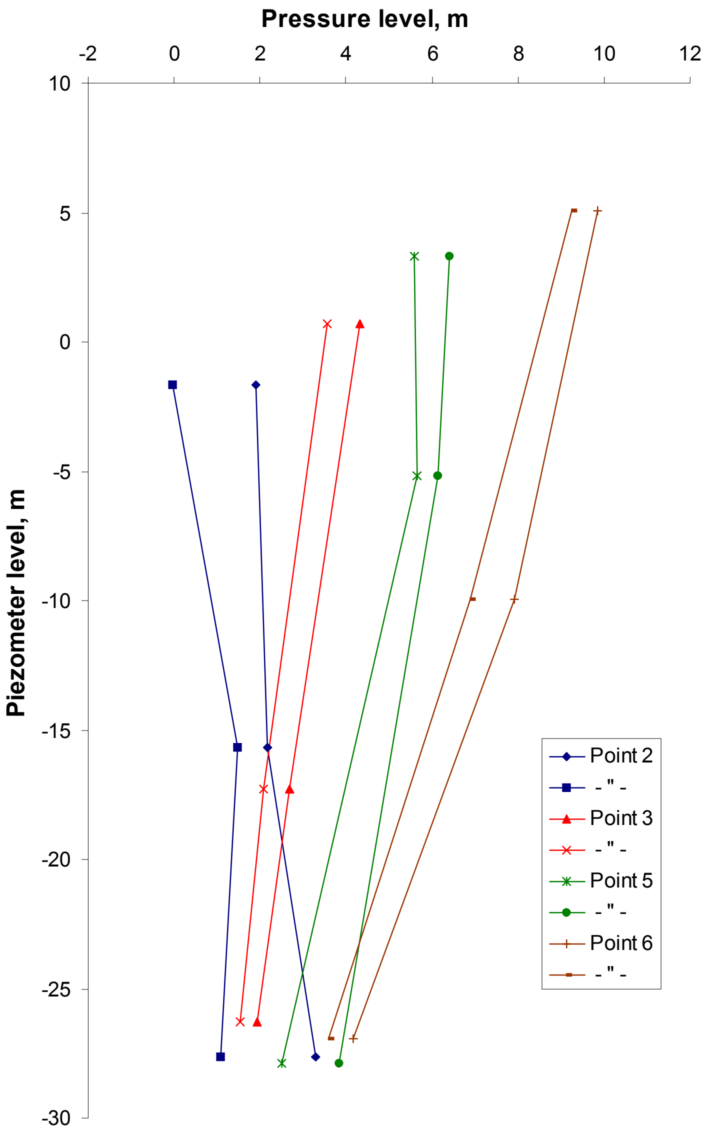


Fig. 102. Measured pore pressure distributions at the different measuring points. b) pressure levels

water pressure in the river in this point too is more difficult to judge because problems with development of gas started shortly after the installation of this tip. More exact and continuous measurements were thereby rendered difficult. However, the measured variations in the tip in the thick clay layer, which is here installed closer to the bottom layers, indicate that the variation in the water level in the river is reflected. The pore pressure distribution at this point is approximately linear with a slight downward gradient.

The free groundwater level in the boreholes at the middle of the excavated terrace has been observed to vary between 0 and 1 metre below the ground surface. In the measuring station at Point 5 at the rear of the terrace, the free groundwater level varies between 0 and 0.5 metres below the ground surface according to the measurements. Free water can normally be observed on the ground at this point. The lowest filter tip is placed in the coarse embedded layer. A pore pressure that is about 20 kPa higher than in the same layer down at the river is measured here, which indicates that there is a significant resistance to water flow in the thin sand and silt layers within this zone. The pore pressures in the profile are thereby mainly regulated by the pressure in the coarse bottom layers about 10 metres further down, which may be assumed to be in fairly good contact with the water in the river. However, the pore pressures in the embedded layer at level –27 metres are approximately the same at Points 5 and 6, which indicates that this layer too has a certain effect on the pore pressure distribution in the soil mass. At Point 6, which is located in natural ground behind the excavated area, the upper free groundwater level varies from being at the ground surface to 1 metre below this. The pore pressure distributions at Points 5 and 6 have shapes that indicate that they are affected by the higher permeability in the upper soil layers. The higher pore pressures in the soil below the higher ground just behind Point 5 can also be assumed to have a certain effect on the pore pressures at this point.

The pore pressures at the upper levels in natural ground and on the excavated terrace are affected by climatic variations. The measured variations in the bottom layers and at the middle of the clay layer are too small for any conclusions to be drawn about the influence of climatic conditions and water level in the river. However, the mean pore pressure in the coarse bottom layers may be assumed to be related to the mean water level in the river.

3.8.3 Stress history and current stress conditions

The stress history of the soil has been estimated from what is known about the geological history of the area and from evaluated preconsolidation pressures from a large number of CRS oedometer tests on specimens from all test points where sampling has been performed. From the geological history, it may be assumed that the area was once very flat with a groundwater level located at the ground surface. The soil has then consolidated for the self-weight of the overlying soil masses and possibly also obtained a small overconsolidation because of creep effects. Thereafter, the erosion process has been going on, involving a decrease in overburden pressure under the river and the eroded slopes. At the same time, the effective stresses have increased behind the crests of the slopes because of the lowered groundwater levels and the lowered water level in the river, which has brought lower pore pressures throughout the profiles. The largest increases in effective stress may be assumed to have occurred in those areas where the sand and silt layers are thick and the distances to the crest also have been short and the lowering of the groundwater level consequently has been large. In the Strandbacken area, this corresponds to the now excavated area. It should be borne in mind that this process has been gradual, starting with a very narrow and shallow river-course, which has become successively wider and deeper. It is difficult to estimate whether the previous development of the area with small buildings and accompanying drainage systems has had any effect on the remaining soil masses, but this is probably negligible.

The excavation performed has entailed that the vertical stresses have decreased and probably that any ongoing creep processes have been stopped or retarded. It is uncertain whether the pore pressures below the excavated terrace have been significantly affected since it was mainly sand and coarse silt that was taken away. Most of the previous pore pressure observations were made in open systems during a short period of time and their accuracy is uncertain. Some of the observations of upward gradients that were made seem very questionable in the light of what has later been measured. The excavation can with certainty be expected to have brought a slight lowering of the pore pressures in the area directly behind the upper excavated slope. This has entailed a corresponding increase in effective overburden pressure, for which only very little adaptation in terms of consolidation and following creep can have occurred.

From the geological perspective, it can thus be assumed that the soil below the centre of the river-course has consolidated for stresses corresponding to the same ground level as in the surrounding areas above the eroded slopes, the same stratigraphy and a groundwater level approximately at this ground surface. The soil closer to the riverbank can be expected to have consolidated for somewhat higher stresses since the groundwater level and the pore pressures may be expected to have become somewhat lower before the approaching erosion slope entailed that the overburden pressure decreased. The corresponding or somewhat enhanced preconsolidation pressures may be expected behind the crest of the slope since the effective stresses here have increased because of even lower pore pressures and that the stresses have been applied for a longer period of time. The groundwater level in this area can be expected to have been close to the lower boundary of the layers containing only sand and silt. Further away from the river, the soil can be expected to have consolidated for the current stresses. In the area just behind the upper slope above the excavated terrace, the effective stresses have recently increased somewhat as a result of a lowered groundwater table and the overconsolidation due to creep effects that is normally found therefore does not necessarily exist.

The results of the oedometer tests generally support this picture. The evaluated preconsolidation pressures below the river bed at Point 1 show that the soil has consolidated for the assumed maximum effective stresses at the centre of the river bed and has an overconsolidation ratio of about 1.2 in relation to these, Fig. 103. On the other hand, the preconsolidation pressures are still lower than the assumed maximum stresses behind the former crest of the slope. The unloading due to the erosion has entailed that the soil below the river bed is heavily overconsolidated in relation to the stresses that prevail today.

The evaluated preconsolidation pressures at Point 2 at the riverbank are somewhat higher than at Point 1, Fig. 104. They correspond approximately to the assumed maximum effective stresses behind the former crest without any significant overconsolidation. Due to the erosion, the soil is overconsolidated at this point.

The measured preconsolidation pressures at Point 4 at the middle of the excavated terrace correspond to the assumed maximum effective stresses with a certain overconsolidation in relation to these, Fig. 105. The overconsolidation ratio is somewhat lower than would be expected from empirical experience (Larsson and Sällfors 1995). This may be due to an assumption of a too low ground water level and/or that the period of time during which these maximum stresses have acted is relatively short from a geological perspective. The present effective stresses after the excavation are so low that the overconsolidation ratio has become sufficiently large for any ongoing creep settlements to stop according to empirical experience.

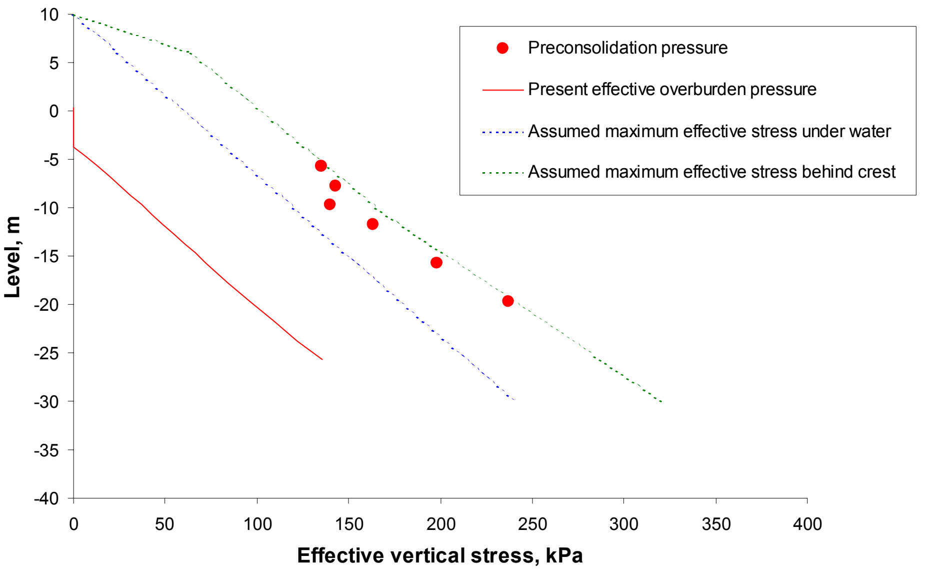


Fig. 103. Preconsolidation pressure from CRS oedometer tests and assumed maximum stresses at Point 1 below the river.

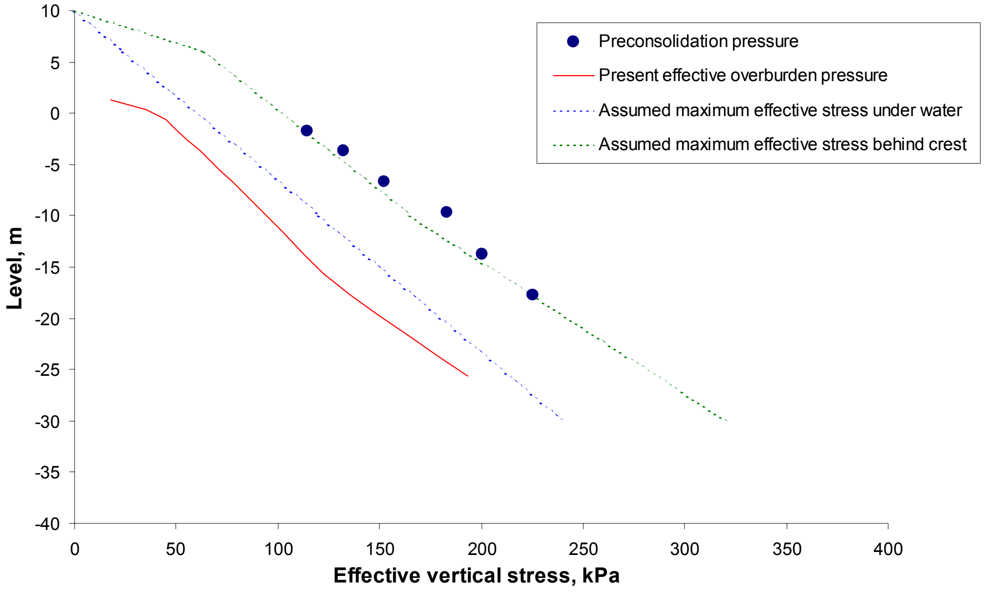


Fig. 104. Preconsolidation pressure from CRS oedometer tests and assumed maximum stresses in Point 2 at the riverbank.

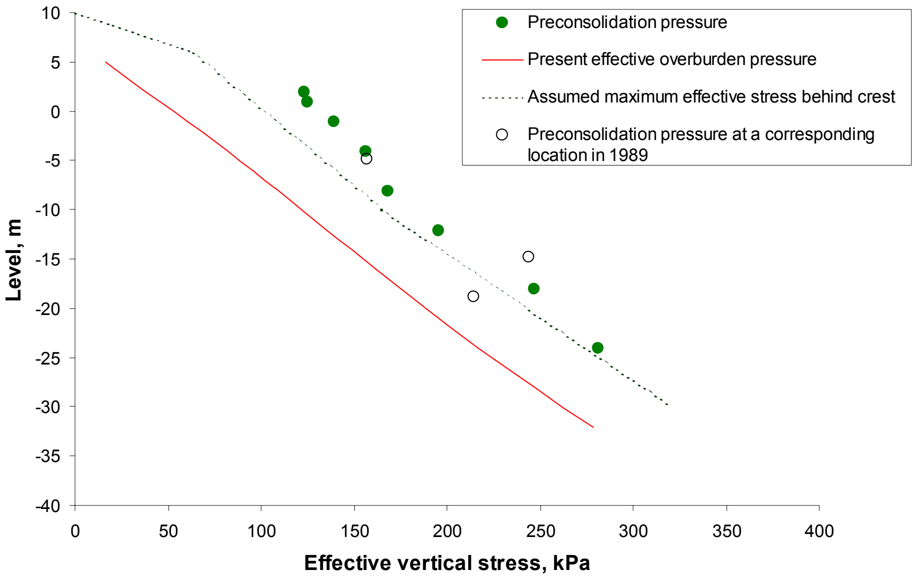


Fig. 105. Preconsolidation pressure from CRS oedometer tests and assumed maximum stresses at Point 4 below the excavated terrace.

Finally, at Point 6 behind the upper crest the preconsolidation pressures are lower than the assumed maximum stresses behind the former crest, where the upper sand and silt layers were considerably thicker and the ground water level was lower, Fig. 106. The measured preconsolidation pressures are only marginally higher than the current effective vertical stresses. The samples from the levels –22 and –28 metres may have been somewhat disturbed because of embedded layers and coarser particles. If these results are neglected, the overconsolidation ratio is about 1.1, which is less than would have been expected from empirical experience. This may be related to the fact that the current stresses have only acted during a relatively short period of time from a geological point of view. This in turn is related to the fact that the water level in the river and the pore pressure in the bottom layers have been lowered gradually in relation to the surrounding ground and possibly to the fact that the upper groundwater level has been somewhat lowered after the excavation. Significant changes in properties because of leaching or other changes in the pore water chemistry are unlikely since no indications of this can be observed in the determined liquid limits.

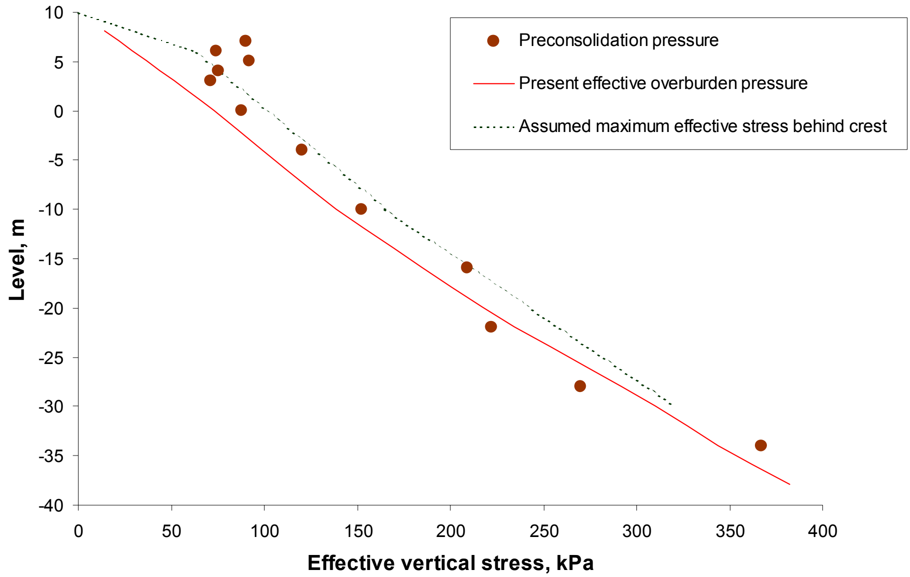


Fig. 106. Preconsolidation pressures from CRS oedometer tests and assumed maximum stresses at Point 6 behind the upper crest.

The results of the oedometer tests thus provided a consistent picture, and the variations that were found between the different points are logical and can be explained by local differences in the stress history and variations in the thickness of the upper sand and silt layers.

A measure of the preconsolidation pressure is also obtained from the results of dilatometer tests and CPT tests. The method of evaluation of preconsolidation pressures from dilatometer tests in clay that has hitherto been commonly used in Sweden is based on results from normally consolidated and only slightly overconsolidated clays, with the assumption that more overconsolidated clays consist of fissured crusts (Larsson and Eskilsson 1988).

The equation used with these assumptions is

$$OCR = 10^{0.16(K_D - 2.5)}$$

Where K_D = horizontal stress index – a base parameter evaluated from the dilatometer test results.

These assumptions are not valid in soils that have been unloaded, for example by erosion or excavation. For non-fissured overconsolidated British clays, Powell and Uglow (1988) proposed an evaluation of the preconsolidation pressure according to

$$OCR = 0.24 {K_D}^{1.32}$$

This equation has previously been found to give reasonable estimates also in heavily overconsolidated clay till in Sweden (Larsson 2001). However, it does not work for normally consolidated and only slightly overconsolidated Swedish clays, in which the overconsolidation ratio becomes underestimated. Based on the results in this and other recent projects, it is therefore proposed to use a hybrid of these formulas with a gradual transition from the first to the second according to

$$\begin{aligned} &OCR = 10^{0.16(K_D - 2,5)} && K_D \leq 5 \\ &OCR = 2.51 + 0.368(K_D - 5) && 5 > K_D \geq 7.5 \\ &OCR = 0.24 {K_D}^{1.32} && K_D > 7.5 \end{aligned}$$

The overconsolidation ratios estimated in this way are shown in Fig. 107, where the decreasing effects with depth of the erosion and the excavation are illustrated.

The evaluated overconsolidation ratios can be used to calculate the preconsolidation pressures. This method provides a good estimate of the preconsolidation pressures both for the heavily overconsolidated clay below the river bottom and for the only slightly overconsolidated clay below the excavated terrace, Figs. 108a and c. Use of only the first equation would have resulted in much too high values in the heavily overconsolidated clay whereas use of only the last equation would have shown underconsolidated clay at larger depths at Point 4.

An evaluation of the preconsolidation pressure from the CPT tests is somewhat more uncertain because of the sensitivity of the method to the type of soil represented by the liquid limit. As with the findings in Munkedal, much too high values were evaluated by the previous method with correction for the overconsolidation ratio. More relevant results, but with a somewhat larger scatter than for the dilatometer tests, were obtained when this correction was omitted, Figs. 108 a-d.

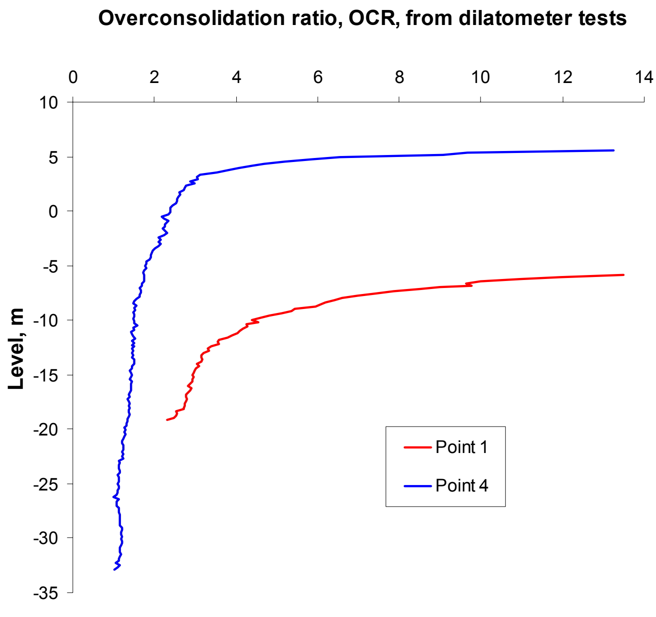
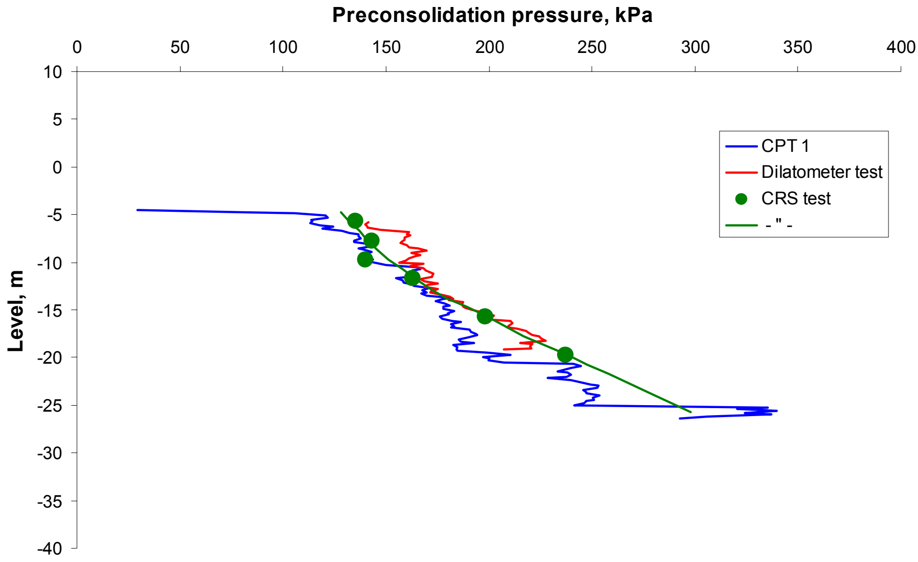
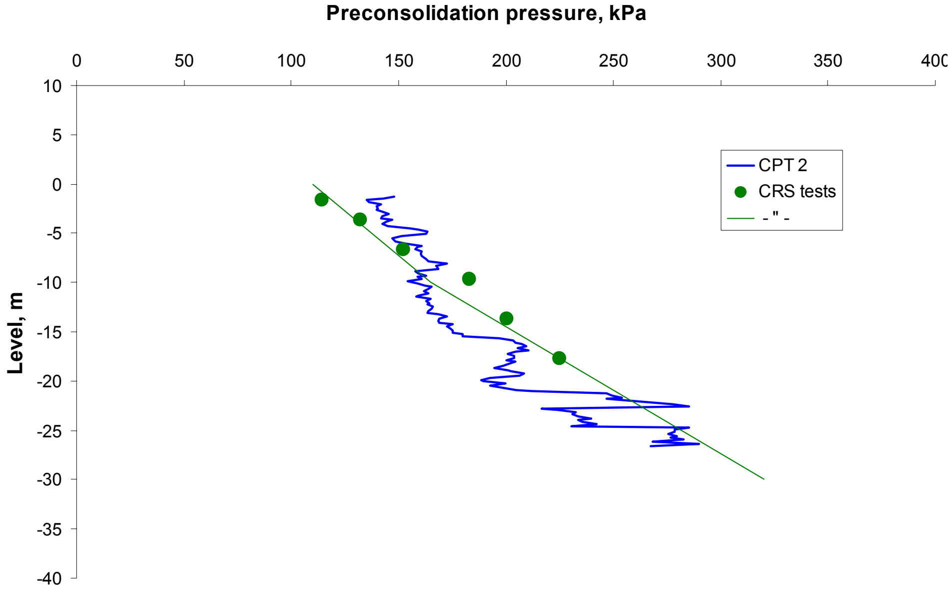


Fig. 107. **Evaluated overconsolidation ratio from dilatometer tests in Points 1 and 4.**

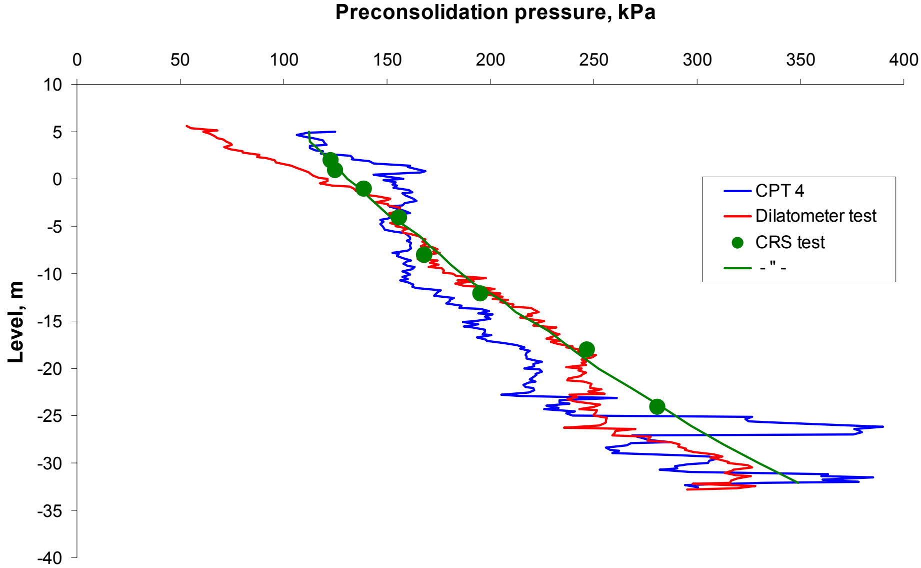


a)

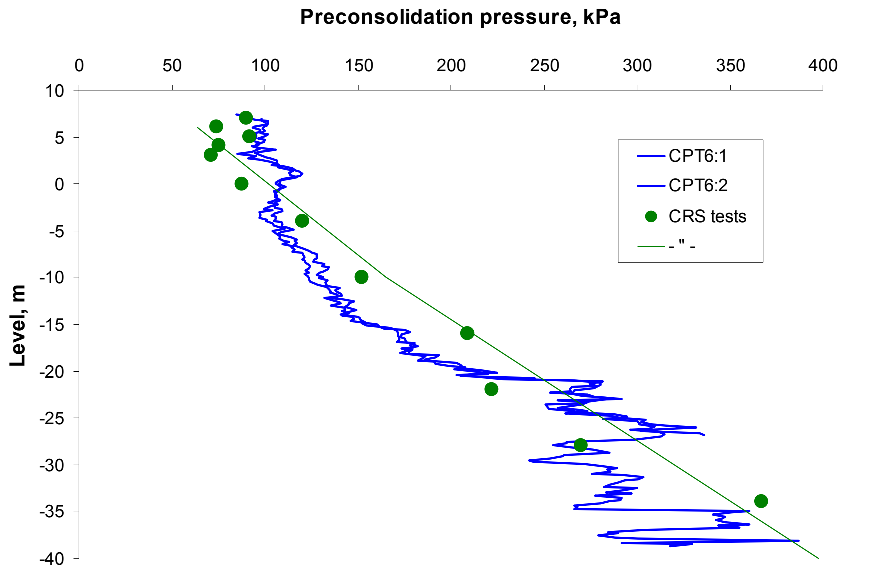


b)

Fig. 108. **Preconsolidation pressures evaluated from dilatometer tests and CPT tests in Strandbacken.**
a) Preconsolidation pressures evaluated from dilatometer tests and CPT test at Point 1.
b) Preconsolidation pressures evaluated from CPT test at Point 2



c)



d)

Fig. 108. **Preconsolidation pressures evaluated from dilatometer tests and CPT tests in Strandbacken.**
c) Preconsolidation pressures evaluated from dilatometer tests and CPT test at Point 4
d) Preconsolidation pressures evaluated from CPT test at Point 6.

The coefficient of earth pressure, K_0, is also evaluated from the dilatometer tests. Fig. 109 shows how this evaluated coefficient decreases with depth in the profiles from maximum values around 4 in the superficial heavily overconsolidated soil layers to approach values around 0.5 in the deepest soil layers below the excavated terrace. The latter value is fairly typical of normally consolidated clay. The K_0 values in Point 1 are probably also affected by the fact that this point is located in a passive zone where the horizontal pressures are elevated more than results from the overconsolidation only. However, as may also be expected, the calculated horizontal stresses below the river are still smaller than those at the same levels below the excavated terrace are, Fig. 110.

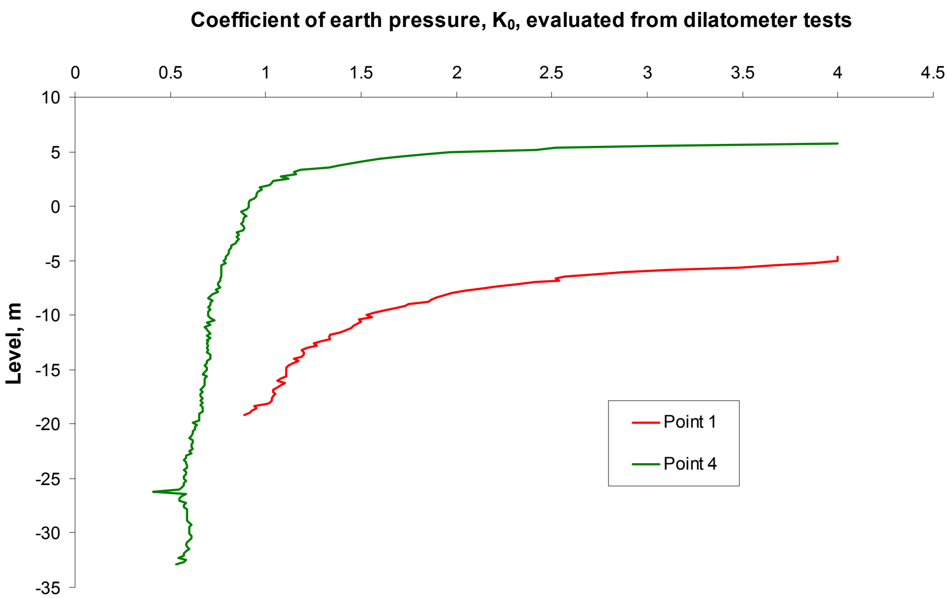


Fig. 109. **Evaluated coefficient of earth pressure from dilatometer tests at Points 1 and 4.**

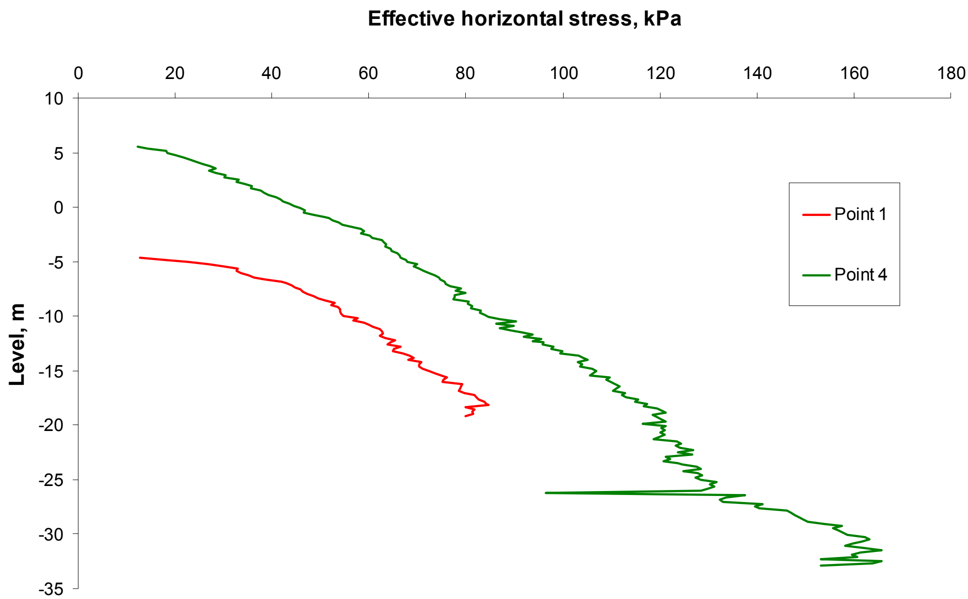


Fig. 110. **Evaluated effective horizontal stresses from dilatometer tests at Points 1 and 4.**

3.8.4 Shear strength

The undrained shear strength has been determined by field vane tests, CPT tests and dilatometer tests in the field and by direct simple shear tests, triaxial tests and fall-cone tests in the laboratory. An unusually large spread in results between the different methods was obtained, particularly in the upper soil layers at Points 4 and 6, where the soil consists of silty clay with organic matter.

Field vane tests, CPT tests and fall-cone tests were performed at all four main test points and dilatometer tests were performed at Points 1 and 4. Supplementary direct simple shear tests were then performed at selected levels at all main test points. Series of tests with different degrees of unloading were also performed at two levels at Point 4 in order to determine more accurately the effect of unloading on the undrained shear strength. Supplementary triaxial tests have also been performed in order to check the validity of the empirical relations for shear strength anisotropy in this type of soil.

The test series with different degrees of unloading were performed on specimens from 7 and 16 metres depth in Point 4. The results showed a normal effect of the unloading and that the undrained shear strength could be expressed as

$$c_u \approx 0.23\,\sigma'_0 \cdot OCR^{0.8}$$

The value of factor $a = 0.23$ is somewhat low in relation to what might be expected from empirical experience for a relatively high-plastic clay. The results have been summarised together with the other direct simple shear test results in the section in Fig. 111.

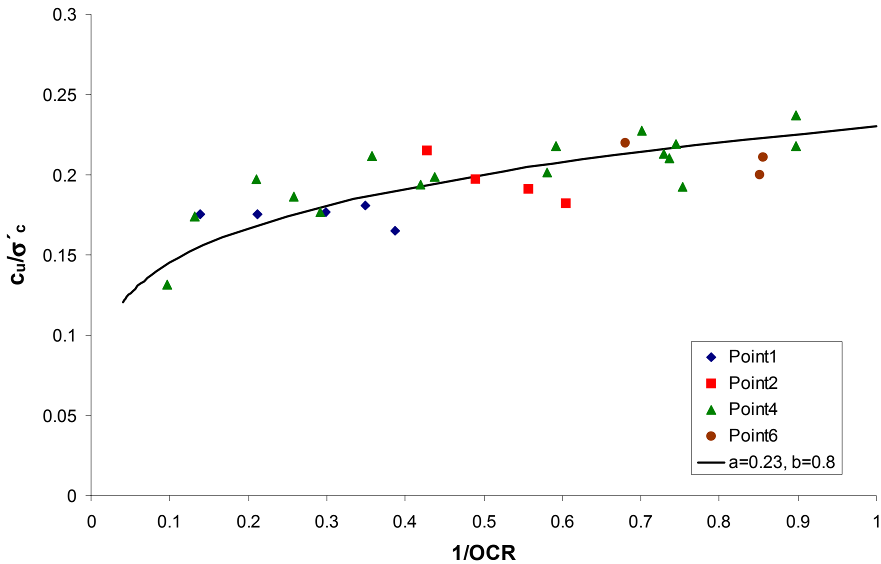
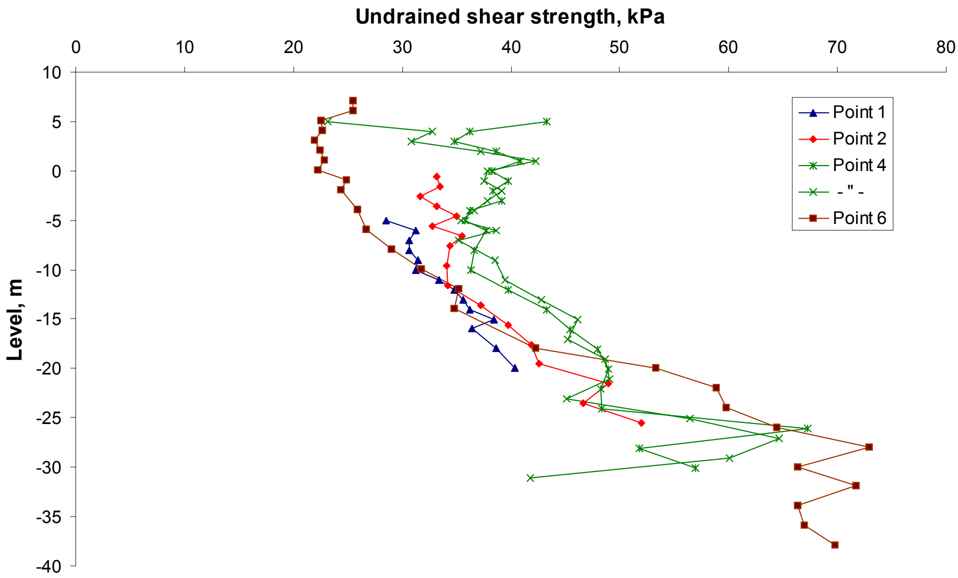


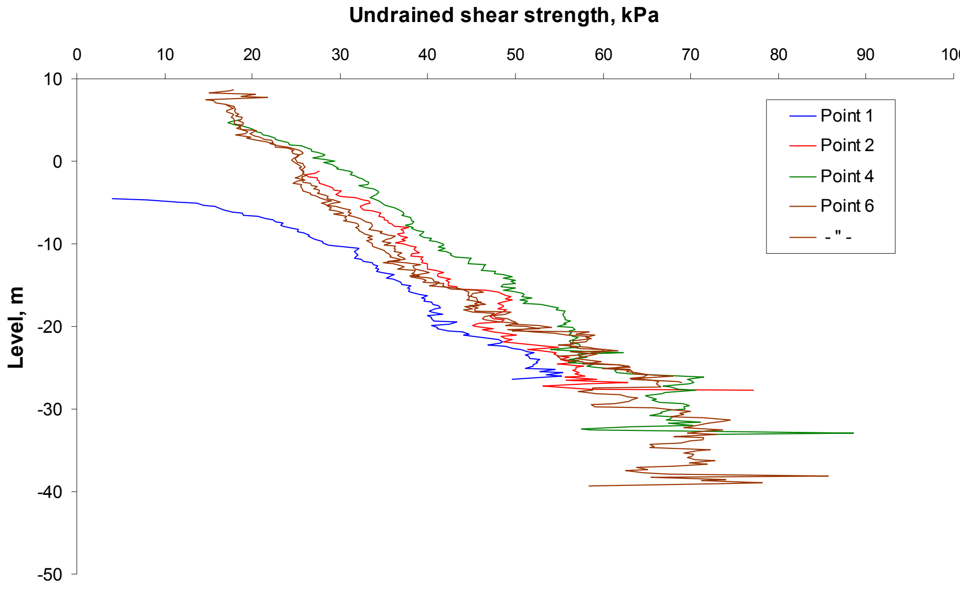
Fig. 111. Compilation of all undrained shear strengths determined by direct simple shear test on clay from Strandbacken. The results have been normalised against preconsolidation pressure and plotted against the inverse of the overconsolidation ratio.

The combined effect of preconsolidation and unloading entails that the undrained shear strength can be expected to be lowest below the river, somewhat higher at the riverbank, even higher behind the former crest and then decreasing somewhat at greater distance from the river. In principle, this was found in the results from the Göta-älv investigation and is also reflected in the new results. A compilation of the field vane test results is shown in Fig. 112a. The scatter in the test results is fairly large, but the relations between the results from the different points are still indicated. A clearer relation with less scatter is obtained from the results of the CPT tests after correction for overconsolidation ratio, Fig. 112b. A further illustration of the relation is obtained from the results from the dilatometer tests even if these have only been performed in two points, Fig. 112c and d. It should be observed that the differences measured at the different points in this section are not only a result of different degrees of unloading but to a large and in some cases major extent also due to differences in preconsolidation pressures.

The agreement between the different test results varies when all results at each point are put together. At Point 1 below the river, there is good agreement between the corrected CPT tests and the direct simple shear tests. The dilatometer tests yield strengths according to the first interpretation method that are higher, whereas those evaluated by the second method are almost identical to those from the first two types of tests, Fig. 113. All these values are slightly lower than is estimated by empirical relations based on preconsolidation pressure, overconsolidation ratio and liquid limit. The field vane tests showed similar strengths but these were generally higher and did not show the same reduction in strength at the low overburden pressures close to the river bottom. The strengths from the fall-cone tests were approximately the same as those from the field vane tests at shallow depths but as usual became too low at greater depths. All values from methods that are normally considered to be relevant, i.e. all except the fall-cone test, are within a band with a maximum deviation from the mean value of ±6 kPa. However, this is rather a lot when the mean value of the shear strength starts at about 22 kPa and the arithmetic mean is not necessarily the most relevant value. The values of the direct simple shear tests and methods that give approximately the same results are estimated to be most appropriate for design.

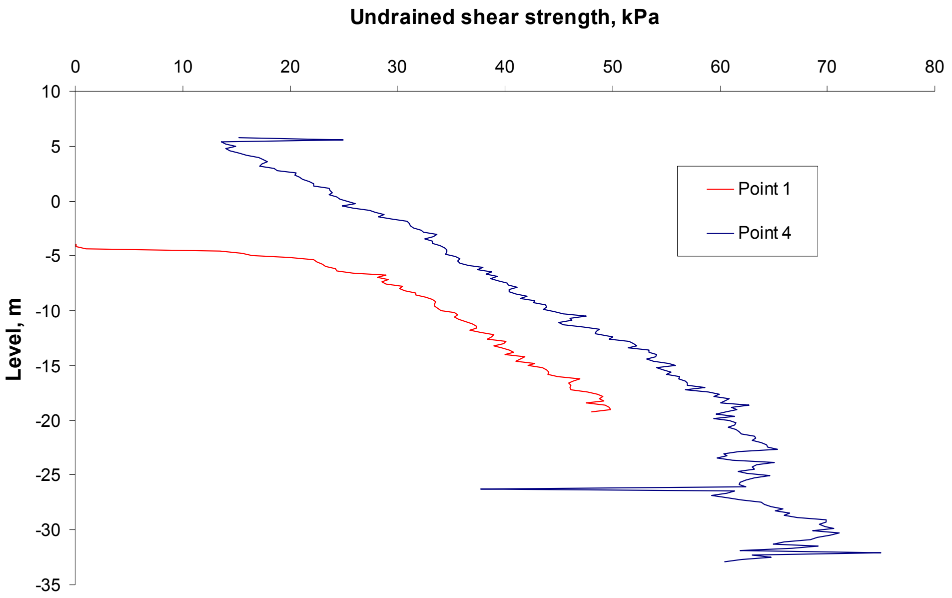


a)

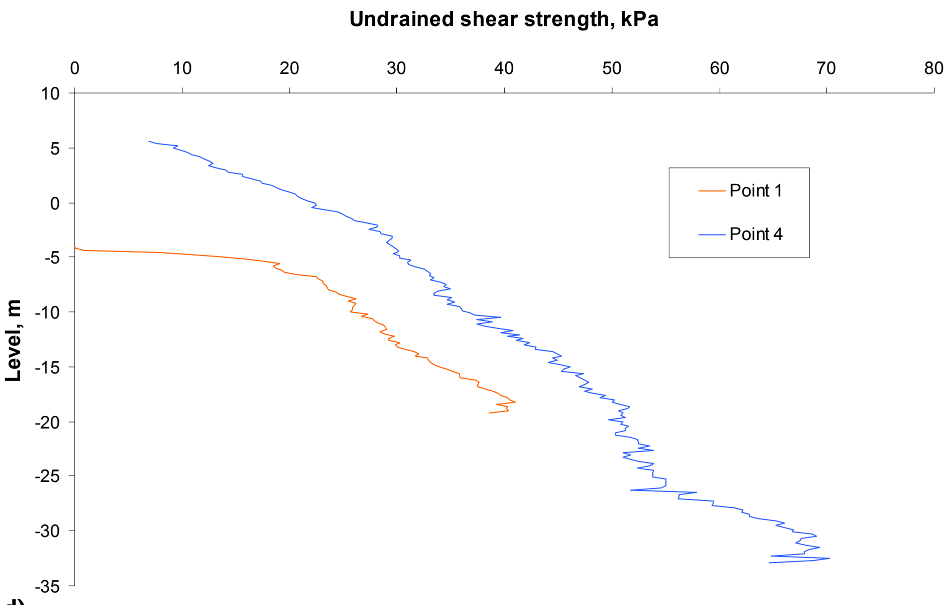


b)

Fig. 112. Comparisons between shear strength determinations at the different test points.
a) Field vane tests
b) CPT tests corrected for OCR



c)



d)

Fig. 112. **Comparisons between shear strength determinations at the different test points.**
c) Dilatometer tests evaluated according to SGI Information No. 10.
d) Dilatometer tests evaluated according to the alternative method using normalised shear strength.

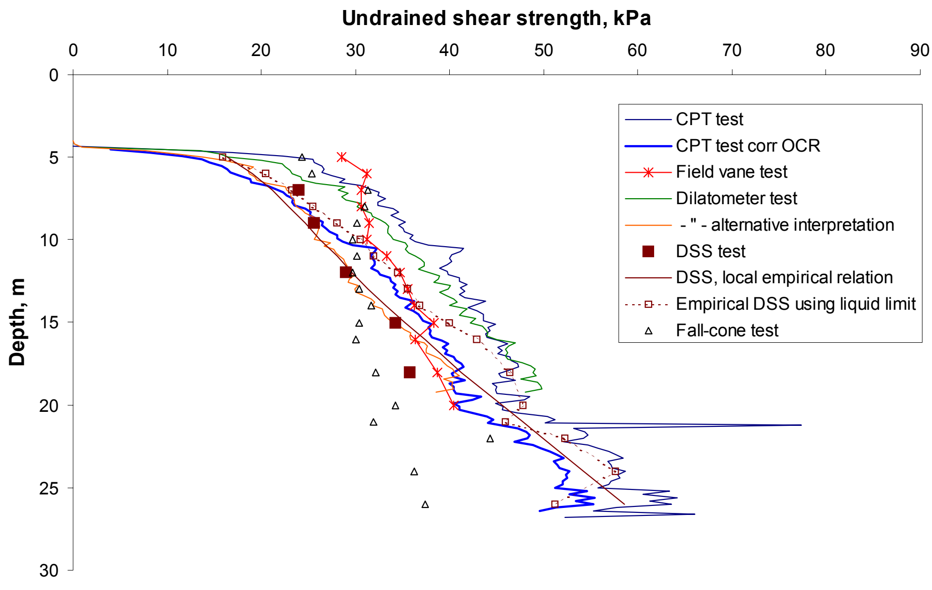


Fig. 113. Compilation of undrained shear strength determinations at Point 1.

Similar relations were obtained at Point 2. The strengths of the corrected CPT tests are again similar to the results of the direct simple shear tests, although they are somewhat higher at this point, Fig. 114. The direct simple shear test results were somewhat lower than is estimated from empirical experience and the strengths evaluated from the field vane tests are higher in the upper part of the profile. The field vane test thus do not show the same reduction in shear strength at low overburden pressures, but correspond well to the results from the direct simple shear test at larger depths. The results of the fall-cone tests are of the right size in the upper part of the profile but too low at greater depths. All relevant values are within a band with a maximum deviation from the mean value of ±4 kPa. Since the shear strength is somewhat higher at this point, the relative spread in results is also lower.

The difference between the shear strengths determined by different methods is larger at Point 4, particularly in the upper 10 metres or so of the profile, where the soil mainly consists of clayey silt and silty clay with plant remnants, other organic matter and shells, Fig. 115. In this material, the direct simple shear tests, the dilatometer tests and the corrected CPT test give very similar results, even though the alternative method of evaluation of the dilatometer tests yields somewhat too low values. The concordance also encompasses the empirically estimated shear

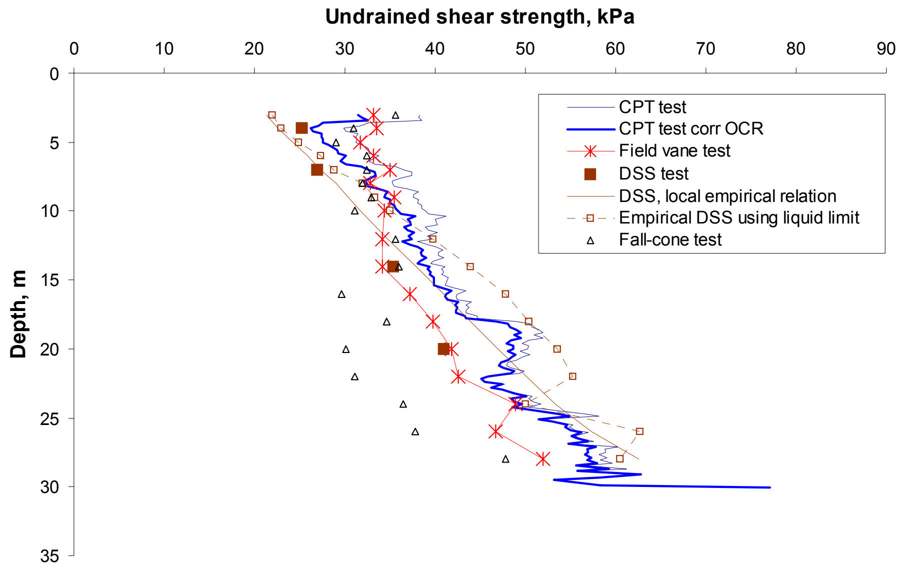


Fig. 114. Compilation of undrained shear strength determinations in Point 2.

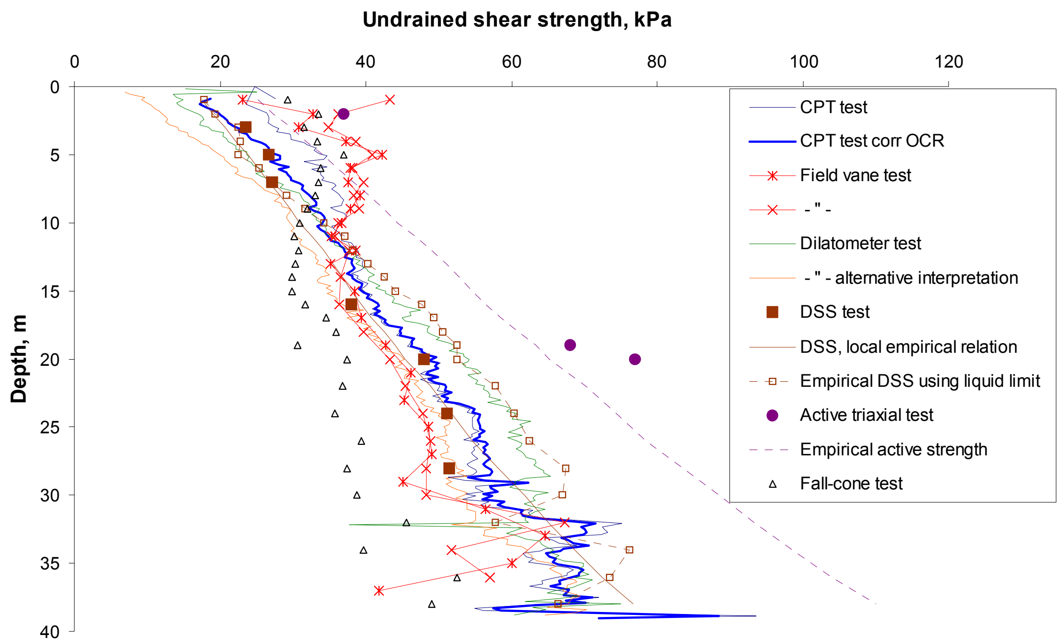


Fig. 115. Compilation of undrained shear strength determinations at Point 4.

strength at direct shear. In this upper layer, the fall-cone tests gave considerably higher shear strengths and the field vane tests even higher values. In the soil below, the direct simple shear tests, the field vane tests, the CPT tests and the dilatometer tests evaluated by the alternative method give roughly the same results. The empirically estimated values and the results obtained by the first method of interpretation of the dilatometer tests are somewhat higher. The results of the fall-cone tests are too low at these depths.

Undrained active triaxial compression tests have also been performed on specimens from Point 4 to check the shear strength anisotropy. The measured active shear strengths are fairly well in line with the empirically estimated values.

The same pattern with significantly higher values from the fall-cone tests and particularly the field vane tests is found also in the upper 11 metres or so of the profile at Point 6, where the soil consists of clayey silt and silty clay with plant remnants, other organic matter and shells, Fig. 116. Also the shear strengths from corrected CPT tests are here somewhat higher than the results from the direct simple shear tests. In the clay below, the results of direct simple shear tests, field vane tests and CPT tests are very unanimous. The empirically estimated shear strengths here are somewhat higher whereas the values from the fall-cone tests are generally too low.

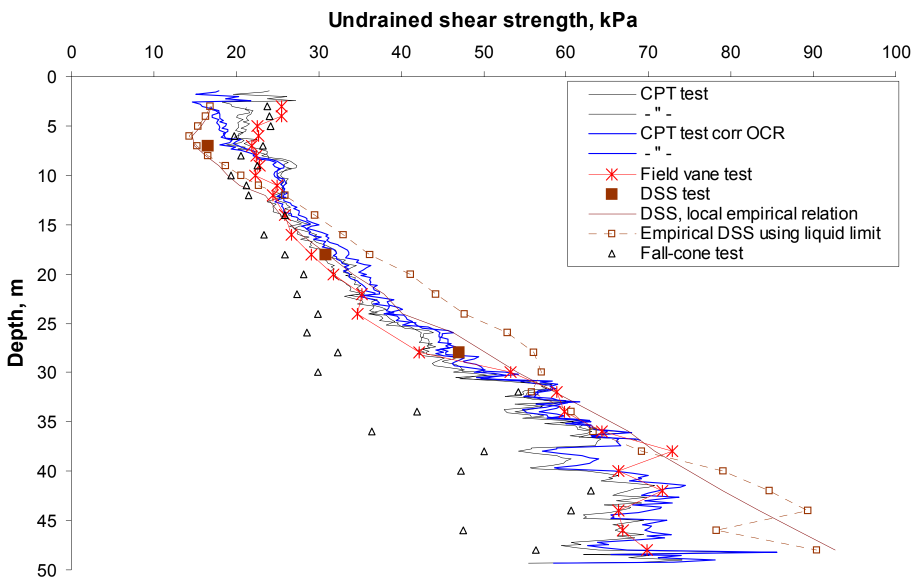


Fig. 116. Compilation of undrained shear strength determinations at Point 6.

When comparing the different test results, it should be remembered that field vane tests and CPT tests uncorrected for overconsolidation gave higher values in the overconsolidated clay in Torp as well. This appears to be related to the test conditions and measuring principles, which is currently studied in detail in another project (Löfroth 2002). In a comparison between the results in the organic clayey silt and silty clay, it should be observed that all the test methods, except direct simple shear tests and dilatometer tests, use empirical factors based on the liquid limit in evaluation of the results. The empirical basis for these factors does not contain any large number of soils of this type and composition. The relevance of the factors for this particular type of soil is thereby uncertain.

The drained shear strength parameters have been checked in a few triaxial tests on specimen from Point 4 and were found to be well in line with the empirical estimates that are normally used (Larsson et al. 1983).

3.9 CHANGES IN SHEAR STRENGTH DUE TO UNLOADING

That unloading affects the undrained shear strength has been shown by the comparisons that have been performed between the shear strengths measured by field vane tests at different points located below the river, at the riverbank and behind the crest of the slope(s). This has also been shown in the direct simple shear tests. It would have been desirable to make a direct comparison between the strength before and after excavation of the new terrace. However, the scatter in the results of the previous investigations is too large to allow this, particularly since the unloading is relatively moderate and it mainly affected those upper layers in which the soil consists of clayey silt and silty clay with plant remnants, other organic matter and shells and in which the results are particularly difficult to interpret. The indirect comparison, which on the other hand is rather comprehensive, will therefore have to suffice in this area.

3.10 STABILITY CALCULATIONS

3.10.1 Previous calculations

The stability of the slope at Strandbacken has been calculated many times from the investigation by SGI in 1951 to the present project. In the large Göta-älv investigation, the safety factor was found to be between 1.0 and 1.2 depending on what section was calculated, what method was used and what shear strength parameters were assumed. Calculated safety factors lower than unity have then probably been set to 1.0.

The limited excavation that was performed in 1959 in connection with an observed crack and an assessment that a slide was imminent was, as far as is known, not based on a criterion for a minimum required safety factor. Instead, it appears to have been designed to eliminate the immediate danger at a minimum of cost and interference with the existing built-up area.

Fairly large changes have occurred since the days of the Göta-älv investigation in how shear strength in clays is looked upon and how it is determined and evaluated. The shear strength values obtained in field vane tests and fall-cone tests have generally been corrected since 1969 and the correction factors were revised in 1984. Other types of tests have come into use, the loading history of the soil has come into consideration to a much larger extent and in new ways, and nowadays the shear strength anisotropy is also considered. The drained shear strength parameters are determined and considered in new ways and the combined analysis has come into use.

Much of this was taken into account in the investigation that was performed by SGI in 1988–89, which led to the large excavation works afterwards. The stability was then once more found to be close to 1.0. The excavation was designed to give calculated safety factors in undrained analyses of at least 1.5 for all potential slip surfaces reaching the remaining buildings in the area. However, the present guidelines of the Commission on Slope Stability were not issued until 1995 and could thus not be followed in all details.

3.10.2 Conditions in the new calculations

The new calculations that have been performed in this project concerning the conditions after the excavation have been based on the actual measurements of pore water pressures and shear strengths that have now been made. The calculations have also followed the new guidelines of the Committee on Slope Stability (1995).

The undrained shear strength in these calculations is largely based on the results of the direct simple shear tests and the local experience that has been created by these. These shear strength values differ from and are lower than those determined by field vane tests in the upper organic and silty soil layer directly below the superficial layers of sand and silt. At larger depths and for the major part of the clay deposits, the agreement between the direct simple shear tests and the field vane tests is fairly good, with a small tendency to higher values from the direct simple shear tests. The influence of the choice of shear strength in the upper layer is moderate since its thickness is limited and it does not exist at the riverbank or further out below the river. The difference in calculated safety factor for large slip surfaces is thereby marginal. For more superficial slip surfaces closer to the lower slope towards the river, a choice of a higher undrained shear strength mainly entails that the drained shear strength becomes governing for a larger part of the slip surface and that a slight displacement of the location of the most critical slip surface occurs.

Because of the location of the organic and silty layer in the section, it also mainly constitutes a zone with active shear in all slip surfaces. Its shear strength in more elaborate calculations is therefore mainly based on results of active triaxial tests.

The drained shear strength is based on empirically estimated values, whose relevance has been checked by triaxial tests on the particular type of soil.

The calculations have been performed with assumptions about the lowest water level in the river combined with the highest prognosticated pore water pressures in the soil. This condition is fully realistic. The water level in the river is regulated by the outlet of water through the power plant and can be lowered very rapidly. On the other hand, the pore pressures in the soil mass and particularly in its upper levels are regulated by climatic conditions. In the calculations for the section before the excavation, an estimation has been made of the groundwater level in the upper sand and silt layer and the pore pressures in the soil below. This has been based on the previous measurements of pore pressures in the area and assumptions about pore pressures corresponding to the mean water level in the river in the coarse bottom layers. The estimations about the pore pressures have been aided by calculations with the computer program SEEP/W, see Chapter 5.

The calculations mainly refer to the investigated section. The geometrical variations along the river are mainly related to the position and depth of the fairway in the river, but other variations in the bottom contour exist. No detailed measurements of the bottom contour have been performed lately but the assumptions are primarily based on observations in the area around Point 1 in the river, the charts of the fairway provided by the fairway authorities and data from sections in previous investigations. An erosion protection structure for the riverbanks has been in effect since these investigations were made but changes in the contour of the river bottom can possibly have occurred. A new detailed investigation of the bottom topography along a large part of Göta-älv is planned.

The basic section in the calculations has a bottom contour in the river in which the water depth increases gradually over a shallow shelf from 1 metre at the riverbank to 5 metres at the border of the fairway and then rapidly increases to full depth in the fairway. The depth in the fairway is set to 8 metres in accordance with the depths at this section in the chart of the fairway. All depths given in this paragraph are given in relation to the lowest water level in the river.

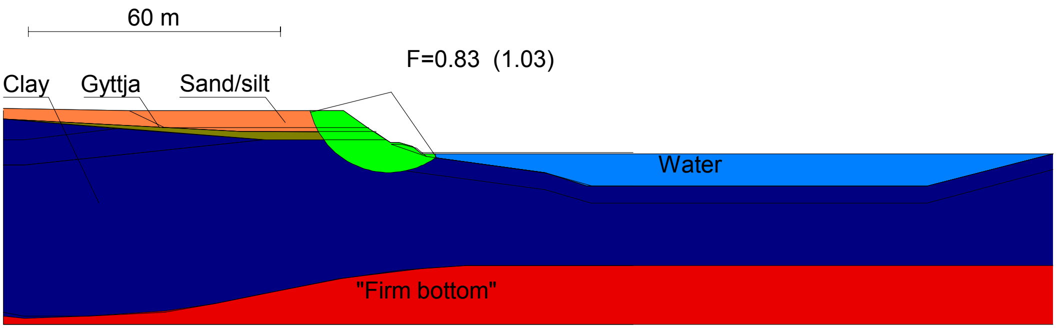
The minimum depth in the fairway should be 6 metres, but it has been found to be up to 12 metres in previously measured sections in the northern part of the area. The width of the shallow shelf varies from being practically non-existent close to the lock at Ström, where the fairway runs along the western side of the river, to more than 50 metres in the southern part of the area from the bollard and further on. The width has been set to 30 metres in the basic section, but the influence of the width of the shelf and the depth in the fairway has been analysed by varying these parameters in the calculations.

Only two-dimensional aspects have been considered because of the length of the excavated area and the similar and only gradually changing conditions along this distance. There are thus no significant end effects to take into account in the considered sections.

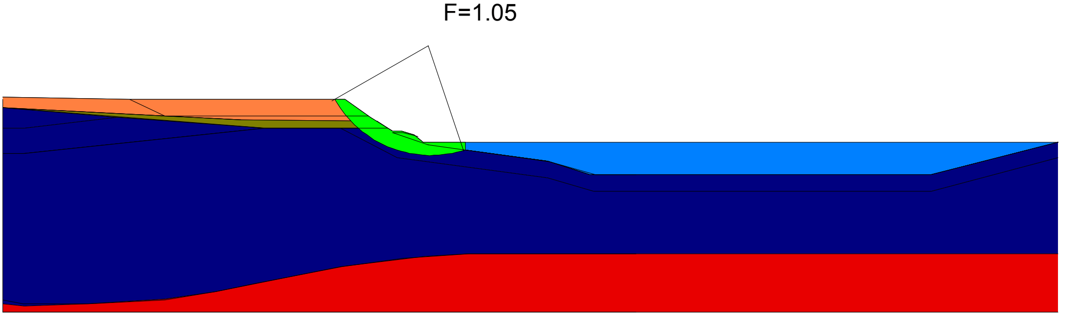
The first calculations have been made using the self-seeking program SLOPE/W and rigorous analyses according to the Spencer method. The shear strength anisotropy was not considered in these calculations. Some of the critical slip surfaces that were found in these calculations have then been modified and control calculations have been performed with Janbu's general procedure of slices using more elaborated shear strength parameters.

3.10.3 Results from the calculations

The calculations for the original section before the excavation without consideration of shear strength anisotropy gave a safety factor of 0.83 for relatively shallow slip surfaces comprising the outer part of the slope, Fig. 117a. In these initial calculations, drained shear strength was assumed in the sand and silt layer and undrained shear strength was used in the gyttja and clay layers. The calculated factor of safety increases to 1.0 (1.03) in a combined analysis where the shear strength anisotropy is considered. The critical slip surface is then shallower and drained shear strength is governing along a larger part of it. The calculated safety factor in a fully drained analysis becomes approximately the same (1.05), but the critical slip surface is then even shallower, Fig. 117b.



a)



b)

Fig. 117. **Calculated safety factor for the original section (SLOPE/W, Spencer's method).**
a) Undrained isotropic shear strength in the clay. (Values in brackets refer to combined analysis with anisotropic shear strength and Janbu's general procedure of slices.)
b) Drained analysis.

The calculated safety factor for the original section increases only slowly when larger slip surfaces are considered. It is thus only about 1.1 for a large slip surface involving all of the excavated area and a small strip behind the new upper crest, Fig. 118. Assumptions of anisotropic shear strength and possible governing drained shear strength in parts of the clay layers have only marginal effects on this large slip surface.

The smaller slip surfaces in the outer part of the slope end at its toe close to the riverbank. The calculated safety factor is thereby mainly unaffected by the width of the shallow shelf in the river until this becomes very small. However, an erosion process that affects the water depth directly at the riverbank has a large impact on the safety factor. A roughly 10 metre wide excavation, such as that performed in 1959, has a large effect on the local stability in this case, since it covers all of the most critical slip surfaces. This effect decreases rapidly for larger slip surfaces and becomes practically non-existent for very large ones.

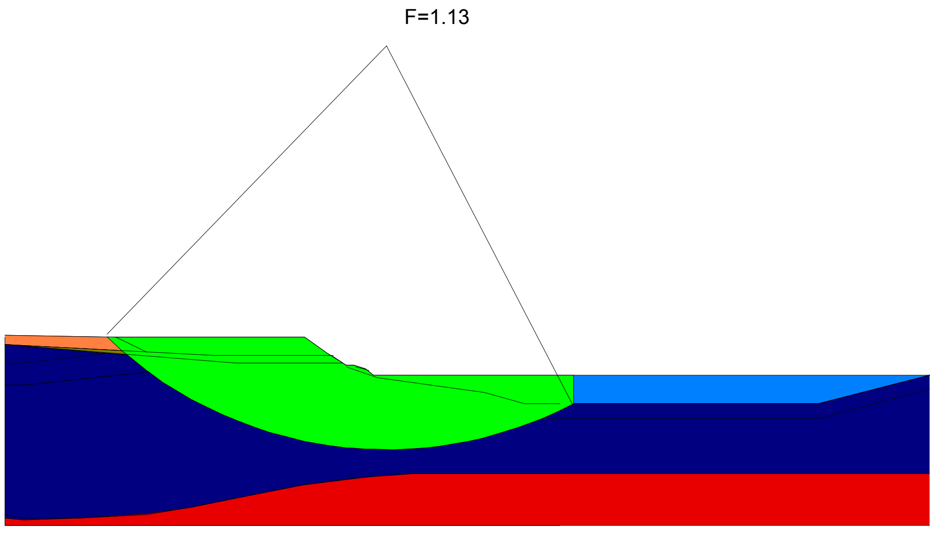
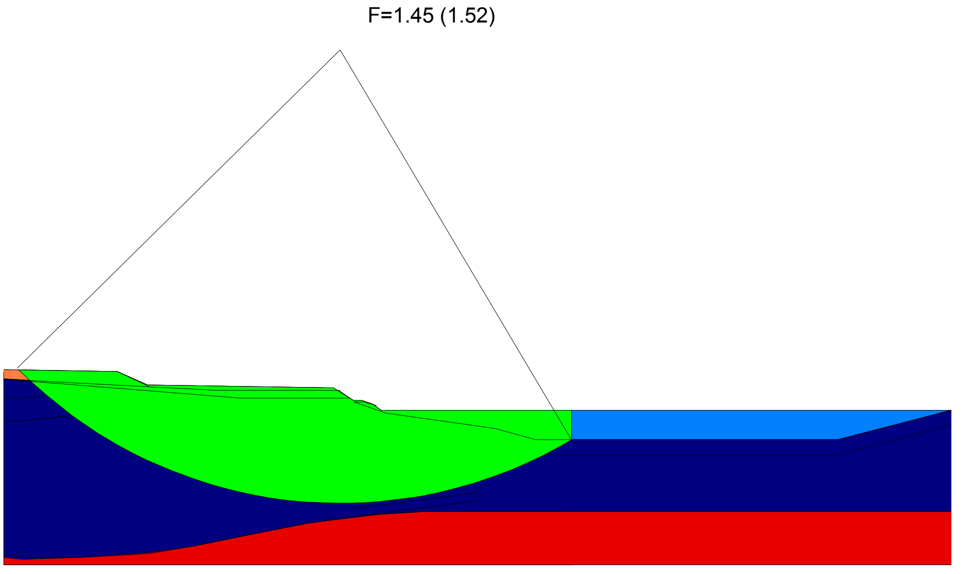
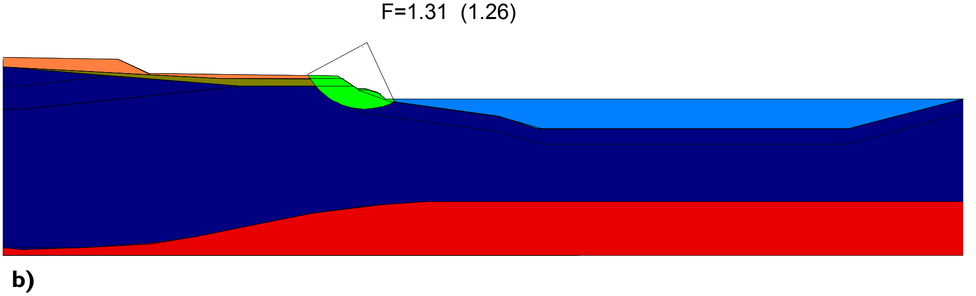


Fig. 118. Calculated safety factor for a large slip surface in the original section with undrained isotropic shear strength in the clay (SLOPE/W, Spencer´s method).

The calculations for the present conditions after adaptation to the effects of the large excavation resulted in safety factors that had increased to 1.45 for the large slip surfaces and 1.3 for more local slip surfaces at the lower slope towards the river, Fig. 119. These factors were calculated with assumptions of isotropic undrained shear strength in the clay and drained shear strength in the sand and silt layers. They represent local minimum values, and the calculated safety factors for slip surfaces



a)



b)

Fig. 119. Calculated safety factor in the present section using undrained isotropic shear strength in the clay (SLOPE/W, Spencer's method).
(Values in brackets refer to combined analysis with anisotropic shear strength and Janbu's general procedure of slices.)
a) large slip surface
b) local slip surface at the lower slope

located in between and outside are higher. In a combined analysis, the drained shear strength becomes governing solely for the smaller slip surfaces. The critical slip surface then becomes smaller and the calculated safety factor decreases to 1.2, Fig. 120.

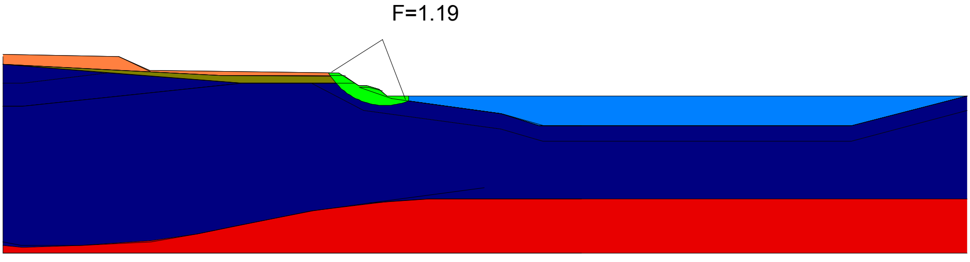


Fig. 120. Calculated safety facor in the present section using drained analysis (SLOPE/W, Spencer's method).

A study of the effect of further erosion shows that if the shallow shelf at the toe of the slope is removed completely, the safety factors decrease to 1.2 in undrained analyses and 1.0 in drained analyses. At even greater water depths than the assumed 8 metres, there has to exist a shallow shelf with a certain width in order to obtain unity for the calculated safety factors.

The control calculation using Janbu's general procedure of slices gave very similar results. The method provided better possibilities to take variations in density and shear strength with depth and distance from the river into account. It also facilitates a simultaneous modelling of the shear strength anisotropy and the drained governing shear strength in superficial layers with low effective stresses. The influences of these aspects are partly counteracting. The influence of all of them is low for the large slip surfaces, and the calculations for these resulted in only slightly higher safety factors, for example 1.52 compared to 1.45 for the long slip surface after excavation shown in Fig. 119a.

The influence of anisotropy and partly drained soil layers is generally larger for shorter slip surfaces. In a combined analysis of the slip surface that with undrained analyses and isotropic shear strength showed a safety factor of 1.3, Fig. 119b, the calculated safety factor decreases to 1.2 when the same isotropic undrained shear strength is used and increases to 1.5 when the shear strength anisotropy is taken into account. However, the calculated safety factor is still about 1.2 for more superficial slip surfaces where the drained shear strength is governing.

The excavation performed has thus resulted in an increase in calculated safety factor according to undrained analyses to about 1.5 for the whole area, which was the aim of the design. Only superficial slip surfaces at the lower slope have lower calculated safety factors. These factors are around 1.2 and the drained shear strength is then solely governing. The requirement for calculated safety factors for the areas and the remaining buildings and constructions according to the directions from the Commission on Slope Stability is thus fulfilled. However, the safety factor for the lower slope depends on the contour of the bottom in the river, and erosion that results in increasing water depths at the riverbank would entail deteriorating stability conditions.

Chapter 4.

Sundholmen

4.1 DESCRIPTION OF THE AREA AND ITS GEOLOGY

4.1.1 Description of the area

Sundholmen is a small village with about thirty dwelling houses located on both sides of the small river Viskan. Highway number 41 between the cities of Borås and Varberg runs parallel to the river, which flows in a valley from north-east to south-west. The distance to Borås is about 50 km and to Varberg about 40 km. At the centre of the village, there is a side-road towards Öxnevalla to the south-east, which crosses the Viskan on a bridge, Fig. 121.

Sundholmen lies just south of a point where the topography changes from a roughly 700 metres wide valley to a plain about 2 km wide. The village is surrounded by farmland. The ground surface in the valley and on the plain is very flat and is only broken through by the river-course and smaller ravines along its tributaries in the form of small streams and brooks. The distance from the centre of the village to the surrounding rocky ridges is about 400 metres towards the north-west and 600 metres towards the north-east. In the south-easterly direction, hills can only be seen at a great distance.

The private grounds and gardens along the river reach all the way to its banks but the buildings are located at distances of at least 30 metres from them. Only a few sheds lie closer. Correspondingly, the highway and the parallel local road on the other side of the river run at distances of at least 50 metres from it.

Small slips and slides are very common along the river, and many shallow slip scarps can be observed when travelling along it. These slips are normally very shallow and then only comprise one or a few metres of the riverbank, but sometimes also larger slides occur. One such slide occurred in Sundholmen in 1956 in front of the Brosätter nursing home, which is located on the eastern riverbank on the ground directly to the north of the bridge. This slide extended about 5 metres into the ground. Later on, cracks were observed at the opposite riverbank as well and

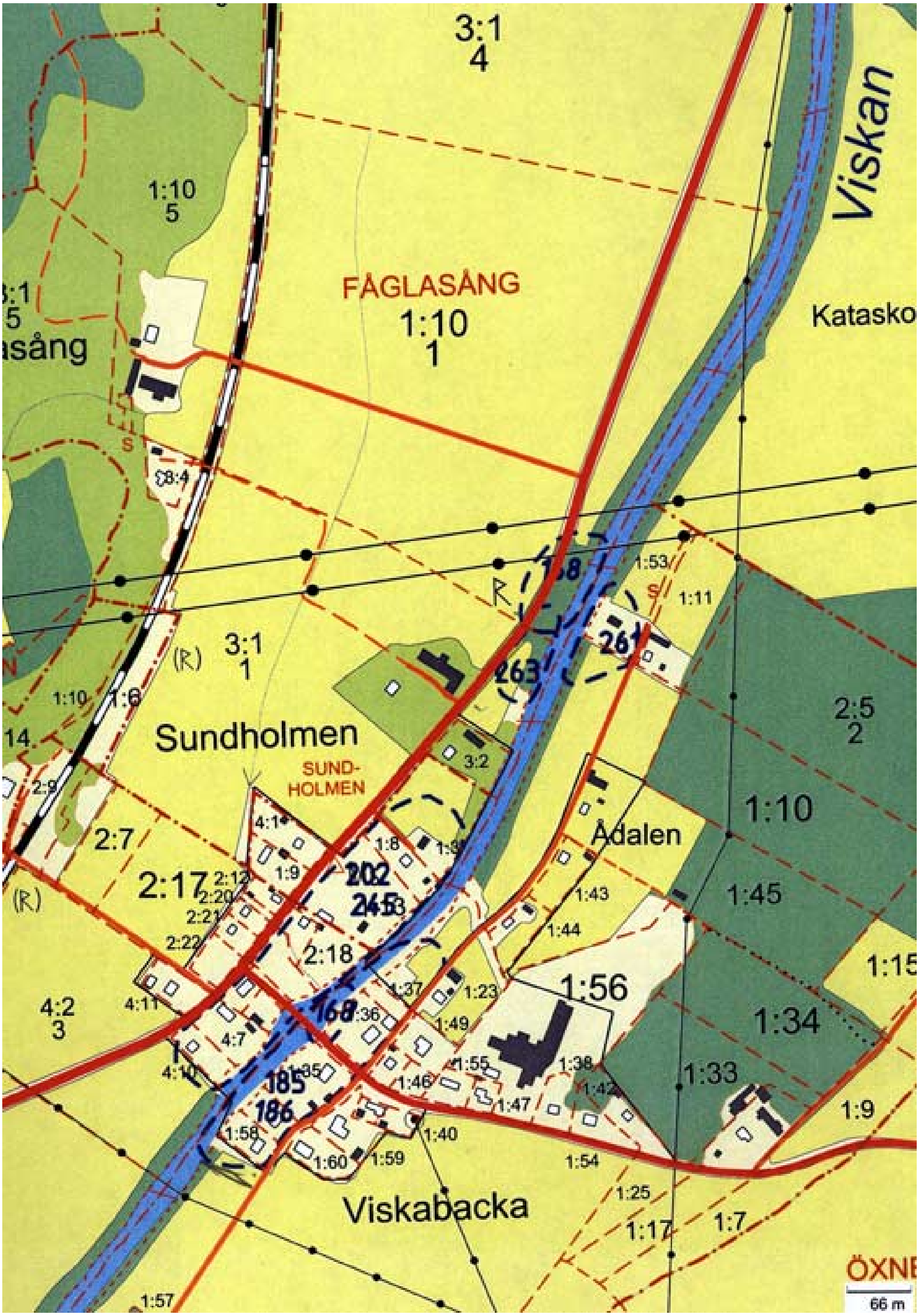


Fig. 121. Map of Sundholmen, (Mark municipality).

a geotechnical investigation was performed by SGI during 1957. The report of this investigation recommended that an excavation should be made behind the crest of the slope to the river, partly to ensure the stability of the nursing home and adjacent grounds and partly to eliminate the risk that slides would affect the bridge and its abutments. However, this excavation was not performed.

Two new slides occurred in the eastern riverbank in June and July 1989, one to the south and downstream of the bridge and one upstream in the ground to the north of the nursing home. These slides had larger extents and reached about 10 metres in from the former crest at the riverbank. As an immediate result, some grounds were roped-off and a new geotechnical investigation was started. This comprised the entire eastern riverbank from about 100 metres upstream from the bridge to about 150 metres downstream. It was conducted by the consulting company GF, which recommended that an excavation with a depth of 1 to 2 metres should be made from the crest almost up to the existing buildings, i.e. with a width of about 25 metres. This excavation was performed during 1990, but its shape and extent was altered somewhat in accordance with the desires of the landowners. The excavation was carried out with a certain inclination of the terraces down towards the river and drainage systems were also installed to facilitate the restoration of lawns and gardens, Fig. 122. Smaller excavations were also performed on the opposite side of the river.

The water level in the river varies strongly. It varies with season and can also vary relatively much within shorter periods depending on regulations and generation of power for industries further up along the river and its main tributaries. The normal variation between the highest and lowest water levels is about 3.5 metres, but the level can rise another metre at exceptional water flows. This entailed, among other things, a limitation for the possible depth of the excavations.

The present bridge over the Viskan is a modern construction in concrete. However, there have been a number of older constructions of stone at the same point, which have successively replaced each other. The remnants of these constructions have formed a barrier resulting in a deeper hollow at the river bottom to the north thereof and a strong torrent starting below the bridge and continuing for some distance to the south. Erosion protection was constructed on both sides of the river in connection with the excavation works. At the torrent, there is also a stronger erosion protection structure in the form of a wall built of stone blocks on the western side of the river, Fig. 123.

Fig. 122. The river Viskan to the north of the bridge in Sundholmen. The white plastic tubes that protrude from the slope are parts of the drainage system for the excavated terrace behind.

a) The bridge seen from the north with the barrier of stone blocks.

b) The bridge seen from the east with the torrent and erosion protection downstream.

c) The river south of the bridge with the torrent and erosion protection.

Fig. 123. The bridge over the Viskan in Sundholmen.

4.1.2 Geology

No detailed geological survey has been made of the area. In general, the geological history is the same as for the previously described areas in Munkedal and Lilla Edet. The main differences are that the soft clay deposits started to be formed a few hundred years earlier because the area is located further to the south and that the thickness of the clay deposits is even greater than at the other places. There are no soundings or sampling operations to the full depth but there are a number of deep wells in the village where the depths to the aquifers with coarser soil below the clay are reported to be between 86 and 93 metres.

In this area too, the sediments were deposited in salt water. Samples have only been taken to a depth of 25 metres below the ground surface, but the results of the CPT tests show that there are a number of clay layers of different characters below this depth. Distinct borders between different layers are thus found at depths of about 40 and 48 metres below the ground surface. The upper clay layer down to 25 metres' depth consists of grey, sulphide-spotted clay. When the water depths had become shallow at the end of the deposition period, organic material also started to be deposited. There is thus a transition zone with an organic content between 6 and 7 metres depth and the soil above this zone is significantly more organic. When the land-elevation had proceeded so far that the shoreline passed the area, delta and lateral fluvial sediments were deposited at the river mouth. These sediments consist of silt and sand with organic matter.

The upper layers vary across the valley. The delta and lateral fluvial sediments are found in the superficial layers at the centre of the valley over about half of its width. In the outer parts, they consist of silt and an upper sand layer is found only within narrow strips on each side of the river. The upper soil layers on the eastern side of the river consist of about 1 metre of sand on top of about 1.5–2 metres of silt with thin sand layers and then organic clay down to 6 metres' depth. The silt layer is thicker on the western side, and in SGI's investigation in 1957 the soil was here designated as clayey silt without any significant organic content down to the transition to the sulphide-spotted clay, which was found at the same level as on the other side.

The river has later eroded its course through the more easily eroded sand and silt layers and the organic clay. Its bottom lies about 8 metres below the surrounding ground in the investigated area north of the bridge. However, the depth varies with shallower and deeper patches and the deep channel, which runs in the eastern part of the course, is up to half a metre deeper. The level of the surrounding ground is at +15 metres and the bottom level in the river is at about +7 metres. The lowest water level is given as + 9 metres and the highest as +12.5 metres. The water depth in the river thus normally varies between 2 and 5.5 metres and the water surface is only 2.5 metres below the surrounding ground at the highest water level. The excavation has entailed that only about half a metre then remains before the excavated terrace becomes flooded. This may happen occasionally since the water level can rise about 1 metre further at exceptional water flows, but this is rare and then only temporary.

The groundwater level in the surrounding ground is located about 1 metre below the ground surface. It varies somewhat with distance from the river because a certain drainage and water flow towards the river occurs in the vicinity of this. It may also vary due to drainage systems around the buildings and along the roads. However, in general the free groundwater level can be assumed to be located at the given level and the groundwater pressure to be hydrostatic from this. The groundwater pressures in the upper soil layers in the vicinity of the river are lower with a gradual adaptation to the water level in the river. The excavation performed and the drainage system installed below this may have entailed that this zone with lower pore pressures has become somewhat enlarged.

4.2 PREVIOUS INVESTIGATIONS AND STABILITY ASSESSMENTS

4.2.1 The investigation by SGI in 1957

The first investigation of the stability of the slopes towards the Viskan in Sundholmen was performed by SGI in 1957. In was induced by a slide that occurred in October 1956 in the eastern riverbank in front of the Brosätter nursing home north of the bridge. The slide was about 35 metres long and reached about 5 metres in behind the old crest. Cracks were also observed on the opposite western side of the river in 1957. Investigations were therefore made in three sections within a distance of 70 metres to the north of the bridge. They comprised weight sounding tests, field vane tests and sampling with piston sampler on both sides of the river in each section.

In the weight sounding tests, turning of the equipment was started at a depth of about 13 metres below the surface of the surrounding ground, which was used as a reference level, and the number of half turns per 0.2 metres of penetration then increased gradually with depth. The deepest penetrations were stopped at a depth of about 30 metres below the reference level without reaching any stop or finding any significantly stiffer layer. The field vane tests were stopped about 20 metres below the reference level after having shown a continuously increasing strength with depth. The piston sampling was at most performed to 15 metres below the reference level.

The soil samples were investigated in the laboratory regarding classification, density and undrained shear strength by both fall-cone tests and unconfined compression tests, which was the standard procedure at SGI at that time. The investigations showed that the conditions were very similar on each side along the river but that the upper soil layers differed between the two sides. On the eastern side, where the Brosätter nursing home is located, there was about 3 metres of coarse silt on top of about 3 metres of organic clay in the upper layers. The ground surface on the western side was about 1 metre lower and the upper soil here consisted of about 5 metres of clayey silt without any observed organic content. Below these upper layers, the soil consisted of similar clay on both sides of the river.

The difference in constituents in the upper soil layers was reflected in the density, which in the upper soil layers was significantly higher in the clayey silt on the western side than in the even-graded coarse silt and the organic silt on the eastern side, Fig. 124.

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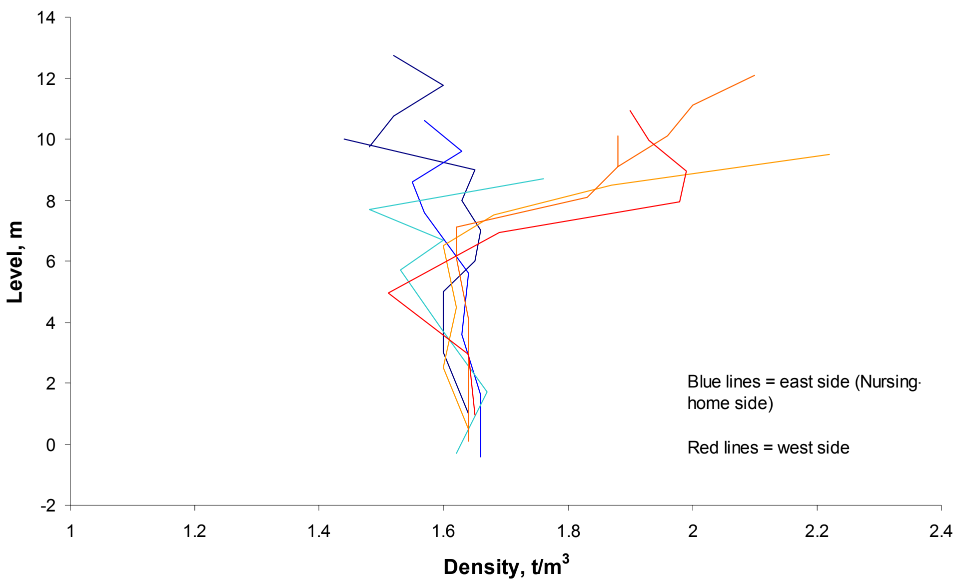


Fig. 124. Densities on each side of the river measured by SGI in the investigation in 1957.

The differences are also reflected in the shear strength values from the fall-cone tests, where considerably higher values were obtained in the clayey silt from the western side than in the organic clay from the eastern side. However, no significant difference between the two sides was obtained in the clays at larger depths, Fig. 125.

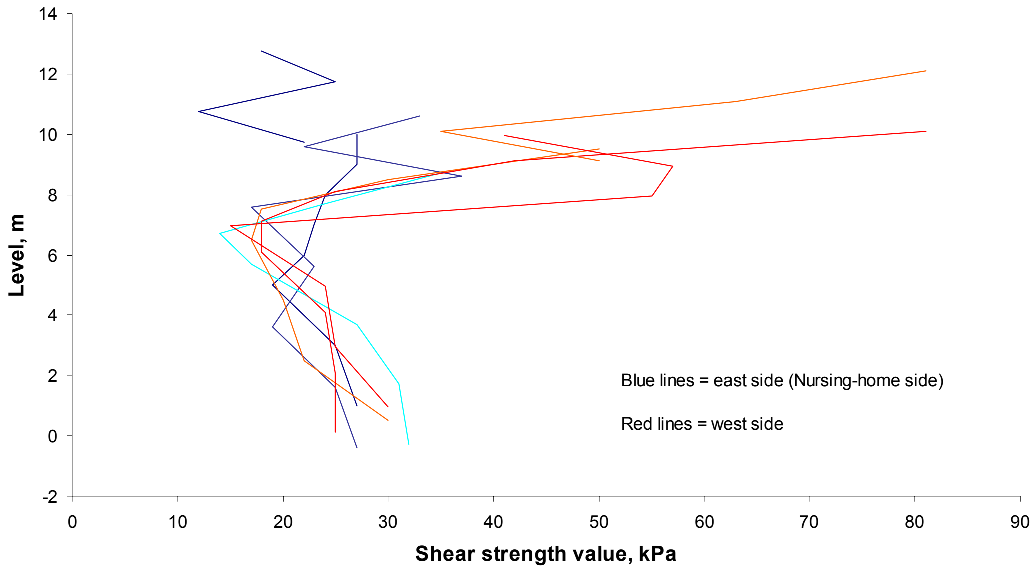


Fig. 125. Shear strength values from fall-cone tests in 1957.

The unconfined compression tests showed strengths that in general were only about two-thirds of those obtained in the fall-cone tests. The latter should be corrected with respect to the liquid limit, which at that time was only done for organic soil (gyttja). However, even after a correction of the fall-cone test such as that used today the values obtained by the unconfined compression tests remain considerably lower. This has been a general finding and, since the values of the unconfined compression tests were found to be unrealistically low, the method has later been abandoned in Swedish practice for testing of natural clays.

The results of the field vane tests in general showed the same relations between the two sides as the fall-cone tests, Fig. 126. The results of the field vane tests and the fall-cone tests are also numerically similar down to a depth of about 10 metres, whereupon the values from the fall-cone tests become lower.

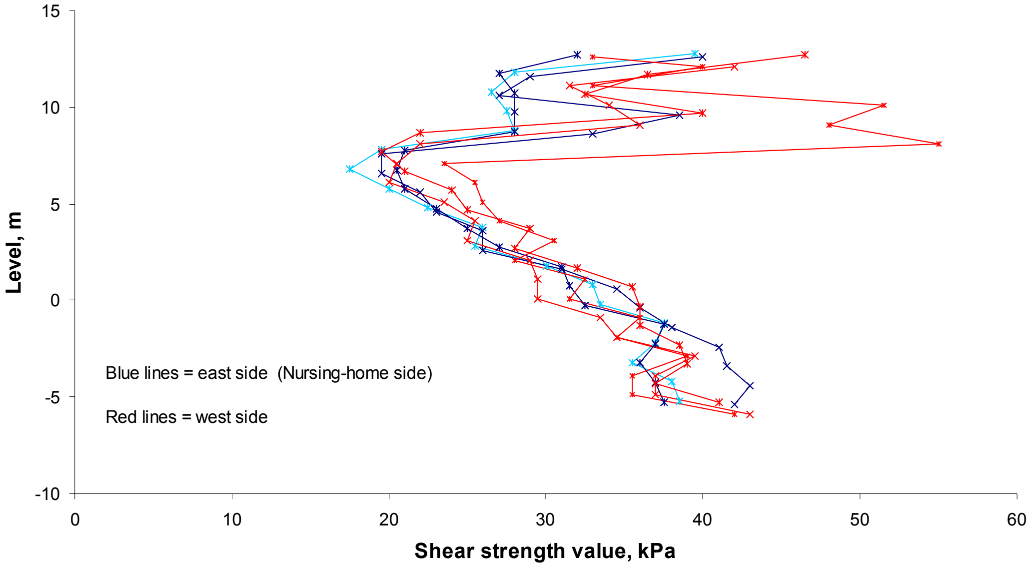


Fig. 126. Shear strength values from field vane tests in 1957.

The report from SGI stated that the stability calculations showed that the ground was "highly strained" (i.e. that the stability was low) on the eastern side of the river, but no specific safety factor was given. An excavation of the upper soil layer to a depth of 1 metre over a width of 7 to 15 metres was recommended along the entire distance investigated at this side. A similar excavation was not considered to be necessary on the western side since the ground surface was lower and the shear strength was equal or even better here. However, this excavation proposed by SGI was not carried out.

4.2.2 The investigation by GF in 1989

Two new slides occurred on the eastern side of the Viskan in Sundholmen during summer 1989. The first slide took place about 100 metres downstream from the bridge. It was about 50 metres long and reached about 10 metres behind the former crest, and a large part of the garden in front of one of the houses was involved, Fig. 127. The second slide occurred about a month later at about the same distance upstream from the bridge. It had a length of about 40 metres and also reached about 10 metres behind the former crest. It took place a short distance to the north of the slide in 1956 and was located outside the grounds of the nursing home and outside the area where the gardens reach all the way to the riverbank, Fig. 128.

Fig. 127. **The slide downstream from the bridge in Sundholmen in 1989 with the back scarp running through the landowner's potato-patch. Photo Å. Johansson.**

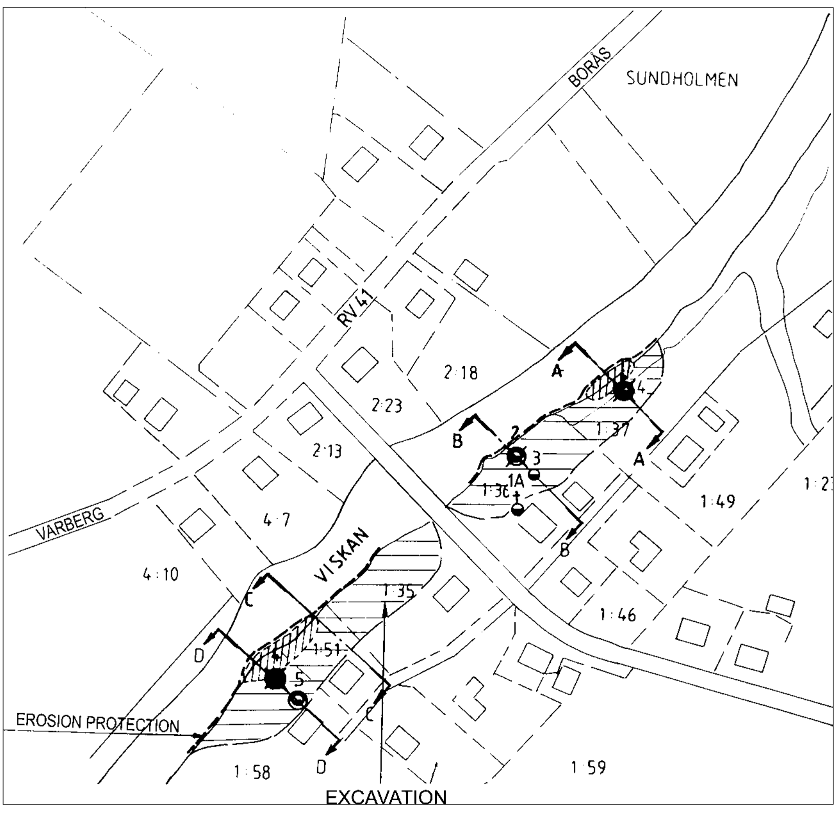


Fig. 128. **Location of the slides, investigations and proposed remedial measures in 1989 (GF 1989).**

The consulting company GF now performed a new geotechnical investigation. This comprised the whole distance along the eastern riverbank where dwelling houses are located in the vicinity of the river, both upstream and downstream from the bridge. The field investigations comprised total pressure soundings at five points, field vane tests at three points, undisturbed sampling by the standard piston sampler in two points supplemented by disturbed sampling in superficial layers at four more points and pore pressure measurements at one point. A detailed survey and levelling of the area was also made.

The penetration tests were stopped at a level of about –10 metres, i.e. about 25 metres below the level of the ground surface in the surrounding area and at about the same level where the previous weight sounding tests had been stopped. The results down to this depth indicated that the soil below the upper layers of sand, silt and organic soil consisted of homogeneous clay with a stiffness gradually increasing with depth. No significant differences could be observed between the results from the different test points. The field vane tests were performed down to the same depths as the penetration tests, whereas the sampling operations were stopped about 5 metres further up.

The inspection of the soil samples and the results of the routine tests in the laboratory showed that the conditions were very even throughout the investigated area. They were also consistent with the results obtained in the previous investigation by SGI. A minor difference can be related to the change in classification system that had been made in Sweden between the two investigations. The upper layer that had previously been designated as ”mo” was now classified as sand turning to silt with depth and the previously designated organic clay was now classified with a more gradual change from silty gyttja to clayey gyttja.

The measured densities were very similar in the investigated area and in both investigations, Fig. 129. Values of the liquid limit were only reported in the latter investigation but were here very unanimous in spite of a distance of about 200 metres between the sampling points, Fig. 130. This means that they could be used for correction of the older strength determinations and that the same corrections should be applicable for all values from field vane tests and fall-cone tests at the respective levels on the eastern side of the river.

In the report by GF it was observed that the consistency in the results from the field vane tests was ”almost improbably good”. This is valid also when the results of the previous investigation by SGI are included in the comparison, Fig. 131. For this reason, the area was included among the sites in a special study where comparative

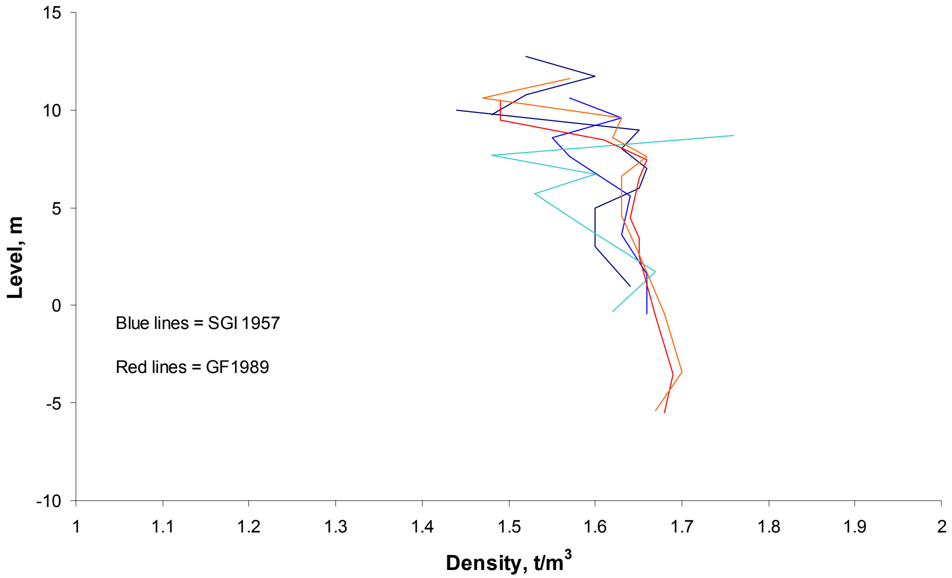


Fig. 129. **Densities measured in the investigation by GF 1989 compared to previous results in the same area.**

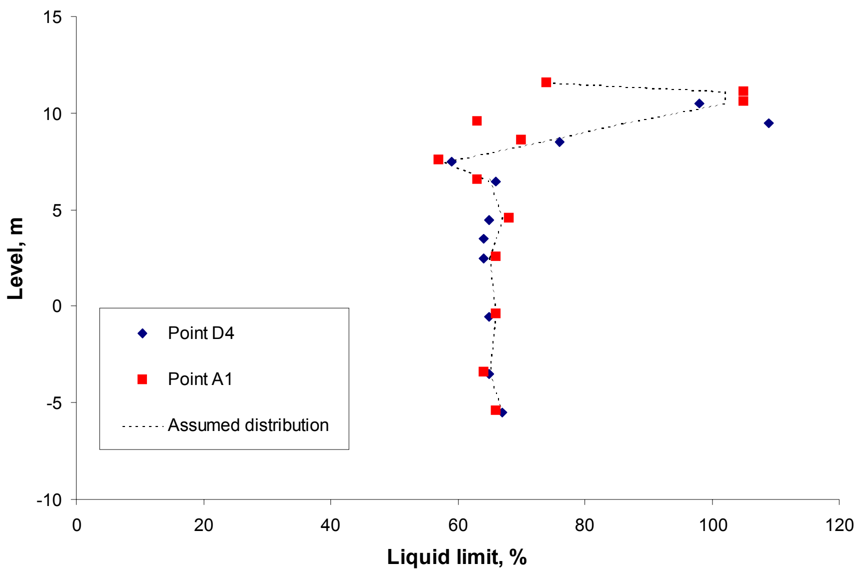


Fig. 130. **Liquid limits determined in the investigation by GF in 1989.**

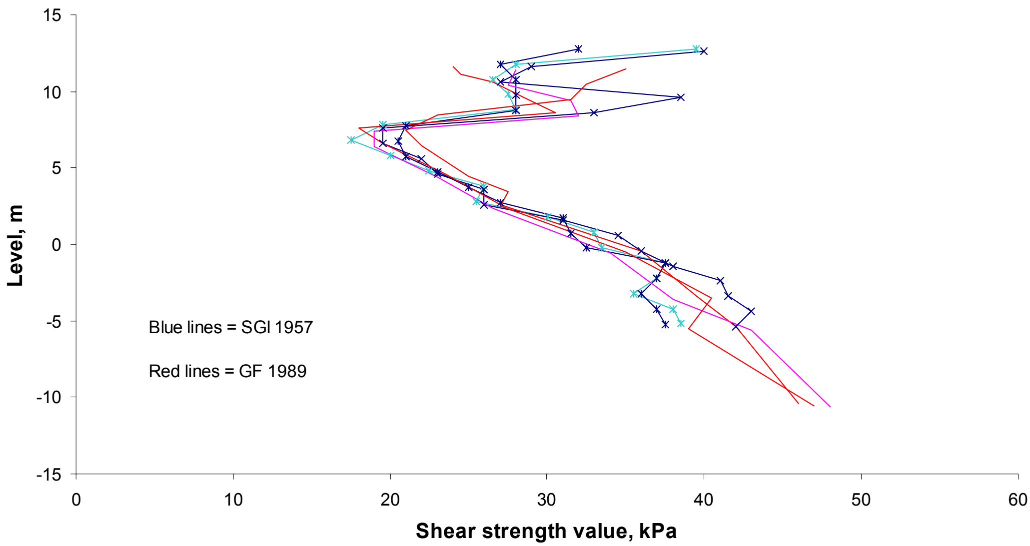


Fig. 131. Shear strength values obtained from field vane tests in the investigation by GF in 1989 compared to previous results.

investigations have been made regarding the influence of the vane size on the results of field vane tests (Åhnberg et al. 2001).

The border between the organic soil and the underlying clay is clearly discernible in the results from the field vane tests. It is also clear that a significant change in the shear strength increase with depth occurs at a level of about -2 metres. However, no corresponding change in liquid limit or other soil properties can be seen in the results of the routine tests.

The pore pressure measurements were made in a station located directly behind the back scarp of the slide in the northern part of the area. About one month after the slide event, the pressures in the three filter tips indicated approximately hydrostatic pressures from a groundwater level at +12.5 metres, which corresponds to the lower boundary of the coarser sand and silt layer and which was about 2.5 metres above the water level in the river at the time of the measurements.

The stability was calculated for three sections, the two sections where the new slides had occurred and one in between. For the latter section a safety factor of 0.98 was calculated, i.e. 1.0, for a slip surface similar to those that had failed in the other sections. The calculated safety factor for a larger slip surface that included the nearest building was only 1.1. The calculated safety factors were larger for the sections where slides had occurred, and the steepest parts towards the river thereby had been flattened out. In spite of this, the safety factor for a new slip surface encompassing the nearest house in the southern slide area was calculated as only 1.2. All stability calculations were performed as undrained analyses.

4.3 STABILISING MEASURES

The recommendation by GF was that an excavation of the ground surface should be made to a depth of 1 to 2 metres over the whole area between the riverbank and the buildings. GF also recommended constructing erosion protection along the riverbank over the whole distance. The extent of the excavation was a compromise between what was desired from a stability point of view and what desirable and possible from other aspects. From stability aspects, it would have been desirable to reach a calculated safety factor of 1.5 for slip surfaces involving a house. The corresponding requirement for other potential slip surfaces would have been safety factors of 1.3. On the other hand, the excavations could not be made so deep that the ground surface would be flooded at high water levels in the river or entail other problems for the future use of the land.

The recommendation was followed in principle but the design was varied somewhat in concord with the wishes of the landowners. The excavation at the Brosätter nursing home was thus modelled in such way that the main building stands on a slightly protruding spit of land with excavations reaching further in on the ground at both sides. On this lot, there were also some installations that were not easy to move and reconstruct at short notice. These were therefore left as small hillocks on the excavated terrace, Fig. 132.

Other examples of excavations in accordance with the landowner´s desires are shown in Fig. 133b. The excavation in Fig. 133b, for example, was staged in several small terraces. Since the excavated areas were to continue to be used as gardens, drainage systems were installed to lead off the water in the superficial layers to the river. This was done over the whole area except for the lot furthest to the north, where no garden extends down to the river.

The recommended erosion protection was constructed by rock-fill. It was also extended to encompass both sides of the river, and a smaller excavation was also performed on the western side of the river, Fig. 134.

a)

b)

Fig. 132. The excavated area at the Brosätter nursing home
a) protruding spit of land on the terrace with the main building
b) hillocks with remaining installations in the form of a deep well and a tank for treatment of waste water

a)

b)

Fig. 133. Excavated areas on the eastern side of the river south of the bridge.
a) the ground closest to the bridge
b) excavation in the form of several terraces furthest to the south.

Fig. 134. **Erosion protection and excavated area on the west side of the river north of the bridge.**

4.4 NEW INVESTIGATIONS

4.4.1 Observations

The inspection in connection with the present investigation revealed that the erosion protection was intact. However, it was to a large extent overgrown by grass and bushes. Several trees, some of them apparently older than the erosion protection, were also found down at the toe of the riverbank. The lawns and gardens had been re-established on the excavated terraces and the ground in general gave a very neat impression. The exception is the northernmost part of the excavation, which is not used as a garden. No drainage system was installed here, which is something that the landowner regrets today, and the ground has a marshy character, Fig. 135.

When the new investigation started in March 2001, the surface of the excavated terrace in front of the nursing home was fairly soft and the free groundwater level

Fig. 135. **The northernmost part of the excavated area with the lawn in front of the nursing home in the background. The tracks of the crawler drill rig can be seen on the ground surface with free water standing in them.**

was close to the ground surface. This time coincided with the thawing of the ground after the winter and the superficial soil layers consist of silt susceptible to frost-heave. It is also very uncertain whether there was any effect of the drainage system at that time of the year since the ends of the drainage tubes in the erosion protection at the river were observed to be blocked by ice plugs.

The contour of the back scarp of the slide in 1989 can still be discerned in the northern part of the excavated area. It is seen as an extra stage and runs in a bend from the border of the nursing home lot to the northern border of the excavation, Fig. 136. The erosion protection appears to be extended here and the whole surface of this lower stage is covered by coarse erosion-resistant material. This is now overgrown by grass and bushes.

Fig. 136. The northern part of the excavated area. The scar from the slide in 1989 can be seen as an extra stage just behind the very low riverbank in this part.

4.4.2 Location of the new investigations

The new investigations were located in the northern part of the area, which was most thoroughly investigated in the previous projects. The purpose was to make investigations in natural ground behind the excavation, on the excavated terrace and below the river bottom. The second investigation point was to be located at the centre of a large excavated terrace to enable measurements of possible changes in the soil properties as a result of the excavation. This point was therefore located on the lawn in front of the Brosätter nursing home and relatively close to one of the investigation points in the investigation by GF before the excavation. The exact position was also selected in such a way that any influence from the protruding split of higher ground at the main building and the small remaining hillocks could be minimized. The time of the investigation was governed by a request from the landowner, who planned to replace the old tank for treatment of wastewater and thereby to perform a fairly large new operation in the ground. The new investigations were therefore started at this point in March 2001 when part of the ground was still frozen.

The requirements for the other two investigation points, below natural ground surface and below the river bottom, were that they should be easily accessible and as unaffected by human activities as possible. The upper point was therefore located behind the excavation in the northernmost part of the area, where the ground consists of farmland. The investigations below the river bottom were performed directly to the north of the excavated area and the slide in 1989. This part of the river bottom may be assumed to be unaffected by recent slides and the raft for the drill rig here could also be secured in position by ropes to sturdy old trees growing on both sides of the river, Fig. 137.

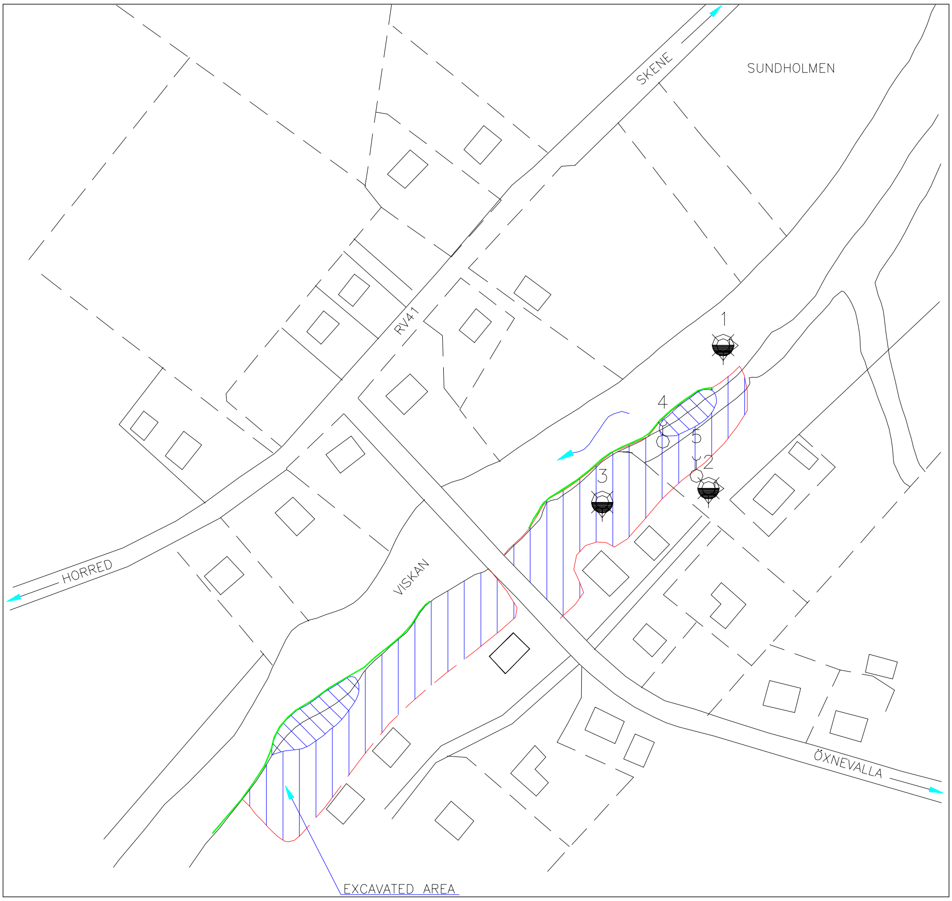


Fig. 137. Location of the new investigations in Sundholmen.

4.4.3 Field tests

The field tests comprised CPT tests and field vane tests at all three investigation points. Dilatometer tests were performed in the points at the natural ground surface and below the river. Closed systems for measurement of pore pressure were installed at two stations, one located at the upper crest behind the excavated area, the other directly behind the riverbank.

CPT tests

The CPT tests were performed with a special clay probe with a capacity, accuracy and resolution adapted to soft soil. In the test below the natural ground surface, where the upper coarser layer of sand and silt remains, a pre-test was performed through this layer using a more robust CPT probe. The first test, which was performed on the excavated terrace with a surface about 2 metres below the surrounding ground, was continued as deep as possible with the available equipment. This meant that no extra arrangements for friction reduction, signal amplification or anchoring apart from the usual pair of screw anchors were used. This test reached a depth of 56 metres, which is about 58 metres below the original ground level. The purpose of this deep penetration test was to check that the shear strength increased continuously with depth and that no weaker layer exists and also to check that there is no permeable layer that can affect the pore pressure conditions. The results show that below the remaining upper layer of about 4 metres of silt and organic soil there is low-permeable clay with gradually increasing stiffness throughout the penetrated profile. The results also show that the clay varies with depth, with more or less distinct borders between different layers. Very distinct transitions are found at depths of 40 and 48 metres below the original ground surface and other borders can be discerned at depths of 17 and 27 metres, Fig. 138. The corresponding levels are –25 and –33 metres and –2 and –12 metres respectively. The uppermost of these transitions was also possible to discern in the results of the field vane tests in the previous investigations.

The CPT test at the natural ground surface was terminated at a depth of 25 metres since no layers of importance for the general assessment of the stability had been found at deeper levels in the first test. Down to this depth, the results showed a roughly 3 metre thick layer of sand and silt followed by a soft low-permeable soil with a significant change in character at about 6 metres' depth. The CPT test below the river bottom, which was performed from a raft without anchoring in the vertical direction, had to be stopped at a depth of about 20 metres below the water level. The results show that the soil in this profile consists of clay covered by a decimetre or so of coarser friction soil. The water depth at the time of the test was about 4 metres. The results of these tests are shown in Fig. 139.

The results from the CPT tests are very even and show clearly that the same type of soil is found at all points unless it has been eroded or excavated away.

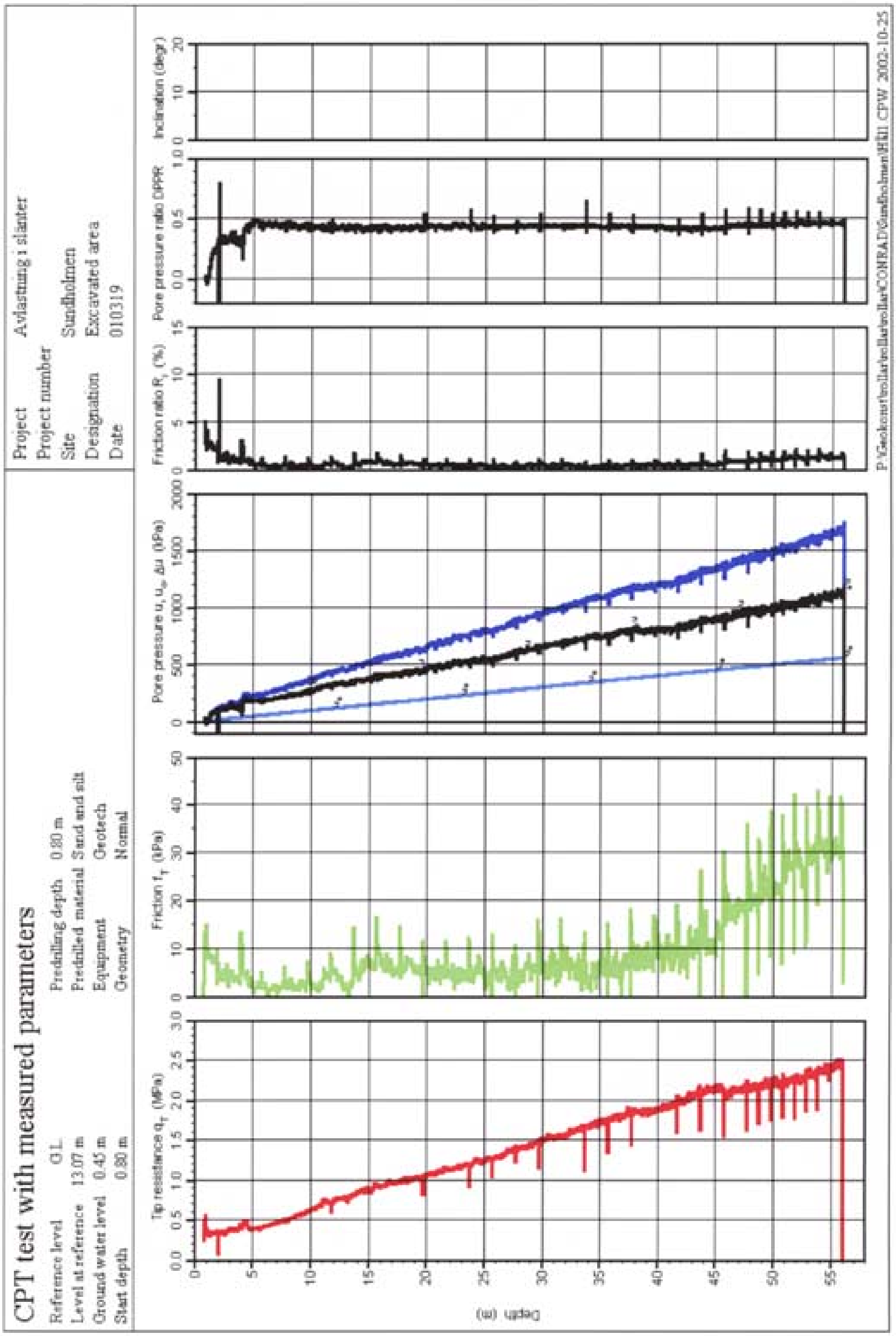


Fig. 138. Results of the deep CPT test on the excavated terrace.

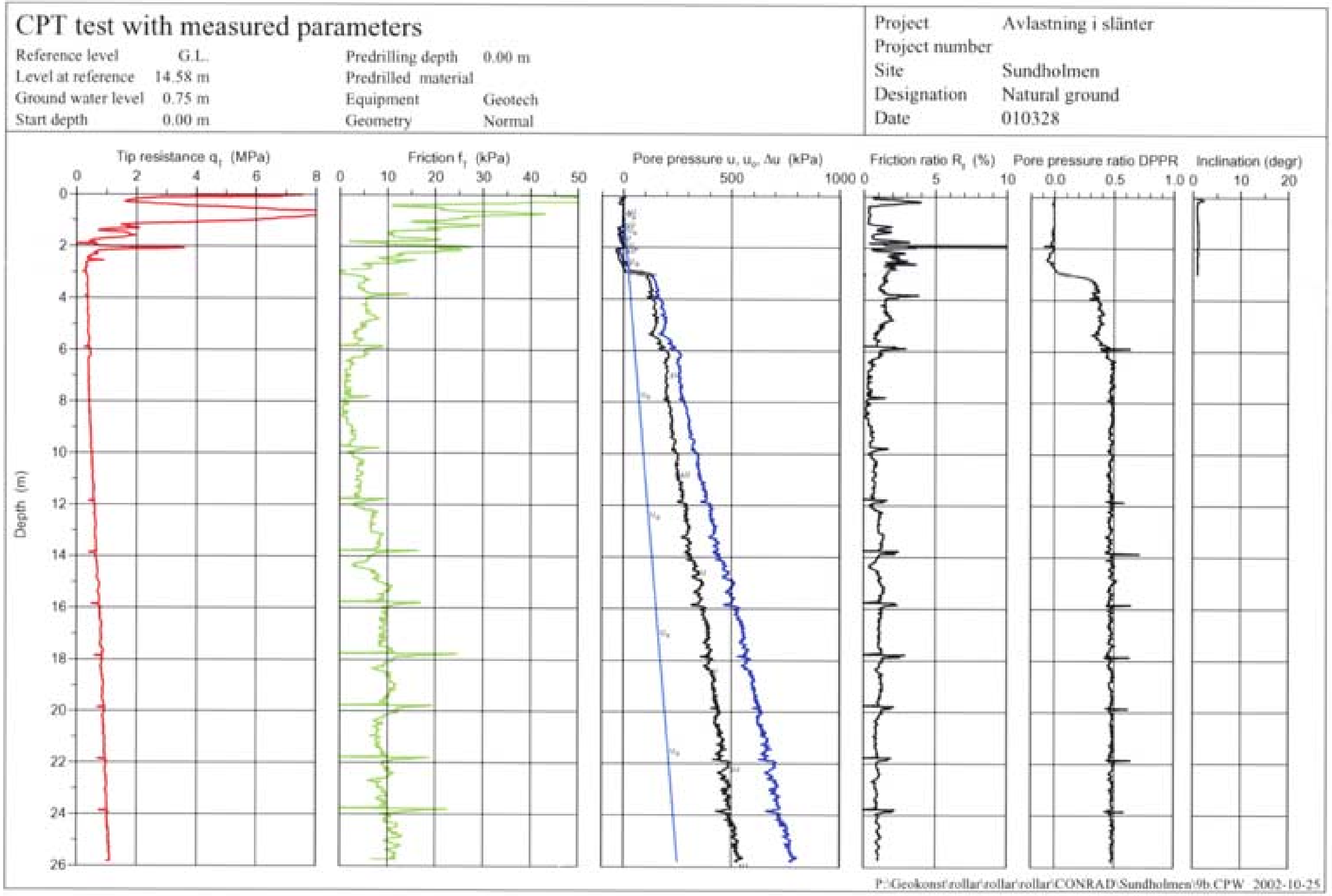


Fig. 139. **Results of the CPT tests below natural ground surface and the river bottom.**
a) below natural ground surface

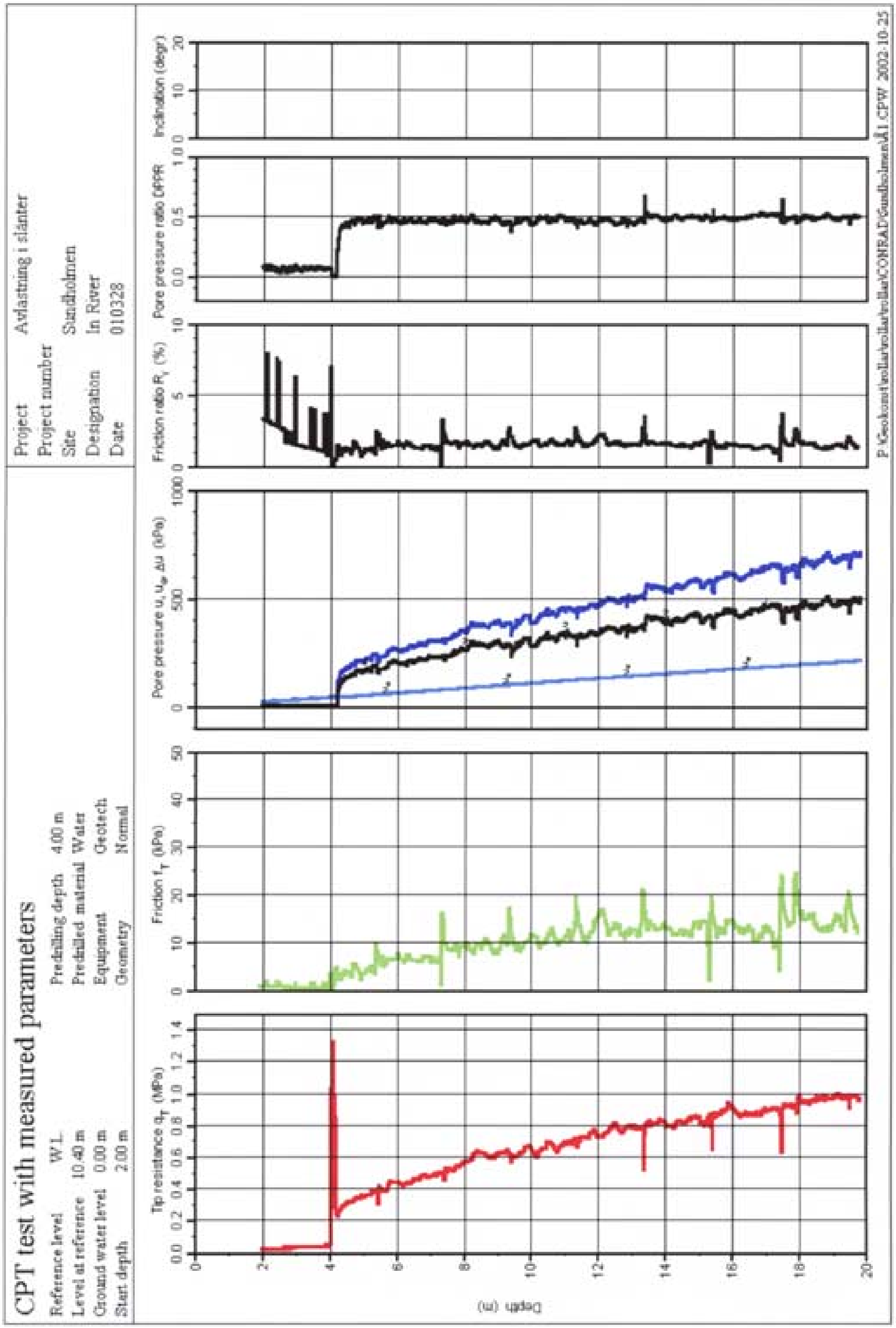


Fig. 139. **Results of the CPT tests below natural ground surface and the river bottom.**
b) below the river bottom

Dilatometer tests

The dilatometer tests in the test point on the natural ground surface were also terminated at 25 metres' depth. The results show little less than 3 metres of sand and silt followed by partly organic cohesive soil down to about 6 metres' depth. Below this, the results indicate normally consolidated or only slightly overconsolidated clay throughout the investigated depth, Fig. 140.

The tests below the river had to be terminated 21 metres below the water level. The results indicate overconsolidated clay, which is heavily overconsolidated in the uppermost layers, Fig. 141.

Field vane tests

Field vane tests were performed at all points down to a level of -10 metres, that is, 25 metres below the ground level in the surrounding area. The tests were repeated at an adjacent point at all three test locations. However, this doubling of the tests was not performed to the full depth at all points.

The tests were performed with the SGI equipment and a vane of normal size. A special study belonging to the afore-mentioned project concerning the influence of the vane size was also conducted below the natural ground surface at the upper test location. There are thus even more results available from this point.

Pore pressure measurements

Closed systems for measurement of pore pressures were installed in two stations in the investigated area. They were placed in the northern part of the area just outside the garden of the nursing home. One station here was placed directly behind the upper crest above the excavation and one down at the riverbank directly behind its crest. Three systems were installed in each station: one at a level of about –10 metres, one at a relatively shallow depth which was still assumed to be safely below the free groundwater level and one approximately midway between these. The closed systems are of the BAT type. Readings of the pore pressures have been made on a number of occasions during a period of about one year. The location of the systems can be seen in Figs. 135 and 136.

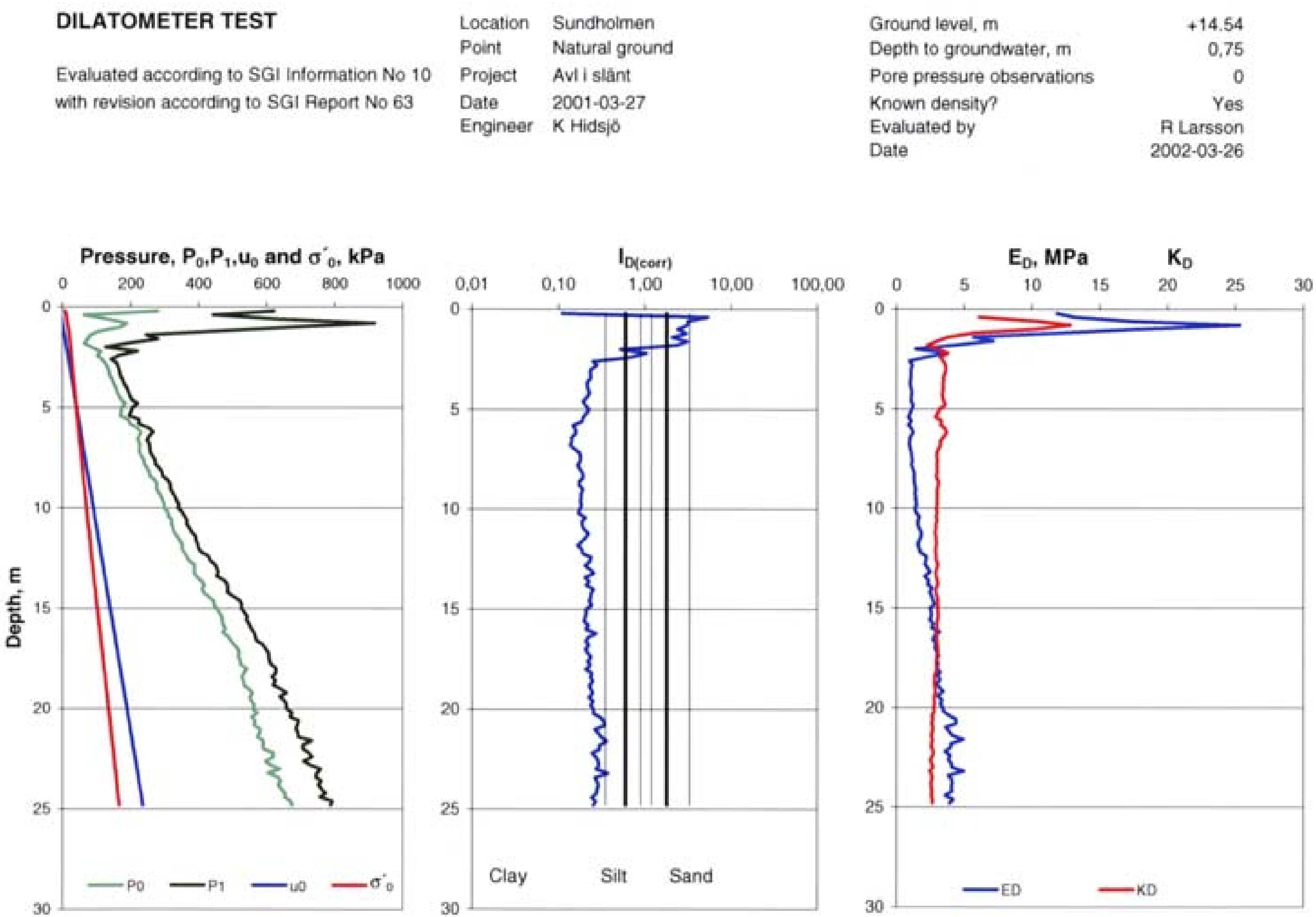


Fig. 140. Results of dilatometer tests below the natural ground surface in Sundholmen.
a) base data

DILATOMETER TEST

Evaluated according to SGI Information No 10
with revision according to SGI Report No 63

Location	Sundholmen
Point	Natural ground
Project	Avl i slänt
Date	2001-03-27
Engineer	K Hidsjö

Ground level, m	+14.54
Depth to groundwater,m	0,75
Pore pressure observations	0
Known density?	Yes
Evaluated by	R Larsson
Date	2002-03-26

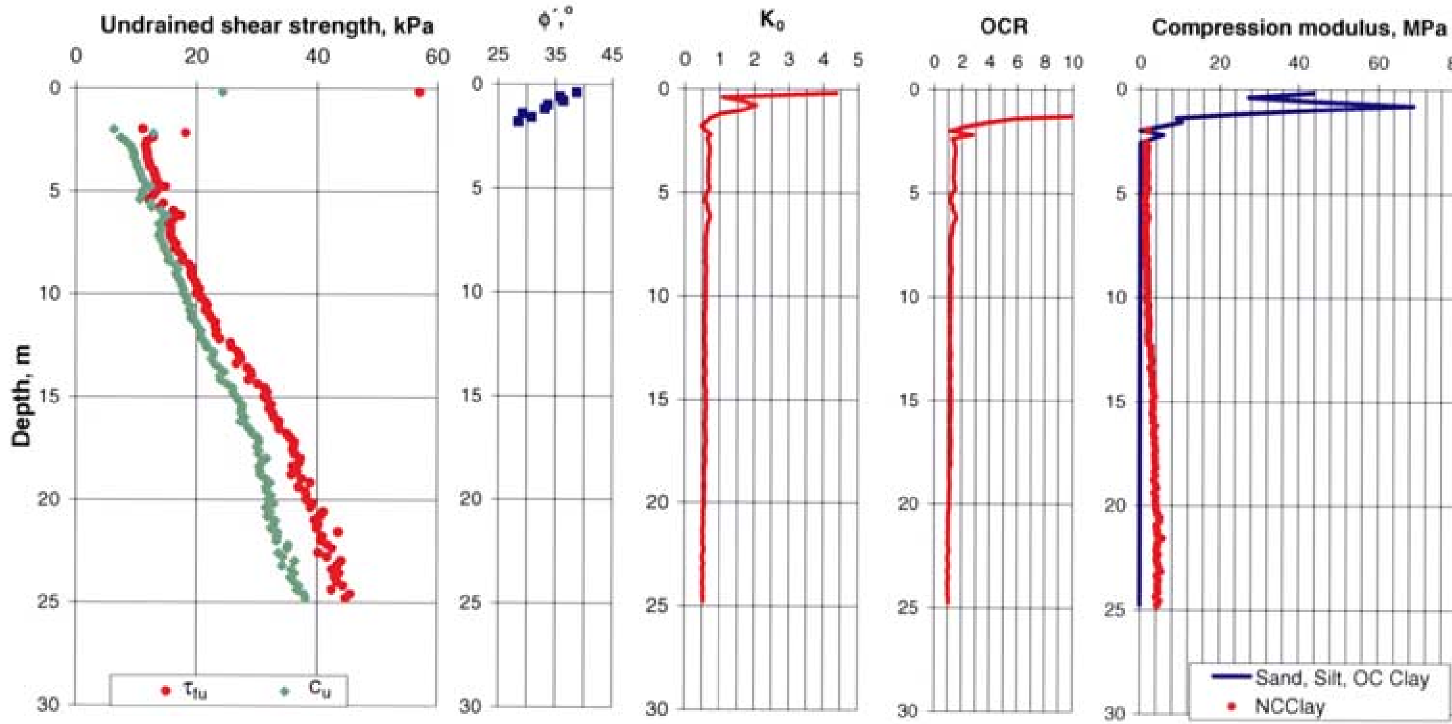


Fig. 140. **Results of dilatometer tests below the natural ground surface in Sundholmen. b) evaluated parameters**

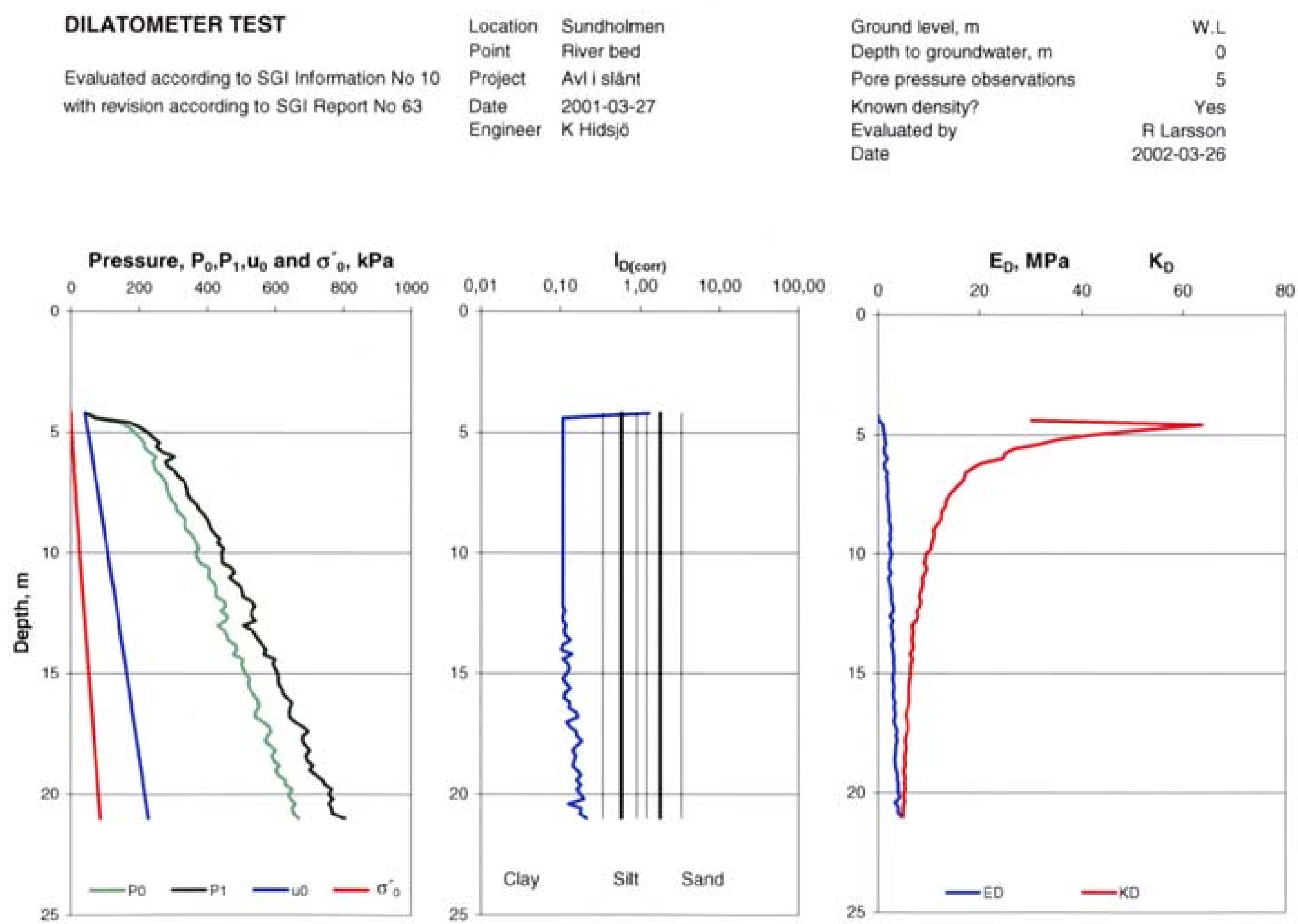


Fig. 141. **Results of dilatometer tests below the river bottom in Sundholmen.**
a) base data

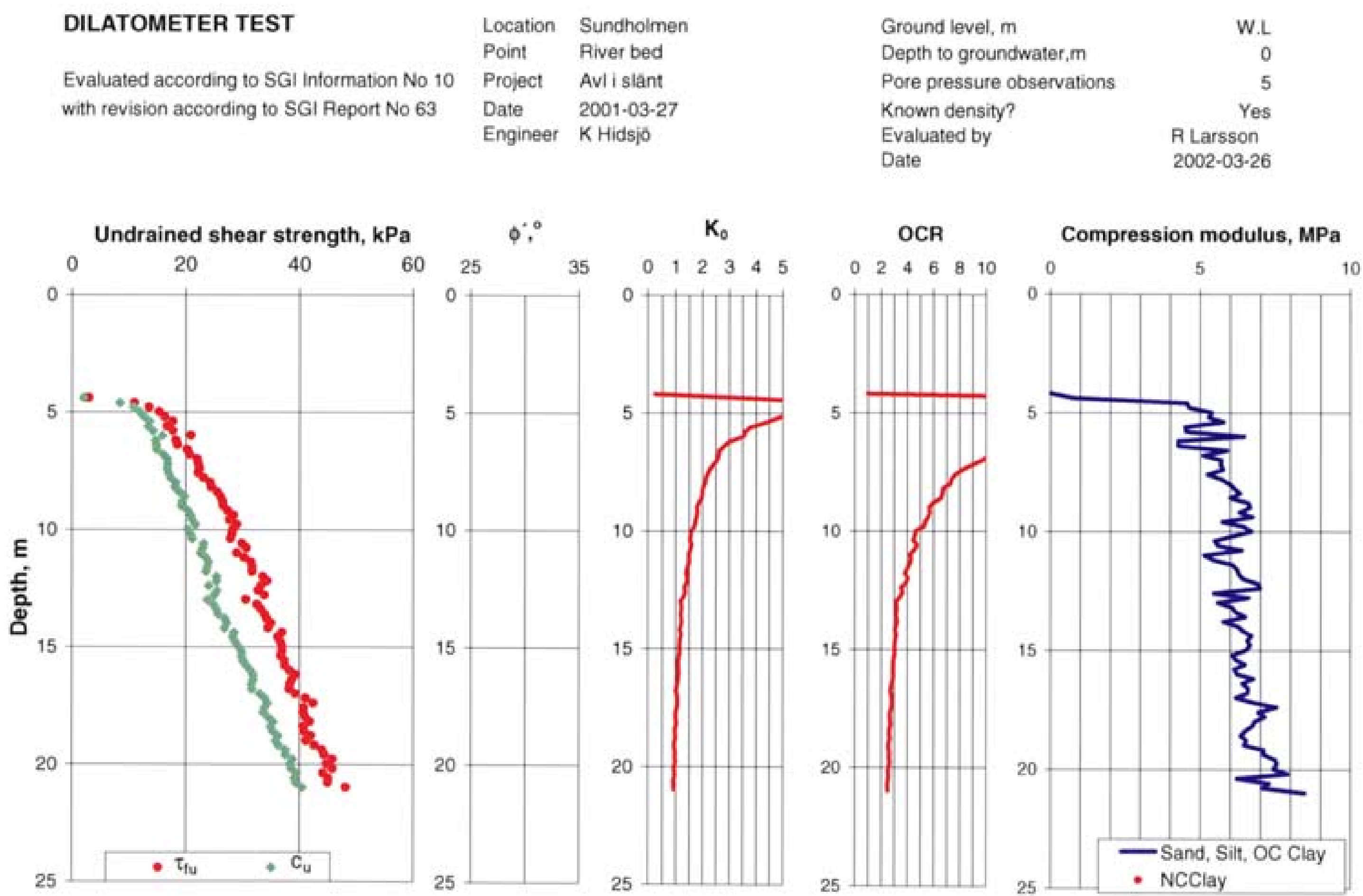


Fig. 141. **Results of dilatometer tests below the river bottom in Sundholmen.**
b) evaluated parameters

4.4.4 Sampling

Undisturbed sampling has been performed with standard piston sampler type St II at all three test points. Samples have been taken at depths every metre down to 10 metres below the ground surface and the river bottom, and then at every second metre. This sequence was followed down to 20 metres' depth below the natural ground surface in the upper test location, whereupon an additional sampling operation was performed at 25 metres' depth. Below the excavated terrace, the deepest samples were taken at 18 metres' depth. The sampling below the river was stopped at 21 metres' depth below the water level, which corresponds approximately to the deepest sampling level below the natural ground surface.

4.4.5 Laboratory tests

All samples have been investigated in the laboratory regarding classification, density, water content, liquid limit, undrained shear strength according to fall-cone tests and sensitivity. The organic content was also determined in samples from the upper part of the profile in order to verify the classification. CRS oedometer tests were performed on a large number of specimens in order to determine the distributions of preconsolidation pressure and permeability, and undrained direct simple shear tests have been performed in order to supplement the shear strength determinations in the field tests.

The undrained direct simple shear tests have been performed on a large number of specimens from the clay layers in the different boreholes. The results have been compiled and normalised with respect to preconsolidation and overconsolidation ratio and in this way local empirical relations have been created. A number of direct simple shear tests have also been performed on specimens from the overlying layer of silty and organic soil.

A few triaxial tests have also been performed in order to verify the applicability of the common empirical relations for shear strength anisotropy and effective shear strength parameters.

4.5 TEST RESULTS

4.5.1 Soil conditions - variations in plan and profile

The results of all investigations have shown that the soil conditions are very homogeneous on the eastern side of the Viskan in Sundholmen. This applies both along the river and within the about 50 metre wide strip from the centre of the river to about 10 metres behind the excavated area that has been investigated. The soil below the natural ground surface behind the excavated area consists of a roughly 3 metre thick layer of sand and silt which has an organic content in its lower parts. This layer is followed by about 3 metres of organic soil (gyttja) and organic clay. The organic content in this part varies between 7.5 and 3.5 % and decreases with depth. Below this, there is normally consolidated or only slightly overconsolidated clay. The clay layer extends to more than 58 metres' depth according to the deepest CPT test and probably to 80 – 90 metres' depth according to observations in connection with installation of deep wells in the area.

According to the results of the CPT tests, field vane tests and dilatometer tests, the clay is divided into relatively thick layers with somewhat different properties. This can be observed from different trends in shear strength increase with depth and different values or trends for the pore pressure parameters *DPPR* and *Bq* in the CPT tests and for the material index I_D in the dilatometer tests. The cause of these variations is difficult to judge because sampling has only been done down to a depth of 25 metres, which is the interval of interest for the stability. However, the differences are probably caused by variations in the grain size distribution in the soil resulting from different conditions during the time when the soil was deposited. Within the investigated depth, there is a point at a level of about –2 metres where the shear strength increase with depth is significantly altered. From the results of the routine investigations in the laboratory, this can be related to the fact that from the top of the clay layer down to this level there is a steady gradual increase in liquid limit, a decrease in water content and an increase in density. Below the level of the change, these parameters are more constant or possibly have slightly opposite trends.

The density in the upper sand and silt layer is between 1.7 and 2.0 t/m^3. It decreases to about 1.5 t/m^3 in the organic soil and then increases to an average value of 1.64 t/m^3 in the clay below. However, a certain regular variation between values of 1.61 – 1.69 t/m^3 can be observed in the clay, Fig. 142.

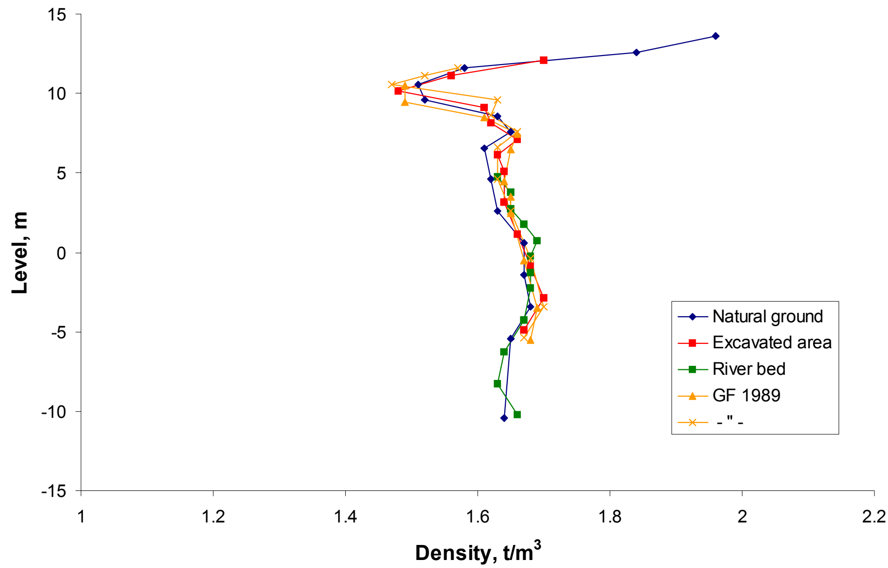
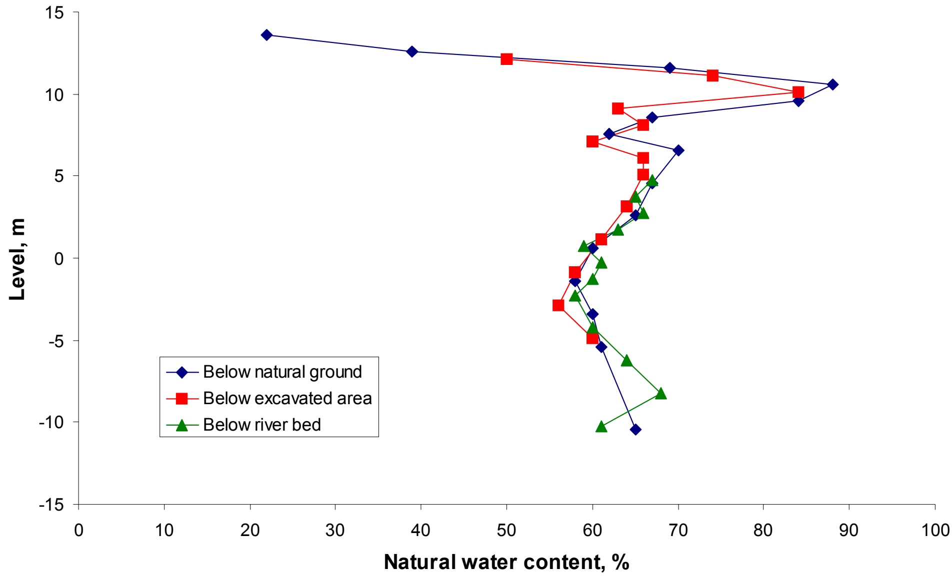
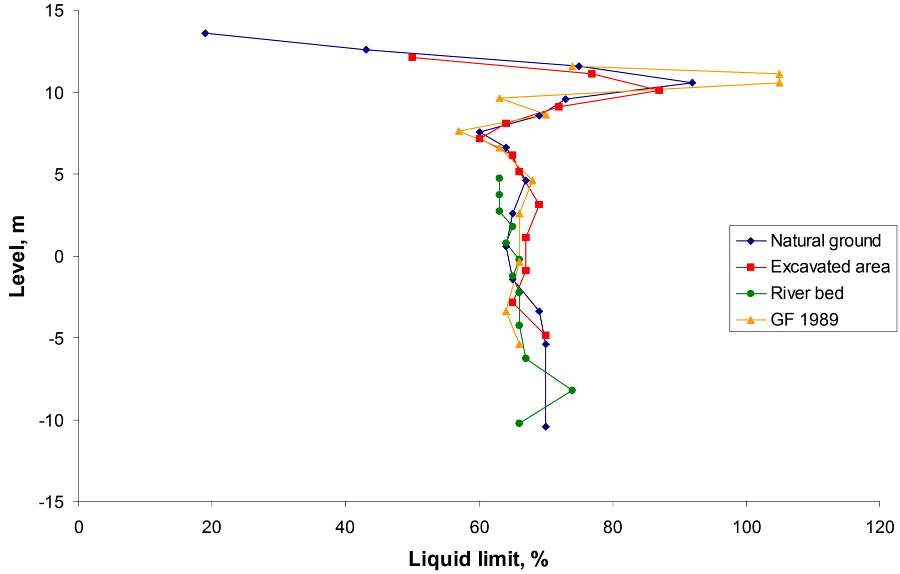


Fig. 142. Measured densities in Sundholmen.

The water content and liquid limit vary with values of 20 – 50% in the sand and silt layer increasing to 80 – 100% in the organic soil and then dropping to being mainly between 60 and 70 % in the clay, Fig. 143. The water content is mainly somewhat lower than the liquid limit and the clay has a normal sensitivity with values between 10 and 20.



a)



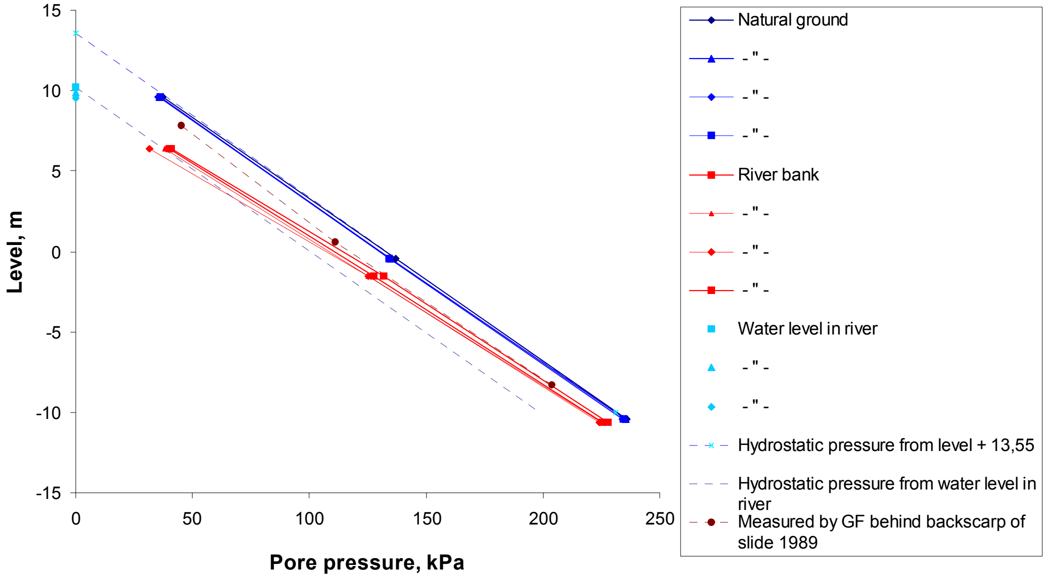
b)

Fig. 143. Measured water contents and liquid limits in Sundholmen.
a) water content
b) liquid limit

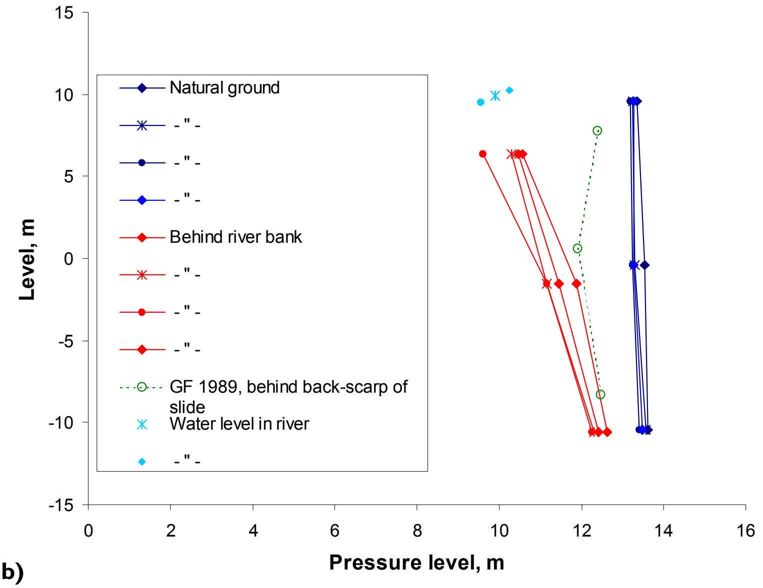
4.5.2 Pore pressure conditions and variations

The pore pressures have been measured on a number of occasions and show a certain seasonal variation. Below higher ground in the area, they are mainly affected by climatic variations whereas the pore pressures in the vicinity of the river are affected by the water level in this. Below the natural ground surface behind the excavated terrace, the pore pressure in deeper layers corresponds to a pressure level located about 1 metre below the ground surface. This pressure level can probably vary in the upper layers and in very dry periods go down to the lower part of the sand and silt layer. However, no such occasion with very dry conditions has occurred during the period of the measurements. At the back of the excavated terrace, groundwater tends to seep out at the toe of the excavated slope except for dry periods. In the gardens, this water is led off by the drainage systems and the free groundwater level is here normally located up to a metre or so below the excavated terraces. However, the drainage tubes may be frozen during wintertime and the groundwater level may then be located close to the ground surface over most of the excavated area. No drainage system was installed in the northernmost part of the excavated area, and here the ground surface is normally wet over about half of its width except for very dry periods. The part of this area closest to the river constitutes the extra stage created by the previous slide. The lower lying ground surface in this part is covered by coarser erosion-resistant material, and any surface water here runs down to the lower boundary of this cover.

In the vicinity of the river, the pore pressures in the upper soil layers are affected by the water level in the watercourse. The influence of this water level is pronounced in the uppermost layers. No extreme water levels have been prevailing at the measuring times, but the variations in water level have then been within 0.8 metres, which also corresponds to the maximum variations in pore pressure that have been registered in the upper soil layers. At greater depths in the vicinity of the river, the pore pressures are more related to the mean water level in the river. There is thus a depression in the pressure levels around the river with lower pressures than at the same levels in the surrounding areas. The lowering of the pressure level increases as the river is approached, and a minimum should occur below the centre of the river bed. The influence of this depression decreases with depth and it becomes insignificant at very large depths. The thickness of the clay layers in Sundholmen is so large that any possible variation in the water pressures in the coarser soil below are unimportant and no permanent lower or artesian water pressure of a size that would affect the pore pressure gradient appear to exist. The measured pore pressures in Sundholmen are shown in Fig. 144.



a)



b)

Fig. 144. Measured pore pressures in Sundholmen.
a) Pore pressure versus level
b) Pressure level versus level

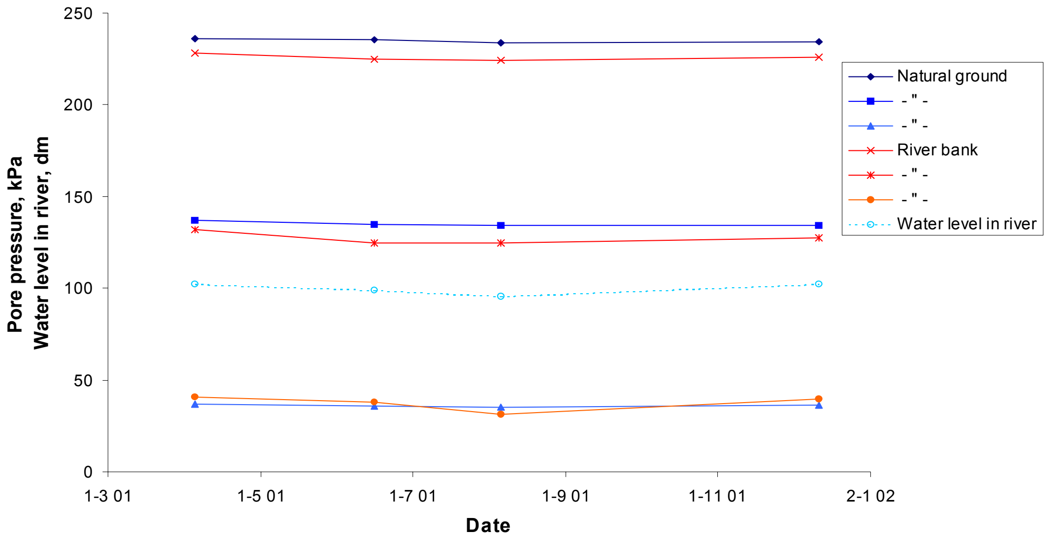


Fig. 144. Measured pore pressures in Sundholmen.
c) Measured pore pressure variation

The permeability of the soil has been evaluated from the results of the CRS oedometer tests in the laboratory. It is about $1 \cdot 10^{-9}$ m/s in the upper organic clay and then gradually decreases to become about $2.5 \cdot 10^{-10}$ m/s at a level of about –5 metres. It then remains fairly constant or possibly appears to increase somewhat with depth, Fig. 145. However, in homogeneous clay, it would rather be expected to find a slight decrease in permeability with depth because of the higher overburden pressures and the accompanying compression of the soil. The variation in permeability reflects the variation in water content and thereby also the variation in void ratio in the soil. The permeability of the overlying sand and silt layer is considerably higher.

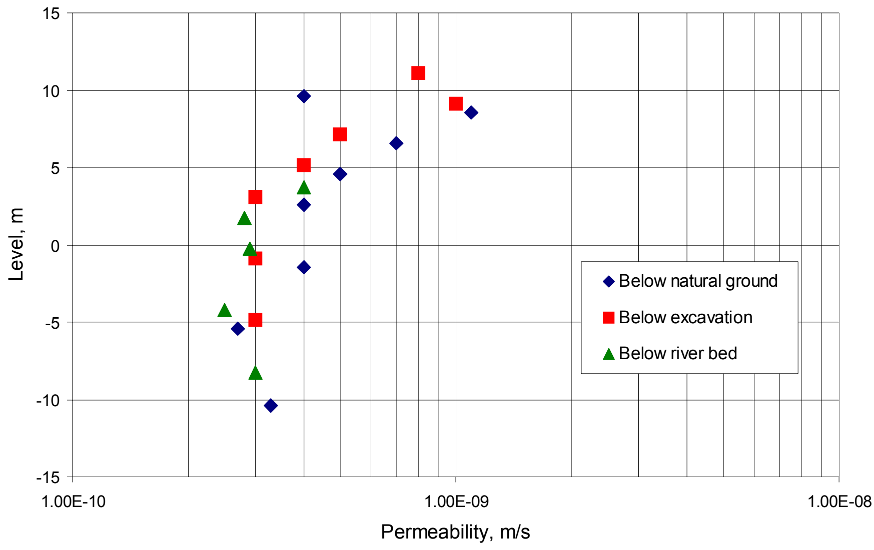


Fig. 145. **Measured values of the permeability in the soil in Sundholmen.**

4.5.3 Stress history and stress conditions

In general, the CRS tests showed the same preconsolidation in all points when the results were plotted versus level, Fig. 146. There is a slight indication of higher preconsolidation pressures below the excavated area, but this cannot be considered to be an established fact. In the figure, a line has been inserted corresponding to the maximum effective vertical stress with a ground surface level with the surrounding area and a free groundwater level in the lower part of the sand and silt layers. The pore pressures below this level have been assumed to gradually adapt to the measured pressure level at 25 metres' depth. Another line corresponding to an overconsolidation ratio of 1.3 in relation to this pressure has also been inserted. This is a normal overconsolidation ratio for this type of soil (Larsson and Sällfors 1995), and corresponds well to the measured preconsolidation pressures.

The stress history can thus be described as the soil having consolidated for the overburden pressure from a ground surface level with the surrounding ground. The corresponding lowest pore pressures correspond to a free groundwater level located about 2.5 metres below the ground surface. The pore pressures below this level are then asuumed to increase gradually with depth until they correspond to a pressure level 1 metre below the ground surface at depths of 25 metres and more. Due to long-term creep effects, the soil has attained an overconsolidation ratio of

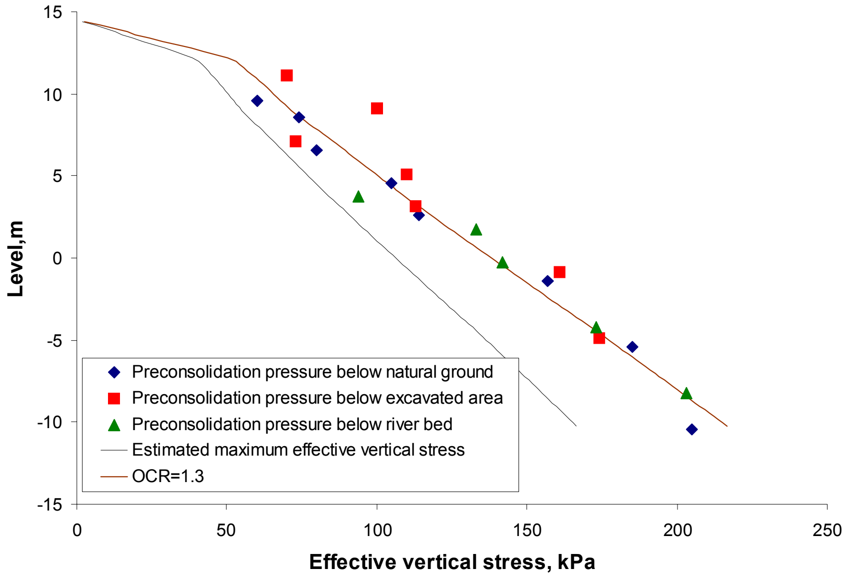
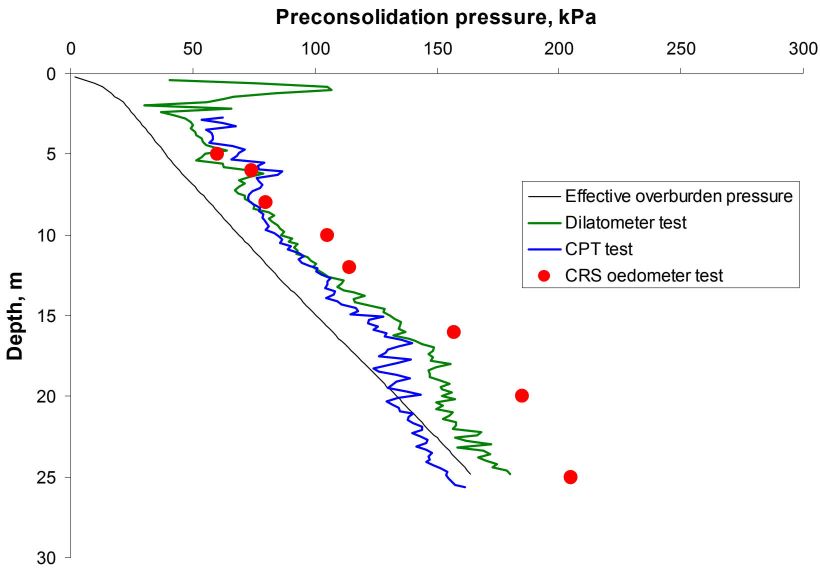


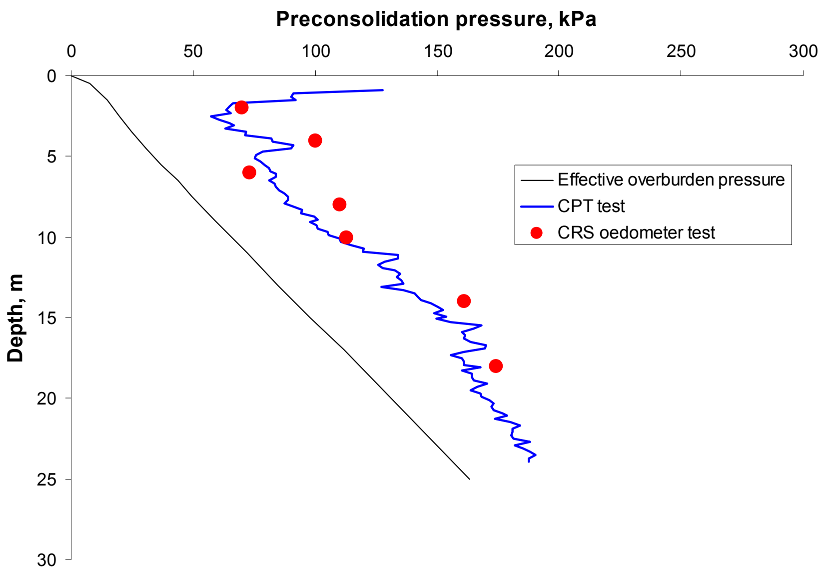
Fig. 146. **Evaluated preconsolidation pressures in soil specimens from Sundholmen.**

1.3 in relation to these maximum stresses. In the previously investigated area at Strandbacken, the clay just inside the former crest of the steep slope down to the river has attained an extra overconsolidation related to the lower pore pressures in this part. A tendency to the same condition in Sundholmen can be observed in the results of the CPT tests and the oedometer tests, but this is too weak to be considered as established. The thickness of the overlying sand and silt layer is smaller in Sundholmen and so are the possible effects of this condition.

The preconsolidation pressures have also been estimated from the results of the dilatometer tests and CPT tests. This has been done for the CPT tests at all test points, and once again it turns out that the preconsolidation pressure can be evaluated from the net cone resistance without any correction for the overconsolidation ratio, Figs. 147 a-c. A slightly better estimate is obtained from the dilatometer tests with the previously described evaluation method (see Chapter 3.8.3). Both types of tests indicated lower preconsolidation pressures below the natural ground surface than under the excavated terrace and the river bed, Fig. 147d, which also affects the evaluated shear strengths from these tests. This trend of a lower preconsolidated pressure being evaluated below the natural ground surface would not have been altered by the previously applied correction for overconsolidation, but would instead have been enhanced.



a)



b)

Fig. 147. Evaluated preconsolidation pressures from CPT tests and dilatometer tests.
a) below natural ground surface
b) below the excavated terrace

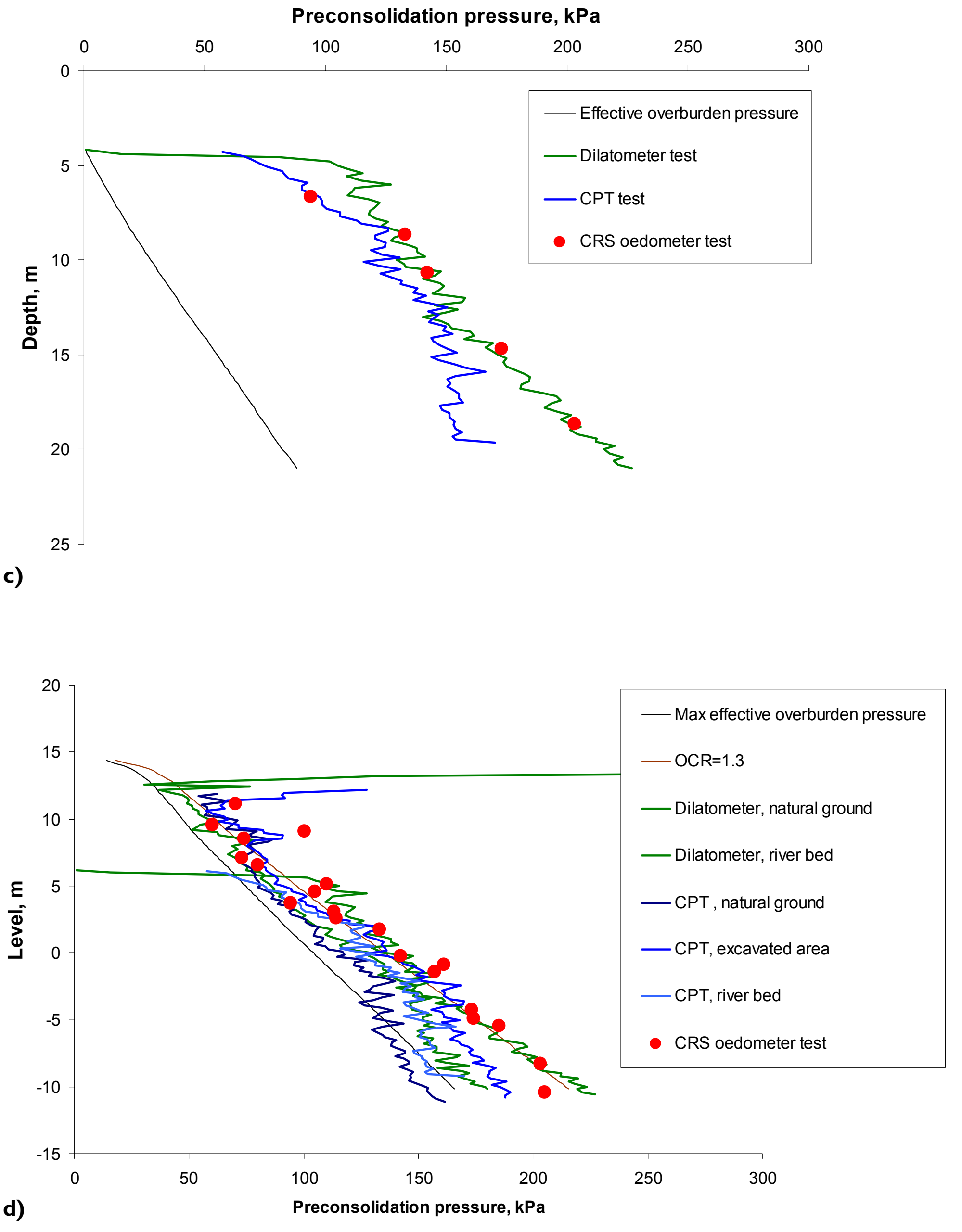


Fig. 147. Evaluated preconsolidation pressures from CPT tests and dilatometer tests.
c) below the river bed
d) compilation of the evaluated values from the field tests versus test level

One problem involved in drawing detailed conclusions about the relations between the evaluated preconsolidation pressures and undrained shear strengths is that the vertical stress is involved in the evaluation. The excavated area and the river-course are fairly narrow and the distance from the test point in the natural soil to the crest above the excavation is small. The influence of the conditions in the other parts of the slope can thus be significant, particularly at greater depths. This should, for example, mean that the net cone resistance from CPT test and the evaluated preconsolidation pressures and shear strengths below the river would be somewhat overestimated, whereas they would be somewhat underestimated below the natural ground. The influence on the results obtained below the excavated area is even more difficult to estimate.

4.5.4 Shear strength

The undrained shear strength was measured by field vane tests, which were repeated at the three test points. The tests were performed down to levels of about –10 metres, which is 25 metres below the surrounding ground surface. The results from all field vane tests are very similar when plotted versus the test level, Fig. 148. An effect of the unloading can only be seen at the most superficial test level below the river bottom. The results also agree very well with the results of previous investigations on this side of the river, Fig. 149.

In the laboratory, the undrained shear strength has been determined by fall-cone tests in the routine investigations and a number of direct simple shear tests. The latter tests have been divided into two groups: tests on the clay and tests on the overlying gyttja. The results from each group have been compiled and normalised versus effective overburden pressure and overconsolidation ratio. According to these compilations, the undrained shear strength in the clay can be expressed by

$$c_u = 0.205\,\sigma'_v\, OCR^{0.86}$$

, Fig. 150.

The corresponding equation for the gyttja is

$$c_u \approx 0.3\,\sigma'_v\, OCR^{0.86}$$

The relation for the gyttja is more approximate since it is based on only 4 tests. However, it fits well into previous experience from this type of soil (Larsson 1990).

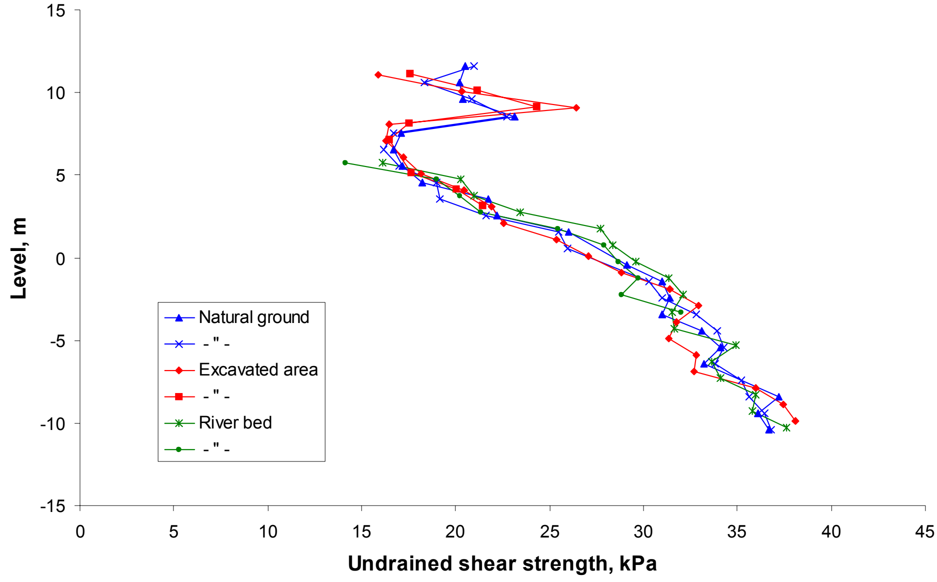


Fig. 148. **Results of field vane tests in the present investigation in Sundholmen.**

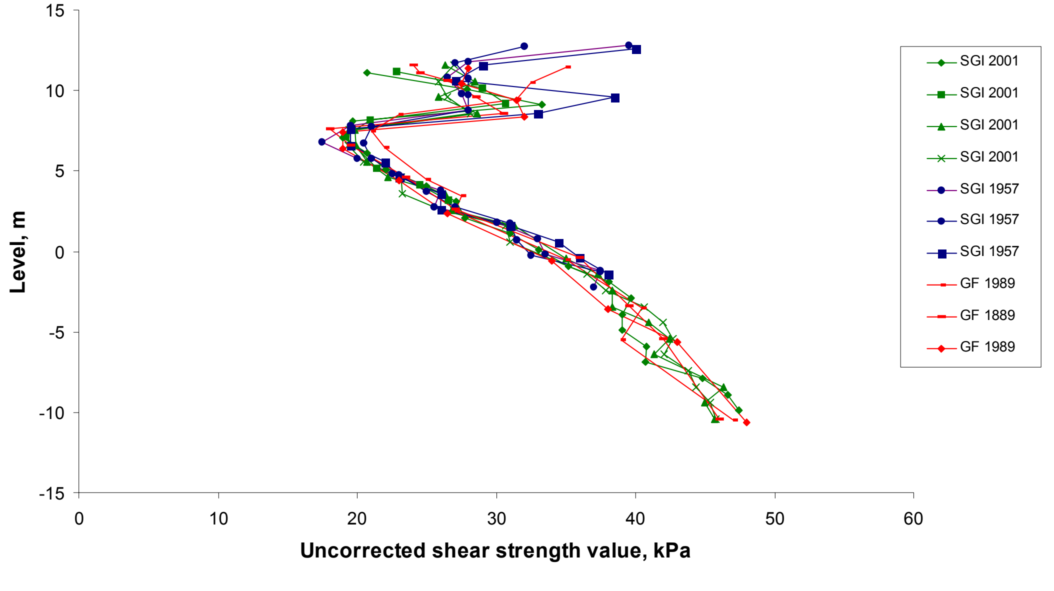


Fig. 149. **Results of field vane tests in all investigations on the eastern side of the river in Sundholmen.**

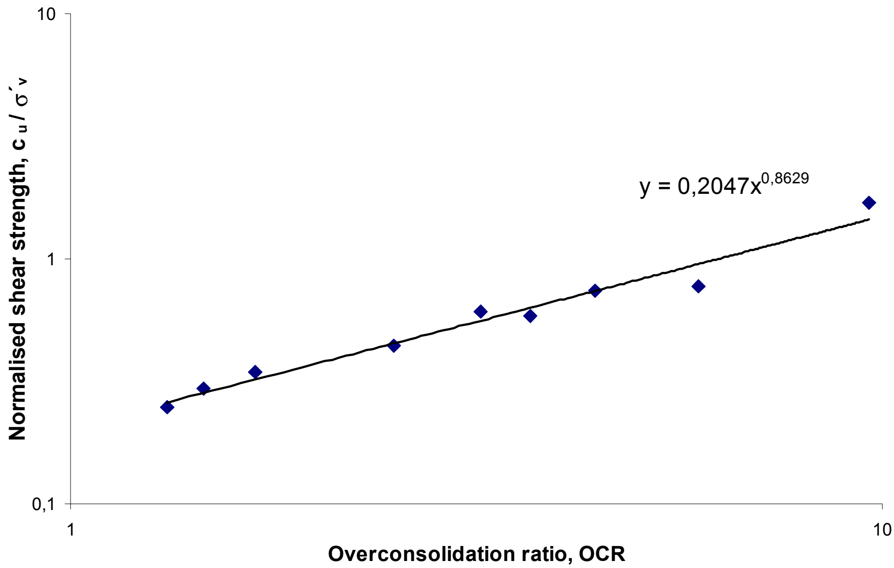


Fig. 150. Normalised undrained shear strength in the clay in Sundholmen determined by direct simple shear tests.

These two equations constitute a local empirical relation for how the undrained shear strength varies with effective overburden pressure and overconsolidation ratio.

The undrained shear strength has also been evaluated from the CPT tests and dilatometer tests. The agreement with the other test results is generally good, but certain significant discrepancies are also observed. In the test point at natural ground surface, the soil is mainly normally consolidated and only slightly overconsolidated. Here a very good correlation is obtained in the clay between field vane tests, CPT tests, dilatometer tests with the alternative evaluation, fall-cone tests and direct simple shear tests, Fig. 151. Only the results of the dilatometer tests using the first evaluation method and the values estimated by the general empiricism based on preconsolidation pressure and liquid limit give somewhat higher values. In the overlying gyttja and organic silt, the field vane tests and the fall-cone tests give significantly higher values than the other methods whereas the alternative evaluation of the dilatometer tests gives lower values when the normal a-factor of 0.22 is used in the evaluation.

Approximately the same relations are obtained in the tests at the excavated terrace, apart from a somewhat higher scatter and the fact that no dilatometer tests were performed here, Fig. 152.

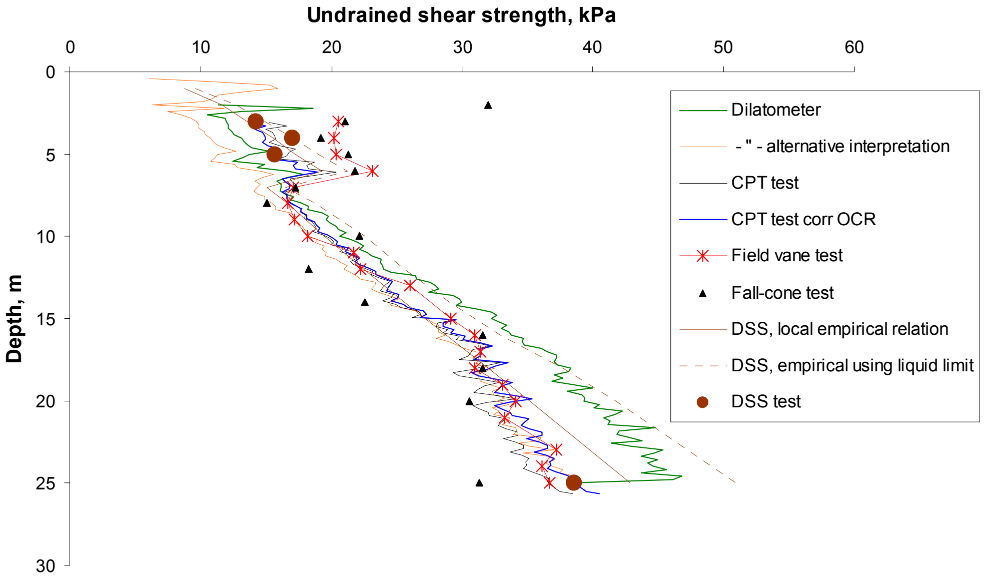


Fig. 151. Measured values of undrained shear strength in the upper test point on the natural ground surface in Sundholmen.

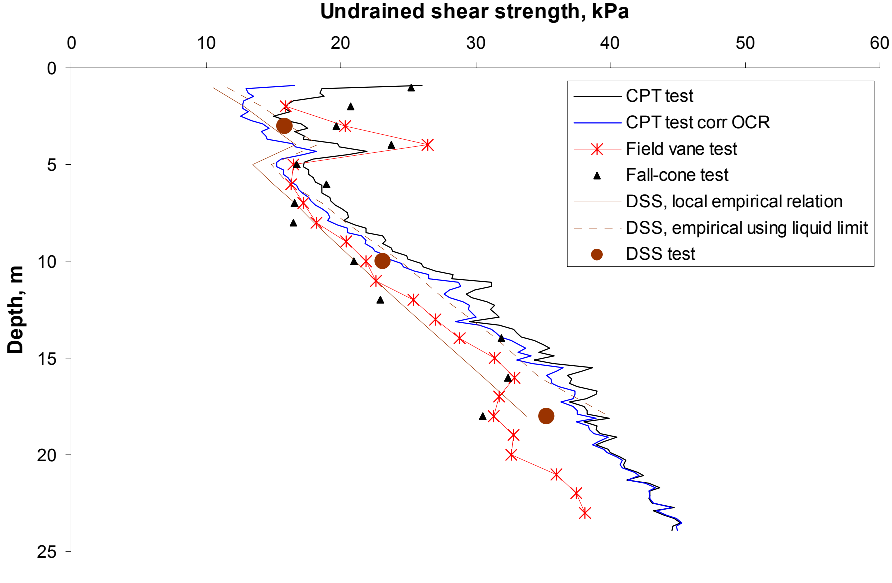


Fig. 152. Measured values of undrained shear strength below the excavated terrace in Sundholmen.

Below the river bed, there is good agreement in the results from CPT tests, fall-cone tests, dilatometer tests with the alternative evaluation and direct simple shear tests after the results of the CPT tests have been corrected with respect to overconsolidation ratio, Fig. 153. The field vane tests, dilatometer tests with the first evaluation method and uncorrected CPT tests show considerably higher values. The results of the field vane tests and the uncorrected CPT tests then gradually approach the results of the direct simple shear tests as the overconsolidation ratio decreases with depth, whereas the results of the dilatometer tests evaluated by the first method remain high. These relations are much the same as those obtained below the river at Strandbacken.

Drained and undrained triaxial tests have also been performed in order to verify the relevance of the empirical relations for active undrained shear strength and effective strength parameters. However, only a few tests were performed since preliminary stability calculations showed that the possible effect of shear strength anisotropy is small. The results of these tests are in general agreement with the empirical relations.

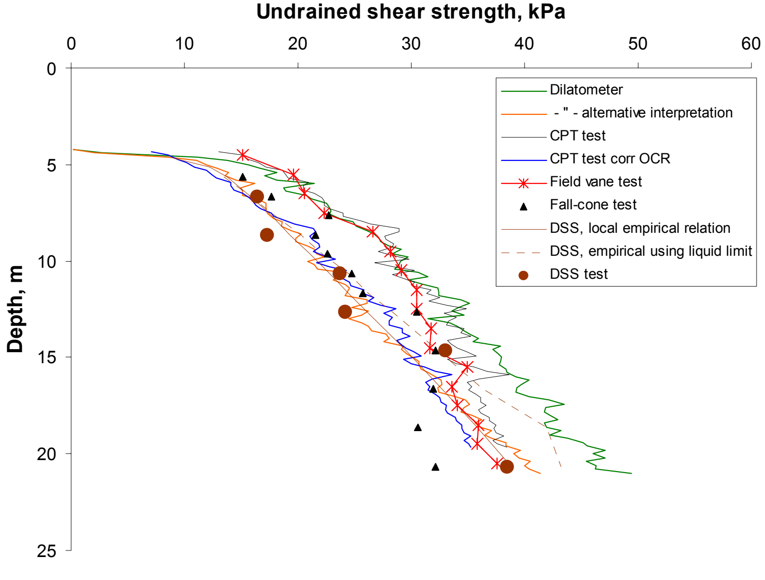


Fig. 153. **Measured values of the undrained shear strength below the river bed in Sundholmen.**

4.6 CHANGES IN SHEAR STRENGTH

As has already been shown, the results of the field vane tests are very similar regardless of whether they were obtained below the river, below the excavated terrace or below the natural ground surface when plotted versus level, Fig. 148. The preconsolidation pressures are also similar over the area when plotted versus level. No significant effect of the unloading can thus be observed in these results either as an effect of the natural erosion process or the man-made excavation. An effect can only be discerned at the uppermost test point below the river bottom.

A more direct comparison of the effect of the unloading can be made for the results from the field vane tests below the excavated terrace because tests were performed at adjacent points in both investigations in 1959 and 1989. This comparison indicates a marginal effect, but this is too small to be considered as a fact, Fig. 154.

The first method of evaluation of the dilatometer tests likewise did not show any significant effect of the natural unloading. Apart from the uppermost 3 – 4 metres below the river bottom, the results from this point and those from the point at natural ground surface are almost identical when plotted against level, Fig. 155a. However, it should be observed that the evaluated preconsolidation pressures from the dilatometer tests are higher below the river than below the natural ground surface and that the general stiffness estimated from these tests follows the same relation. A somewhat higher effect of the unloading is obtained when using the alternative evaluation method, Fig. 155b, but this relation is also affected by the difference in evaluated preconsolidation pressures, and the evaluated decrease in shear strength below the river bottom is thereby less than would have been expected.

The shear strength evaluated from the CPT tests show a clear trend of an effect of the natural unloading on the undrained shear strength, Fig. 156. This, however, is solely an effect of the correction performed for the overconsolidation ratio. The accumulated results also clearly show the effect of the lower evaluated preconsolidation pressures below the natural ground surface, and without the correction the relations would be largely reversed. Because of the low tip resistance measured at larger depths below the natural ground surface, the evaluated shear strength here also becomes lower than under the excavated terrace. In the upper levels, where a reduction should have been expected below the excavation, a small such reduction is also evaluated.

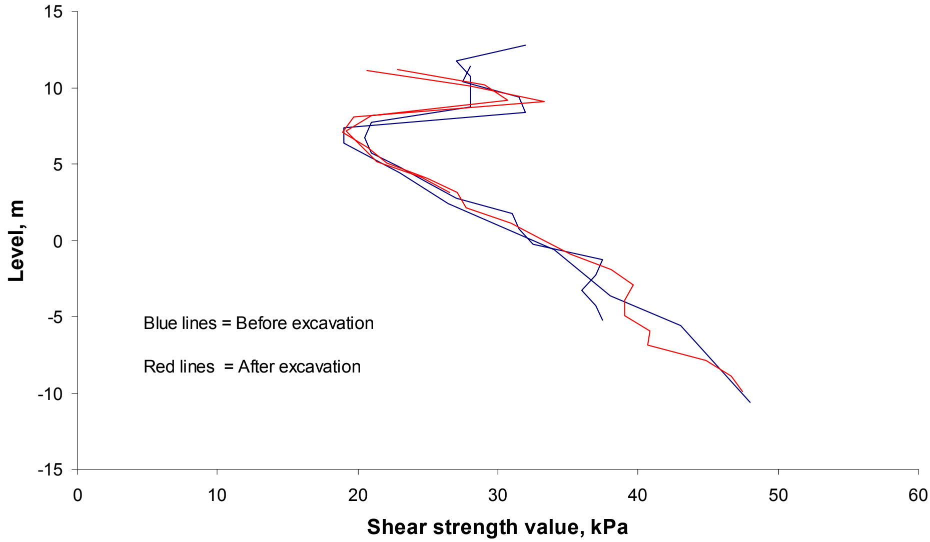
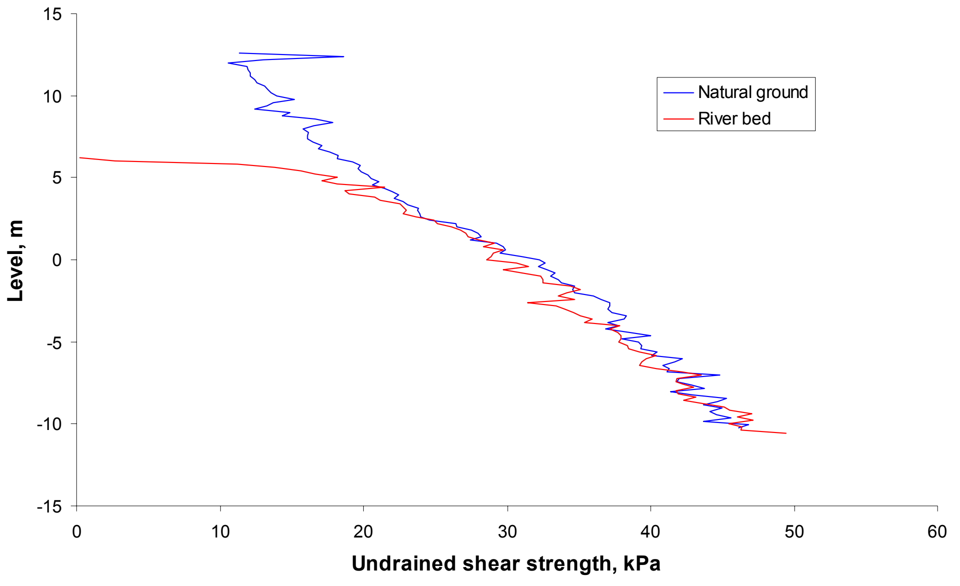
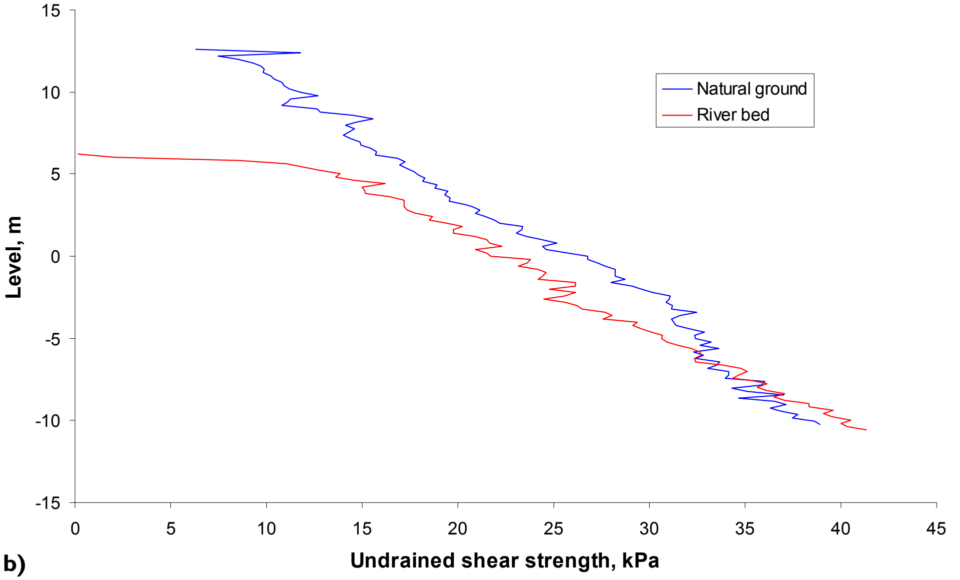


Fig. 154. Results of field vane tests below the excavated terrace before and after the excavation.



a)



b)

Fig. 155. Evaluated undrained shear strength from dilatometer tests in Sundholmen.
a) Evaluation according to the first method
b) Evaluation according to the alternative method

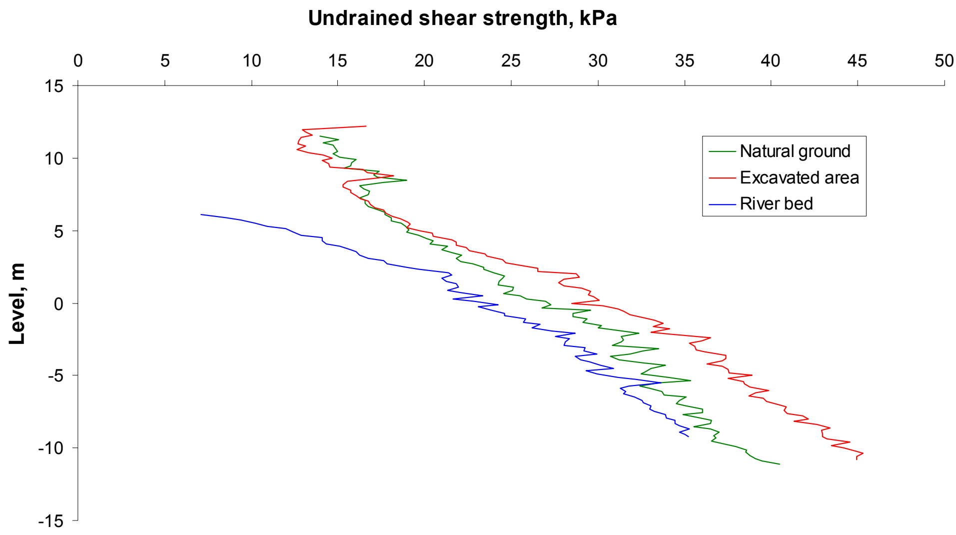


Fig. 156. Evaluated undrained shear strength from the CPT tests in Sundholmen.

The results obtained provide unusually weak and uncertain indications of an effect of the unloading on the undrained shear strength, except for the laboratory tests and evaluations of CPT tests and dilatometer tests, which implicitly assumes that there is such an effect. In this case, the differences in level between the natural ground surface, the excavated terrace and the river bottom are considerably smaller than in the other investigated areas. The fact that the results from both the CPT test and the dilatometer test in the test point located on natural ground behind the excavation show unusually low values in relation both to the preconsolidation pressure and to the results in the other points is probably also a strongly contributing factor to this result.

4.7 STABILITY CALCULATIONS

Stability calculations in this project have been performed for the area in front of the Brosätter nursing home. The topography of the section that was measured in detail in the GF investigation in 1989 and the lowest water level in the river given in the previous investigations have been used to describe the conditions before the excavation. For the conditions afterwards, a typical section based on a survey of the ground in this investigation has been used.

It is somewhat difficult to draw a representative section for the conditions after the excavation, since the excavation has been adapted to existing buildings and constructions and particularly its width can vary strongly within short distances along the river. Furthermore, some existing wells and tanks have been left sticking up as small hillocks on the excavated terrace at the time of the investigation. One of these hillocks is now removed since the waste water system has been reconstructed. An average width has been used for the excavated terrace in the type section. This should be fairly relevant since the width varies within short distances and three-dimensional end-effects should help to smooth out the stability. The small hillocks have been disregarded for similar reasons. The erosion protection has been assumed to be 0.7 metres thick and cover the slope down to the river bottom. Apart from this, the bottom profile in the river has been assumed to be unchanged since 1989.

The undrained shear strength in the calculations has been based on the determinations performed. In cases where these differ, the main stress has been put on the results from the direct simple shear tests. The shear strength in the upper sand and silt layer has been assumed to be drained and when the ground water level has been located below the ground surface, negative pore pressures have been assumed in the overlying silt because of its capillarity. The effect of the latter assumption is very small except for very shallow slips in the riverbank because the free groundwater level is high and in large parts even lies in the ground surface after the excavation.

A modelling of the groundwater level is rendered more difficult since a number of drainage tubes have been installed below the excavated terrace and end in the erosion protection at the riverbank. The exact positions and the function of these tubes are uncertain. In connection with the inspection and investigations, it could be observed that the free groundwater level at the back and the middle of the excavated area seasonally lies in or very close to the ground surface and that ice plugs may block the drainage tubes. Any effects of the drainage tubes have therefore been disregarded in the modelling of the groundwater pressures.

Calculations of the stability with undrained and drained analyses and circular slip surfaces have been performed with the computer program SLOPE/W. The Spencer method of calculation has been used in all of these calculations. Calculations of slip surfaces with more varied shapes, more varied shear strength parameters and combined analyses have then been performed using Janbu's general procedure of slices.

4.7.1 Before the excavation

The initial calculations showed that the calculated factor of safety before the excavation is about 1.0 for both undrained and drained analyses. The undrained analysis results in critical slip surfaces reaching 5 to 10 metres in behind the crest and ending at the toe of the slope at the river bottom, whereas the drained analysis shows lowest values for superficial slips in the riverbank, Fig. 157.

The calculated safety factor increases with distance from the river and approaches a value of 1.3 when the starting point of the slip surface reaches the nearest buildings.

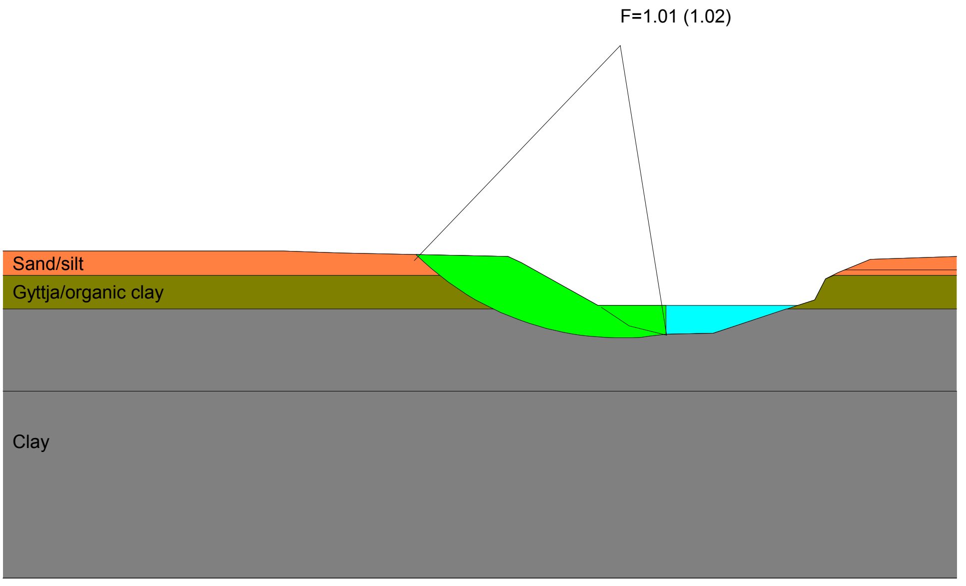
The initial calculations used fairly simple soil models which were based on averages of the values of the shear strengths measured below the later excavated area and the river for short slip surfaces and the measured shear strength below the natural ground surface for longer slip surfaces.

The subsequent calculations using the Janbu method resulted in a safety factor of 1.03 for an undrained analysis with isotropic shear strength. The critical slip surface is approximately the same as that obtained in the initial calculations. This safety factor rises to about 1.12 when the anisotropy is taken into account. A combined analysis on the other hand lowers the calculated safety factor to 0.95, which increases to 1.02 when the shear strength anisotropy is also considered. The drained safety factor also became of the same size as in the previous calculation, i.e. close to 1.0.

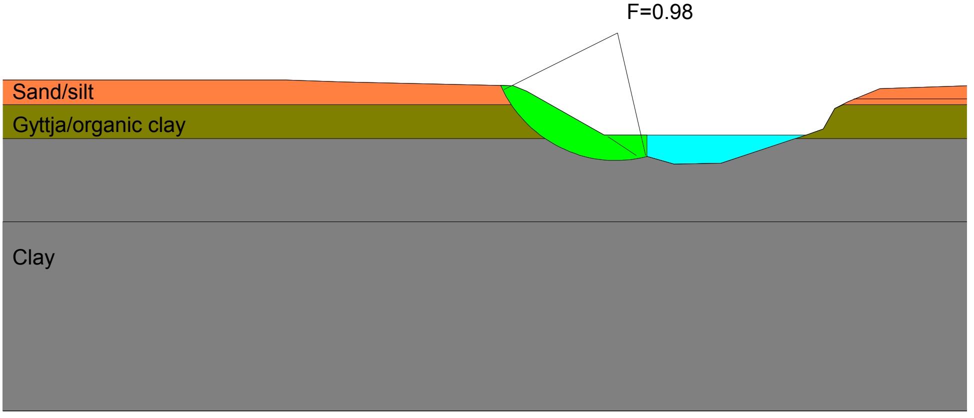
The safety factors in the latter calculations became somewhat lower than in the first ones for longer slip surfaces that reach the buildings. This is mainly a result of a more detailed modelling of the strength variation. The safety factor in an undrained analysis thereby became about 1.18. When anisotropy is considered it increases to 1.23. It decreases to 1.16 in a combined analysis and again increases to 1.18 when the anisotropy is also taken into account.

4.7.2 After excavation

The calculations for the situation after the excavation have been made for two conditions: with and without the erosion protection. This has been done to illustrate the effects of the two stabilising measures, excavation and the weight of the erosion protection, on the calculated stability separately.



a)



b)

Fig. 157. Critical slip surfaces according to calculations with circular slip surfaces using SLOPE/W and the Spencer method.
a) Undrained analysis with isotropic strength
b) Drained analysis
(Values in brackets refer to safety factors obtained in combined analyses using anisotropic shear strength and Janbu's calculation method.)

The initial calculations with undrained analyses and simplified assumptions about the shear strength distribution resulted in a critical shear surface that had moved inwards across the excavated area to encompass part of the nearest building with a safety factor of 1.33, Fig. 158. The erosion protection has no significant effect on this large slip surface. Calculations with the same assumptions give a strong increase in calculated safety factor for slip surfaces reaching about halfway in over the excavated terrace. The calculated safety factor in an undrained analysis for a such surface then amounts to about 1.48 without consideration of the erosion protection and is reduced to about 1.42 when the weight of this is taken into account. The safety factor in drained analyses for relatively shallow slips is strongly dependent on the exact design of the erosion protection and the pore pressure distribution close to the riverbank but can still be assumed to be very low, i.e. close to 1.0.

In the subsequent calculations with the Janbu method and a more detailed modelling of the shear strength properties, the same safety factor of 1.33 was obtained in undrained analyses for slip surfaces reaching in behind the excavated terrace and in below the nearest buildings. This factor increases to 1.36 when the anisotropy is considered, and a combined analysis for a similar slip surface gives safety factors of 1.31 regardless of whether the anisotropy is considered or not.

The safety factor in an undrained analysis for slip surfaces comprising about half the excavated terrace becomes 1.35 increasing to 1.48 if the anisotropy is considered. These factors apply to the case when the erosion protection is not considered. When this is taken into account the calculated safety factors in undrained analyses for slip surfaces of this extent decrease to 1.28 and 1.40 respectively. However, in a combined analysis the increase in safety factor because of the excavation is not that large. Because it was the upper coarser sand and silt layers that were removed, there was no corresponding decrease in pore pressures and the drained shear strength thereby becomes governing to a higher degree. The calculated safety factor for the same slip surface thereby becomes about 1.10 for the first case without considering the erosion protection, and increasing to 1.14 when anisotropy is considered. The effective stress level increases proportionally more than the shear stresses when the erosion protection is considered and the calculated safety factors for this slip surface thereby increase to 1.15 and 1.18 respectively.

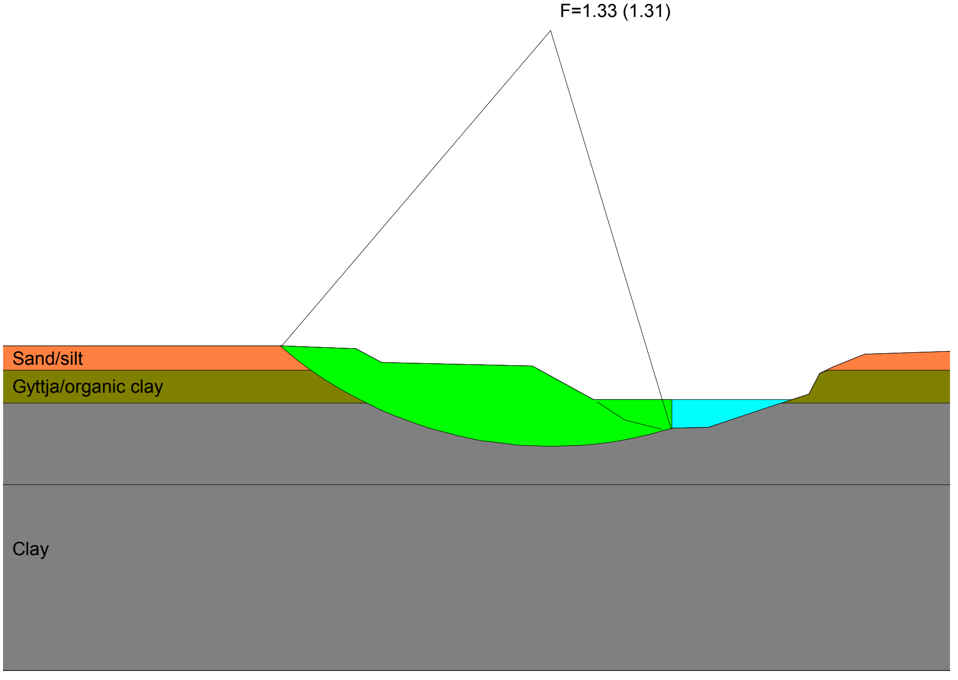


Fig. 158. Calculated critical slip surface after excavation according to undrained analyses with circular slip surfaces and isotropic undrained shear strength. Calculation by the program SLOPE/W and the Spencer method.
(Values in brackets refer to safety factors obtained in combined analyses using anisotropic shear strength and Janbu's calculation method.)

4.7.3 Comments on the results of the calculations

The results of the calculations show that the safety factors required for remaining buildings according to the guidelines of the Swedish Commission on Slope Stability (1995) have barely been reached and that the stability of the outer part of the excavated terrace close to the river is still very low. This is not unexpected since the possible stabilising measures were limited by the risk of flooding at high water levels in the river and the existing buildings, and the measures had been adapted to this. This is particularly the case for the nursing home lot, which is located upstream from the bridge with the highest risk of flooding and where the existing buildings lay closest to the river. The measures taken were therefore less than would have been desirable from a stability point of view alone.

The calculations also show that the works performed primarily have an effect on the stability of slip surfaces with a limited extent behind the riverbank. For such slip surfaces, the excavation has a large effect according to undrained analyses but a considerably more limited effect according to the combined and drained analyses. The latter finding is partly due to the fact that the excavation is not associated with a corresponding lowering of the pore pressures in this particular type of soil profile. The calculations also show that the erosion protection, apart from ensuring the long-term stability, lowers the safety factor in undrained analyses but increases it in the more critical combined analyses.

The effect of the installed drainage system is obviously good from an environmental point of view. The contrast between the lawns on the drained excavated terraces and the marshy ground to the north of them is striking. The effect from a stability point of view is more uncertain since the free groundwater level at times can be observed to be located at or very close to the ground surface over large parts of the terrace and the drainage tubes are seasonally blocked by ice plugs. This means that the calculated safety factor for shallow slips at the riverbank is very low and uncertain. Apart from the pore pressures, it also depends strongly on the exact design of the erosion protection both above and below the water level. This aspect has not been a primary factor for the present project and has therefore not been investigated in detail. The stability of the riverbank is probably also significantly affected by root systems of the rich vegetation with bushes and trees that grow here.

The relatively steep part of the riverbank in front of the nursing home, where the local stability is most uncertain, is closed off by a high fence running parallel to the riverbank a metre or two from the crest. The risk related to a local slip at the riverbank is thereby limited. As a rule, the slope at the riverbank is flatter in the other parts in the area.

Chapter 5. Modelling of pore water pressure

5.1 GENERAL

The pore water pressures have been modelled with aid of the computer program SEEP/W from GEO-SLOPE International (1994). The purpose for this modelling is normally to connect those pressures that have been measured at single points and the boundary conditions into a complete model of the pore pressure conditions within the slope. This model can then be used in the stability calculations. Another purpose would in this case have been to investigate to what extent the measured pore pressure conditions before the excavations could have been used to predict the present pore pressure conditions after adaptation for the new geometries.

However, it is not possible to make a direct such evaluation since the pore pressure observations in the previous investigations were very limited. It is only the conditions afterwards that are known in more detail. The modelling of both the current and the previous conditions has therefore been done under the assumption that the boundary conditions in terms of pore pressures in the bottom layers and pore pressure profiles at larger distances from the excavations have remained unchanged and thus were the same as those measured today. The permeability of the different soil layers has also been assumed to have remained the same. The calculations have then in principle been performed without support from measured pore pressures within the clay profiles; instead the calculated pore pressure distributions have been compared to the measured ones. However, some details in the modelling have been affected by the measured values. It has thus, for example, been necessary to model a highly permeable soil layer on top of the excavated terraces, in which the water that in some cases seeps out at the toe of the excavated slope can flow out over the terrace. It has also been necessary to use the real pore pressure observations as support for the modelling of the two separate aquifers that were found in the sand with embedded clay layers behind the excavation in Section C in Torp.

It has proved to be difficult to model the pore pressure conditions in the superficial parts of the steep lower parts of the slopes. The calculated pore pressures here are highly influenced by assumptions of the permeability in the surface layers with fissures, roots and dry crust effects. They are also strongly influenced by assumptions about the infiltration and surface runoff of water on the ground surface. The general picture of the pore pressure can be modelled fairly well but some details are considerably more difficult to reproduce. This is a serious weakness in the modelling since the pore pressure conditions in the outer parts of the slope are often of major importance for the stability. This can be remedied by closely spaced pore pressure observation points and a more manual estimation in these parts.

The pore pressures in a slope are influenced by precipitation and evaporation. All of the investigated slopes have had delta and lateral fluvial sediments overlying the clay. This has in most cases entailed that in normal conditions water seeps out at the toes of the excavated slopes at the rear of the terraces and flows out over their surfaces. The groundwater level is thereby located in or close to these surfaces and cannot be significantly affected by heavier rainfalls or snow-melting. Only the areas behind the excavations are significantly affected, unless the surface water here is prevented from infiltrating into the ground and raising the groundwater levels by paved surfaces and drainage systems. For the conditions before the excavations, on the other hand, the precipitation can be assumed to have had a significant impact unless drainage systems had been installed in the ground.

It is problematic to simulate the effect of rainfall since the infiltration depends on the condition of the ground surface and its inclination and also on the intensity and duration of the rainfall. However, the calculations show that fairly soon a limit is reached for how much water can be infiltrated without causing the calculations to run away and yield unreasonable results. This limit is far lower than the amount of precipitation that falls during wet periods and also lower that the amounts that are given as guiding values for infiltration capacity in other contexts (Svenska Vatten- och Avloppsföreningen 1983). Larger amounts than the limits found in the calculation may therefore be assumed to run off as surface discharge.

The pore pressure levels are lowered somewhat during dry periods, particularly in the superficial layers because of evaporation and the water take-up by the roots of the vegetation. However, there are no detailed guidelines for this that take into account the particular ground conditions and vegetation. This seasonal groundwater lowering in the upper layers has therefore not been modelled except for an illustration in the slope at Strandbacken. Neither has it any significant importance for the assessment of the stability conditions for which the maximum pore pressures are of primary interest. However, it has a certain importance for the comparison between measured and calculated pore pressures in the upper soil layers.

5.2 MODELLING OF PORE PRESSURE CONDITIONS IN SECTION A IN TORP

The soil conditions and the permeability values that have been measured in the different investigations were used in the modelling of the conditions in Section A. There is some uncertainty about how the thickness of the upper sand and silt layer varies in the area between the upper crest and the test point located 50 metres behind this. This layer is about 5 metres thick at the crest and less than 2 metres far behind, and the exact variation has some importance for the calculated pore pressures in this part of the slope. However, some guidance about this variation is obtained from the results of the geophysical resistivity measurements. In the model, the excavated terrace has been supplied with an upper 0.5 metre thick layer with the same permeability as the sand in order to allow a simulation of the outflow of seepage water over the terrace.

The basic model uses the mean water level in the river, the measured pressure levels in the coarse bottom layers and the measured free groundwater level far behind the crest as boundary conditions. It has also been assumed that no horizontal water flow occurs at the vertical boundary far behind the crest or at the vertical boundary at the centre of the river bed. The model created thus and the calculated pore pressure distribution are shown in Fig. 159.

A comparison between calculated and measured pore pressures shows a generally good agreement, Fig. 160. Only very small variations have been measured because of the slow response in the open pore pressure systems used in this section, and the measured values can be assumed to correspond to the modelled normal conditions. Only the most superficial pore pressure measuring system at the river bank, which can be assumed to have some contact with the water in the river, and the two subsequently installed closed systems show any significant variations.

In evaluation of the comparison, it should be remembered that the pore pressures in the bottom layers are calculated using the pressure levels measured in filter tips in these layers, and the correlation is therefore of necessity good here. All calculated pore pressures in the lower layers are also affected by this fact, whereas they are fairly insensitive to assumptions regarding the top layers.

For the situation before the excavation, it has been assumed that all excavated material has had the same permeability as the remaining sand and silt layers and that the same boundary conditions as for the present conditions apply. These assumptions entail that the groundwater level is raised about 1 metre at the centre of the now

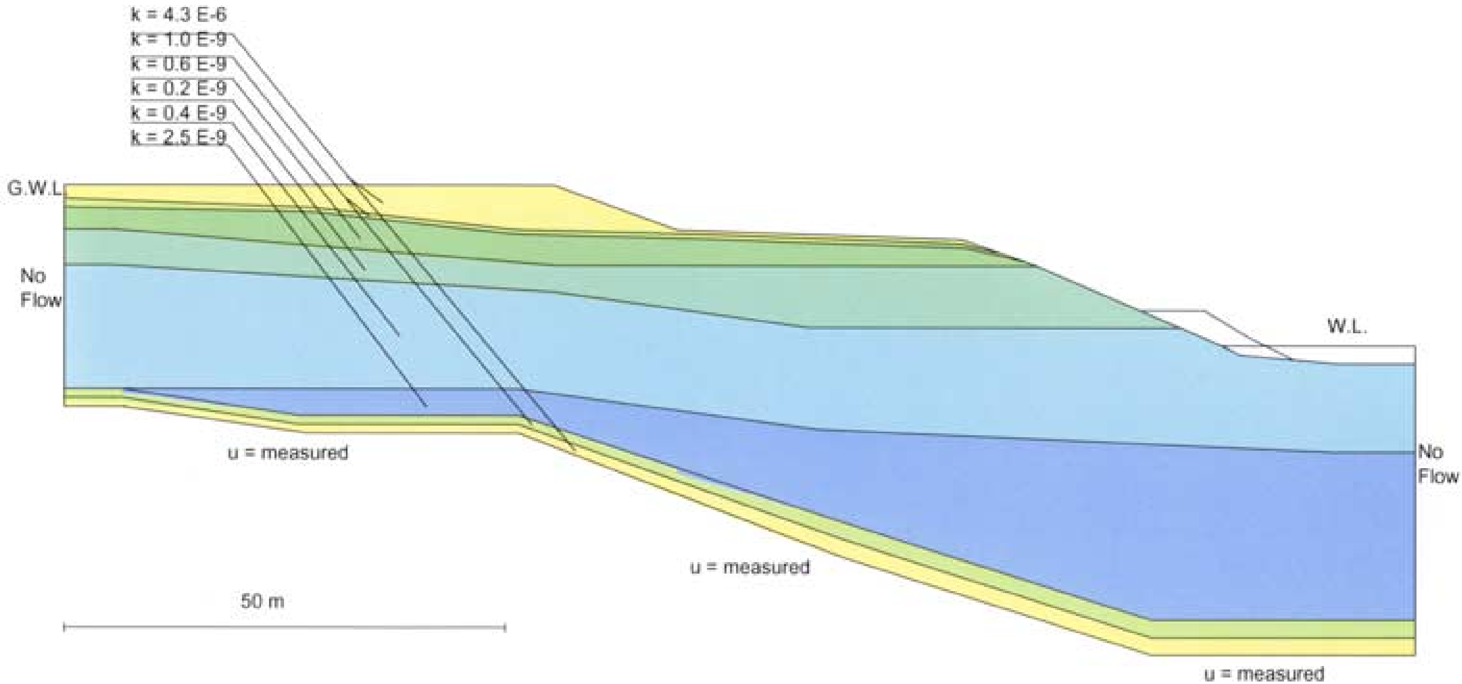


Fig. 159. **Calculation model and calculated pore pressure distribution in Section A in Torp after excavation.**
a) Calculation model

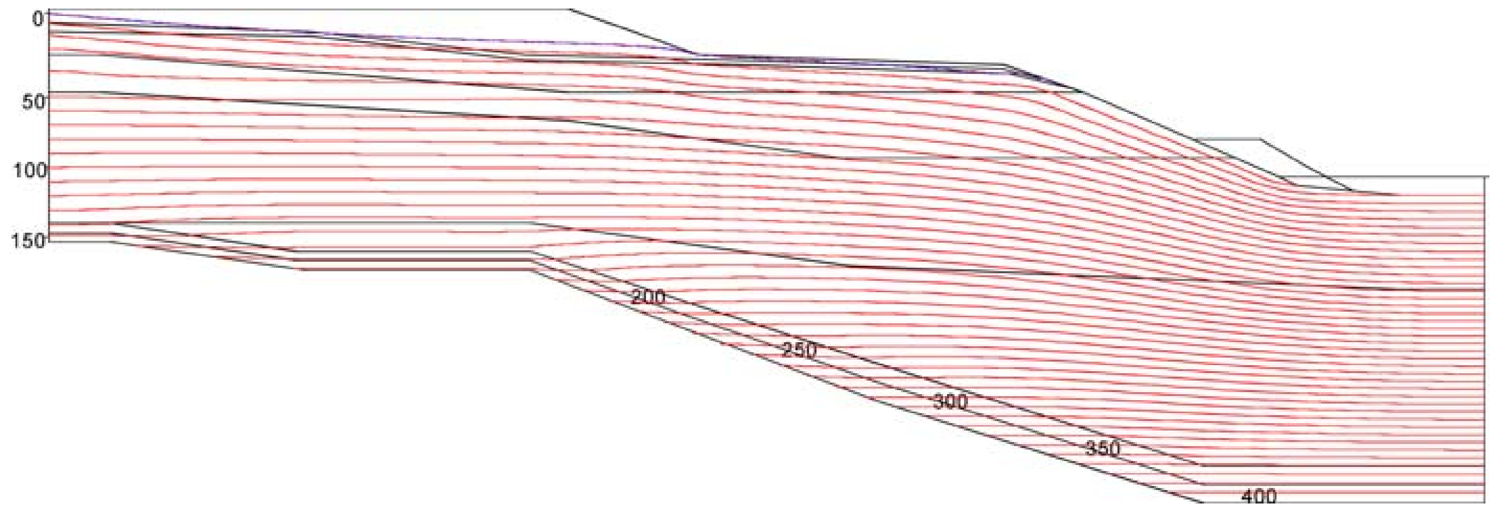


Fig. 159. Calculation model and calculated pore pressure distribution in Section A in Torp after excavation.
b) Calculated pore pressure distribution. Pressure lines for every 10 kPa.

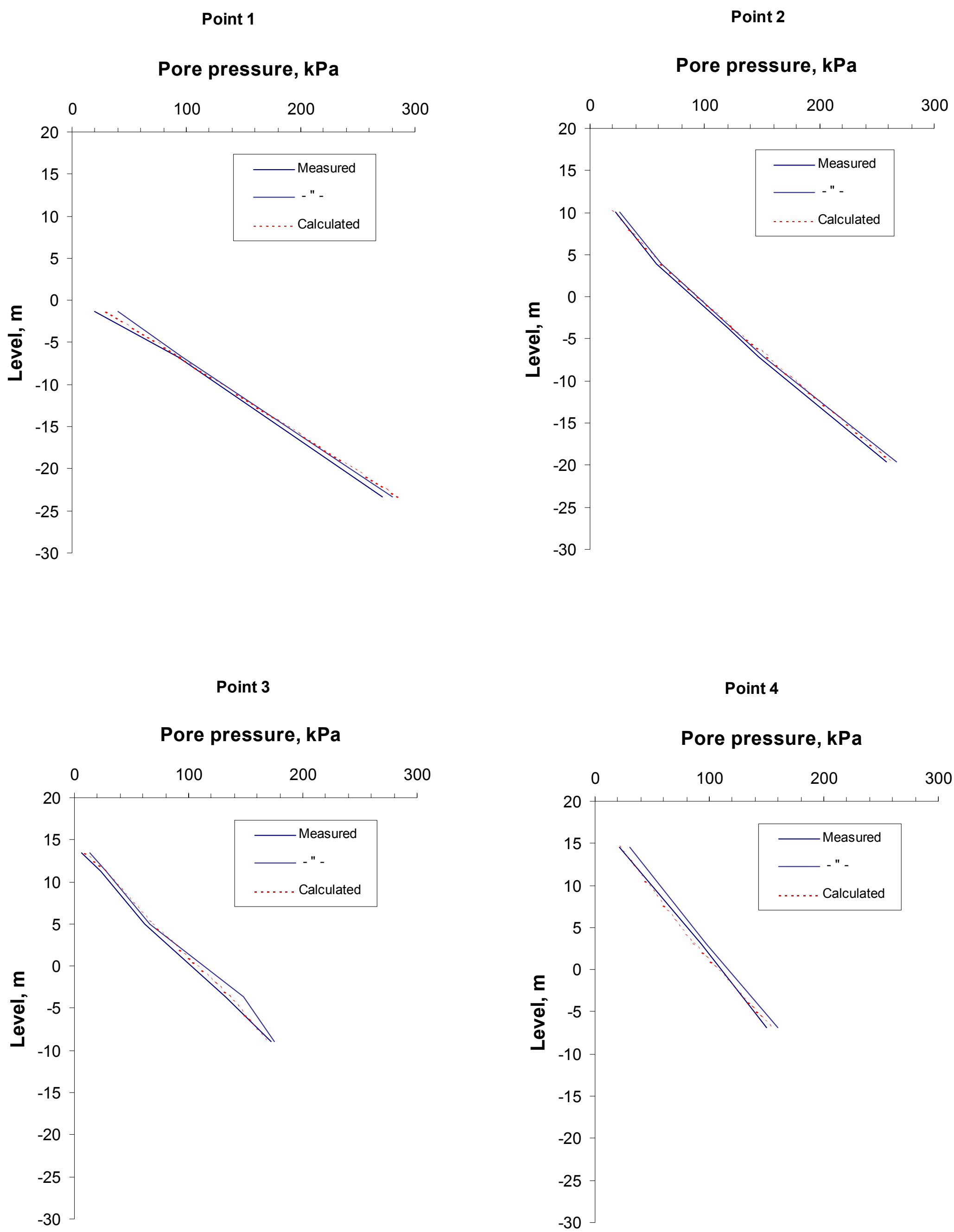


Fig. 160. Comparison between measured and calculated pore pressures in Section A in Torp.

excavated area and an assumption of a prolonged period with heavy rainfall raises it another 1.5 metres at this point, Fig. 161. The only reference value that exists is a pore pressure measurement in the clay at about 9 metres' depth in the vicinity of Point 3. This shows a value that compares well to the calculated one. The other pore pressure system used at that time was installed in the coarse bottom layers at the same point and also shows a corresponding value. This, however, is not due to the calculations but only confirms the assumption that the measured pressure level in this layer has remained the same before and after the excavation.

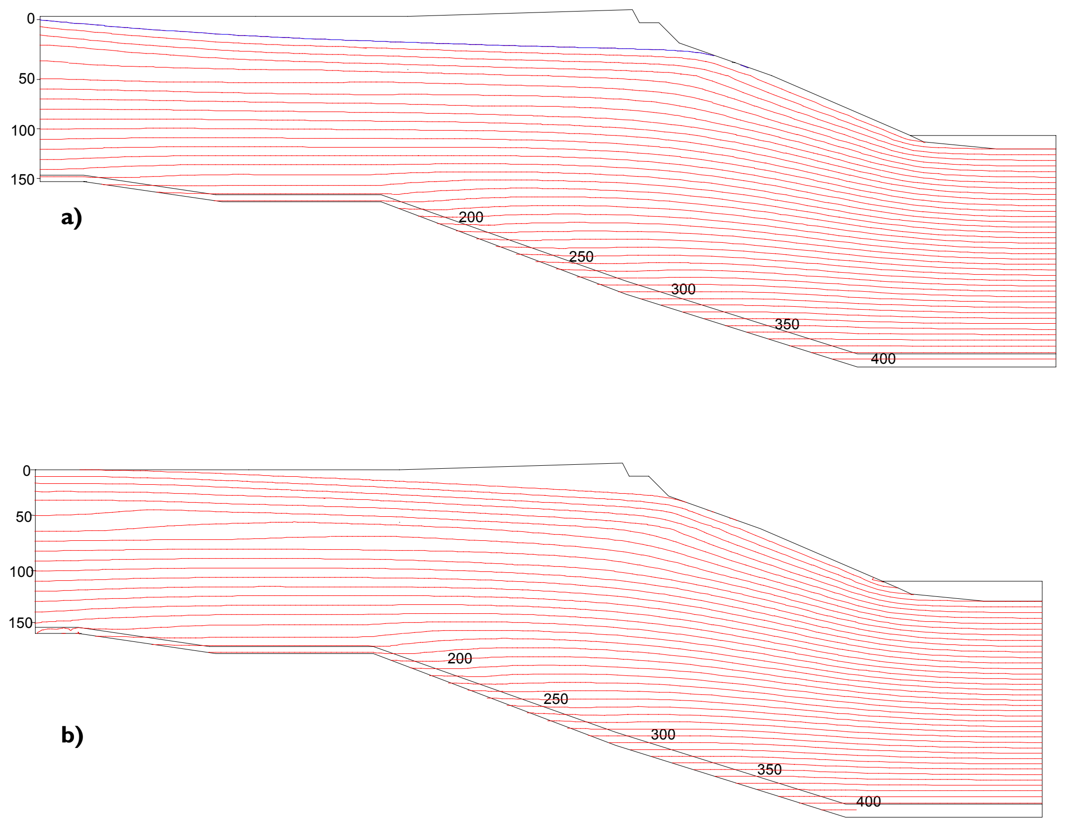


Fig. 161. Calculated pore pressure distribution before the excavation in Torp, Section A. Pressure lines for every 10 kPa.
a) In normal conditions
b) After prolonged rainfall

5.3 MODELLING OF PORE PRESSURE CONDITIONS IN SECTION C IN TORP

The excavation in Section C was performed with two terraces. The ground surface at the upper terrace was first lowered by an extra amount and then supplied with a base of rock-fill covered by a pavement of more fine-grained material. The upper excavated slope is also covered by coarse friction material serving as protection against inner erosion. These parts are thus supplied with coarse surface layers that transport any water coming out of the upper slope. No such layer exists on the lower terrace, but a fictitious layer of sand has been applied in the model to let water streaming out from the upper parts of the slope spread out over the surface.

The results of the pore pressure measurements indicated that the embedded coarser layer at a level of about –30 metres is continuous and free-draining. The model therefore stops at the lower boundary of this layer and the measured pressure level is used as a boundary condition. Like the model for Section A, a vertical boundary has been located at the centre of the river-course with the boundary condition that no horizontal water flow occurs. The rear vertical boundary is located just behind Point 12. It is here assumed that no horizontal water flow occurs in the clay layers and that the water pressure in the uppermost sand layer corresponds to the measured free groundwater level. A clay layer that separates two sand layers in the upper sediments was found in the CPT tests. This layer was found in both Point 11 and Point 12 and it has been assumed to be continuous between these points with consideration given to the measured pore pressures. It cannot be estimated whether it also has reached further out towards the river and then been removed by the excavation. This model entails that the water pressure in the superficial sand layer in the area behind the excavation is affected by the upper groundwater level far behind the excavation, whereas the pore water pressure in the lower sand layer and the clay below is also affected by the groundwater level at the rear of the upper excavated terrace. This modelling can be made with the answer at hand, but would hardly have been possible on the basis of investigations with normal extent before the excavation. The model and the calculated pore pressure distribution are shown in Fig. 162.

As with the conditions in Section A, the groundwater levels here are located very close to the ground surface and rainfall can thereby only have a marginal influence. The measured pore pressure variations in this section are also very small in spite of the fact that closed systems were used. The calculated pore pressures in the bottom layers are governed by the assumed pressure level, which in turn corresponds to the pressures measured in these layers. The correspondence between measured

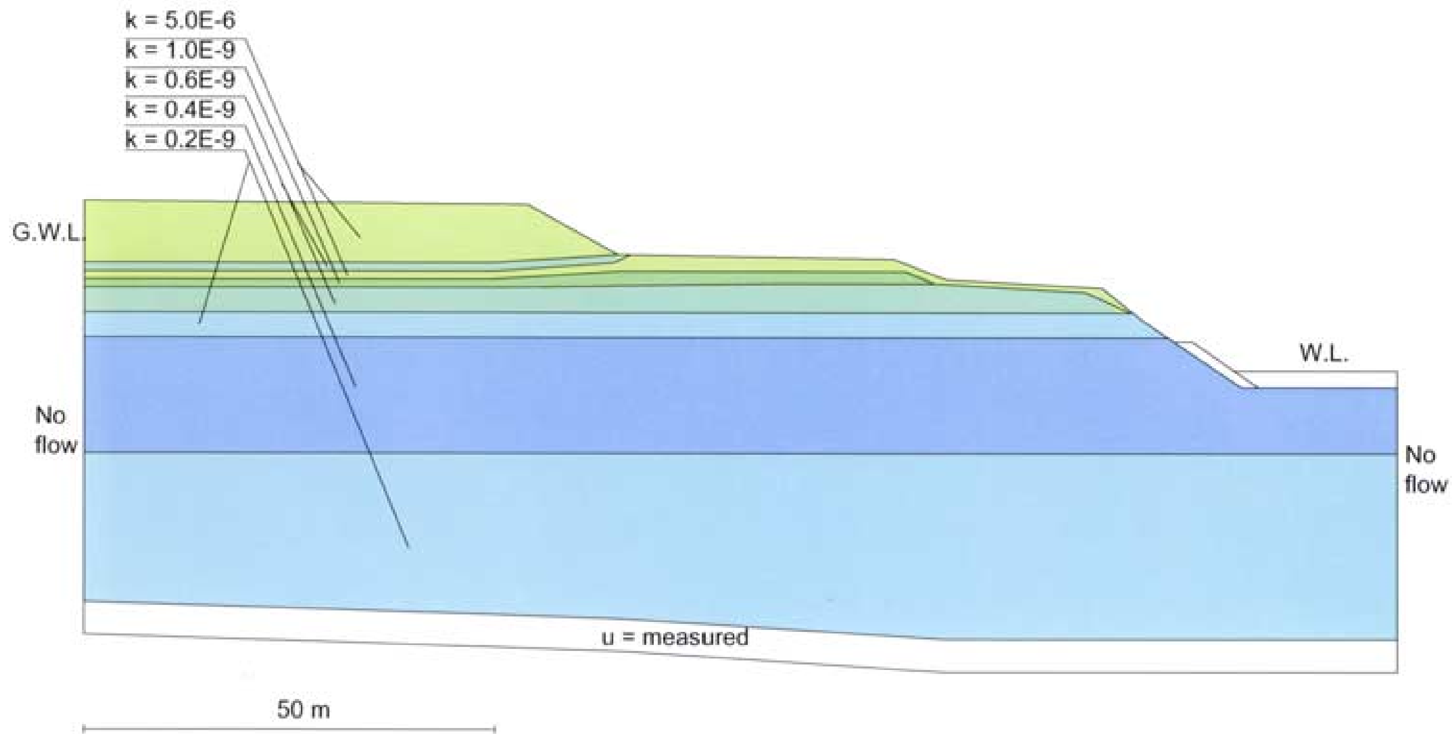


Fig 162. **Calculation model and calculated pore pressure distribution after excavation in Section C. Pressure lines for every 20 kPa.**

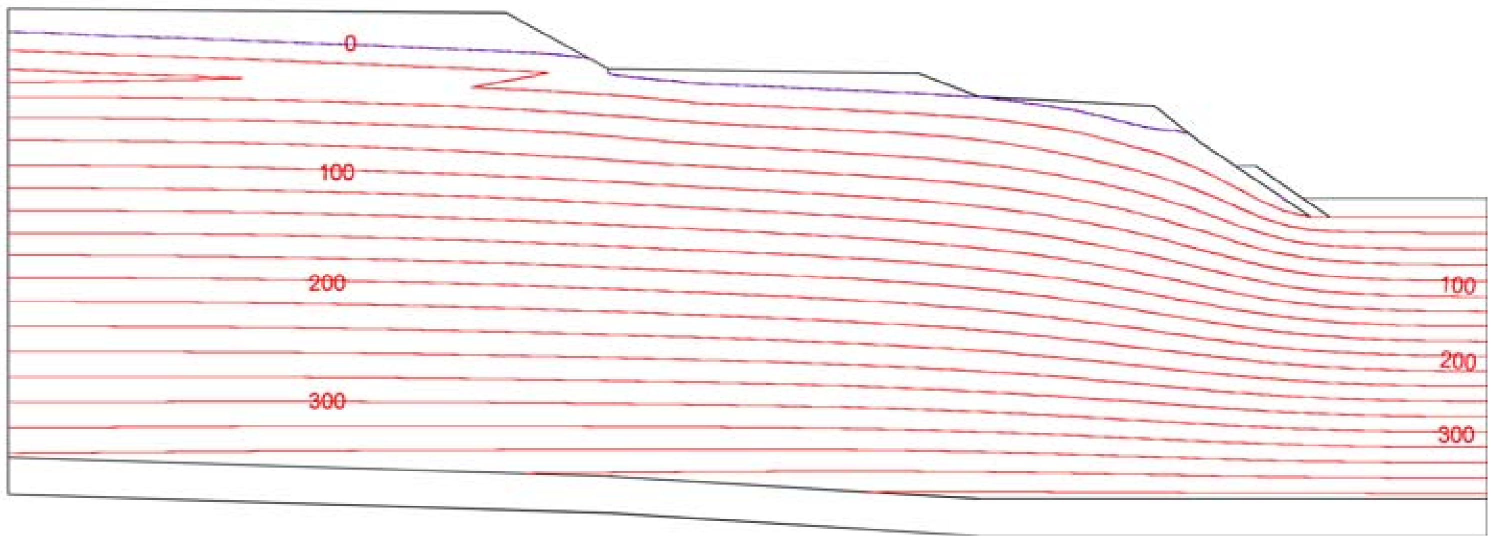
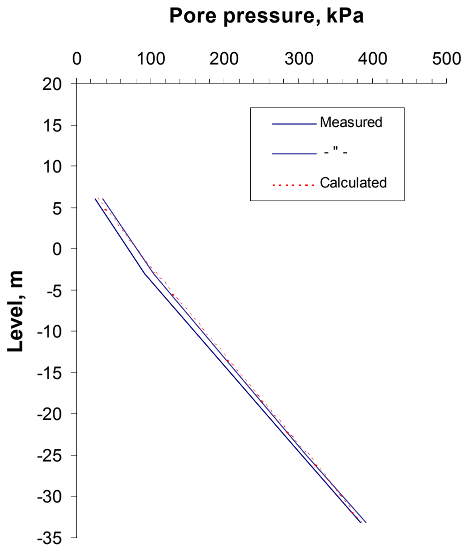


Fig 162. Calculation model and calculated pore pressure distribution after excavation in Section C. Pressure lines for every 20 kPa.

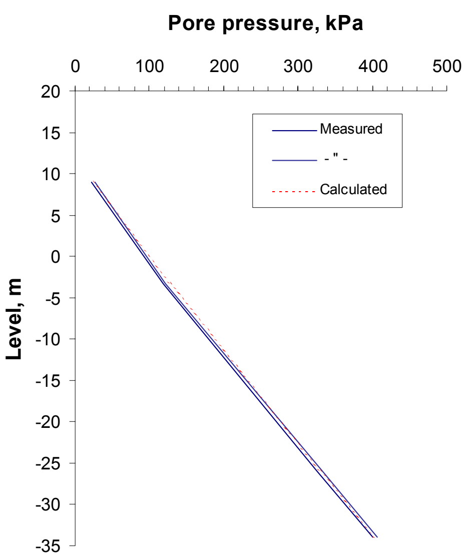
and calculated pore pressure at greater depths is thereby of necessity good. The general concordance is also good between measured values and those calculated with the assumptions described in the model, Fig. 163.

The results of a calculation of the conditions before the excavations are shown in Fig. 164. In this model, the excavated masses have mainly been assumed to consist of sand and silt with the same properties as the remaining layers behind the excavation. The embedded clay layer has not been assumed to continue into the excavated area. The boundary conditions have been assumed to be unchanged, but prolonged rainfall has also been modelled. The groundwater level in the upper sand layer has then not been allowed to rise at the rear boundary since the latter coincides with the road to the Åtorp manor house and the drainage system below this. On the other hand, the water level in the river has been raised and the pore pressures at the lower boundary have been increased. In this way, groundwater levels are calculated, which at both centres of the now excavated terraces lie about 1.5 metre higher in normal conditions and about 3 metres higher after heavy rainfall than after the excavation respectively. There are no reference values from pore pressure measurements before the excavation in this section.

Point 8



Point 9



Point 10

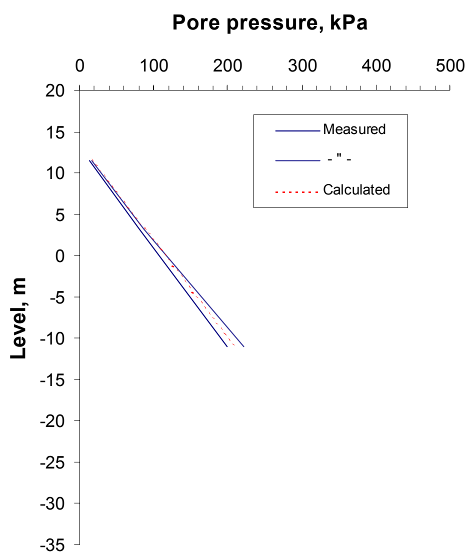
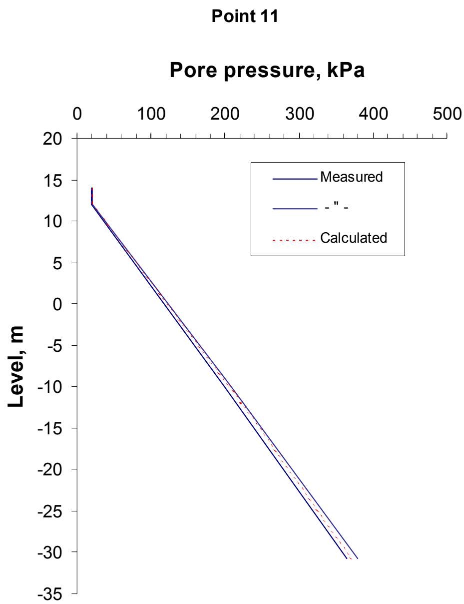


Fig. 163. Measured and calculated pore pressures in Section C.



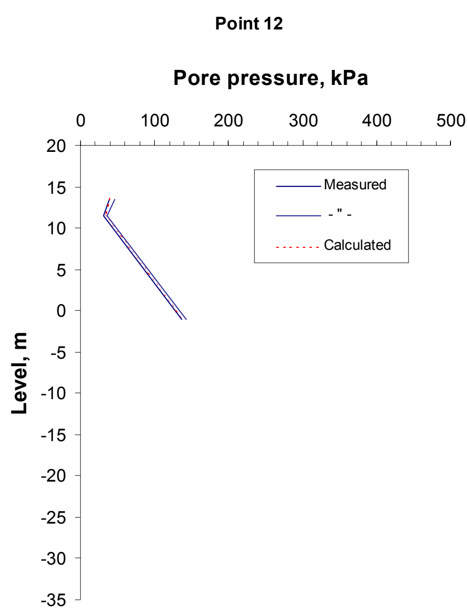
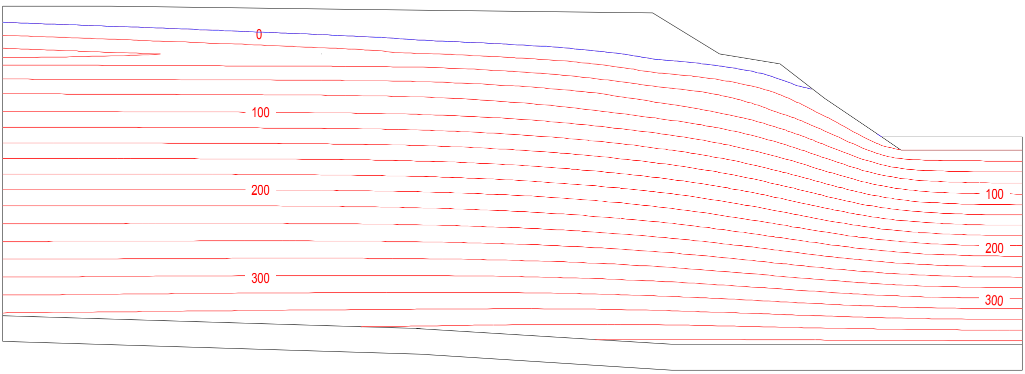
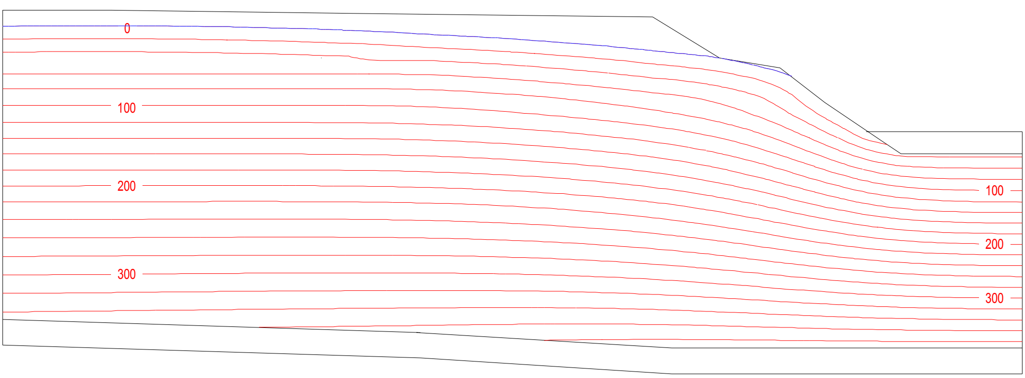


Fig. 163. cont. Measured and calculated pore pressures in Section C.



a)



b)

Fig. 164. **Calculated pore pressure distribution before the excavation in Section C. Pressure lines for every 20 kPa.**
a) In normal conditions
b) After prolonged rainfall

5.4 MODELLING OF PORE PRESSURE CONDITIONS AT STRANDBACKEN

The slope at Strandbacken has been modelled with the geometry, stratification and permeability properties that have been measured without any extra considerations, Fig. 165. In this case, the excavation has been performed solely in the upper layer with coarser sediments and about 1 metre of silt remains as a top layer on the excavated terrace. The modelling has been done with a lower boundary corresponding to the measured transition to the coarse bottom layers. The water level in the river has been varied between the levels 0 and +2 metres in accordance with the measured values and the pressure level in the bottom layers has correspondingly been varied between +1 and +3 metres. A vertical boundary has been placed at the centre of the deep channel in the river and a condition of no horizontal water flow has been applied at this. The other end of the model has been located just behind Point 6 about 30 metres behind the upper crest of the slope. The upper groundwater level here has been varied within the measured limits and a condition with no horizontal water flow through the clay at the boundary has been applied.

The modelling after the excavation does not take rainfall into account because water seeps out of the upper slope and the ground water level is very close to the ground surface over almost all of the terrace even in normal conditions. On the other hand, a simulation has been made of the possible effects of evaporation during drier periods. Empirical guiding values for the monthly evaporation in a relatively near area in western Sweden (Bergsten 1950) have been used for this purpose.

The calculated pore pressure distributions at high and low water levels and in dry periods are shown in Fig. 166a–c.

The calculated pore pressures have been compared to the measured values, Fig. 167. The correlation has to be good in the lower parts of the clay layers since the water pressures at the lower boundary in the model are based on measured pore water pressures in the bottom layers. The results show that an effect of the evaporation also has to be modelled to enable a calculation of the measured variations in pore pressure below the excavated terrace. However, the calculated variations are somewhat larger than those measured in the central parts of the clay profiles. This is probably a result of the calculations having been made for steady states without time limits for the boundary conditions, whereas both wet and dry seasons are limited in time. These periods have thus not been prolonged enough to let the calculated effects fully affect the central parts of the clay layers.

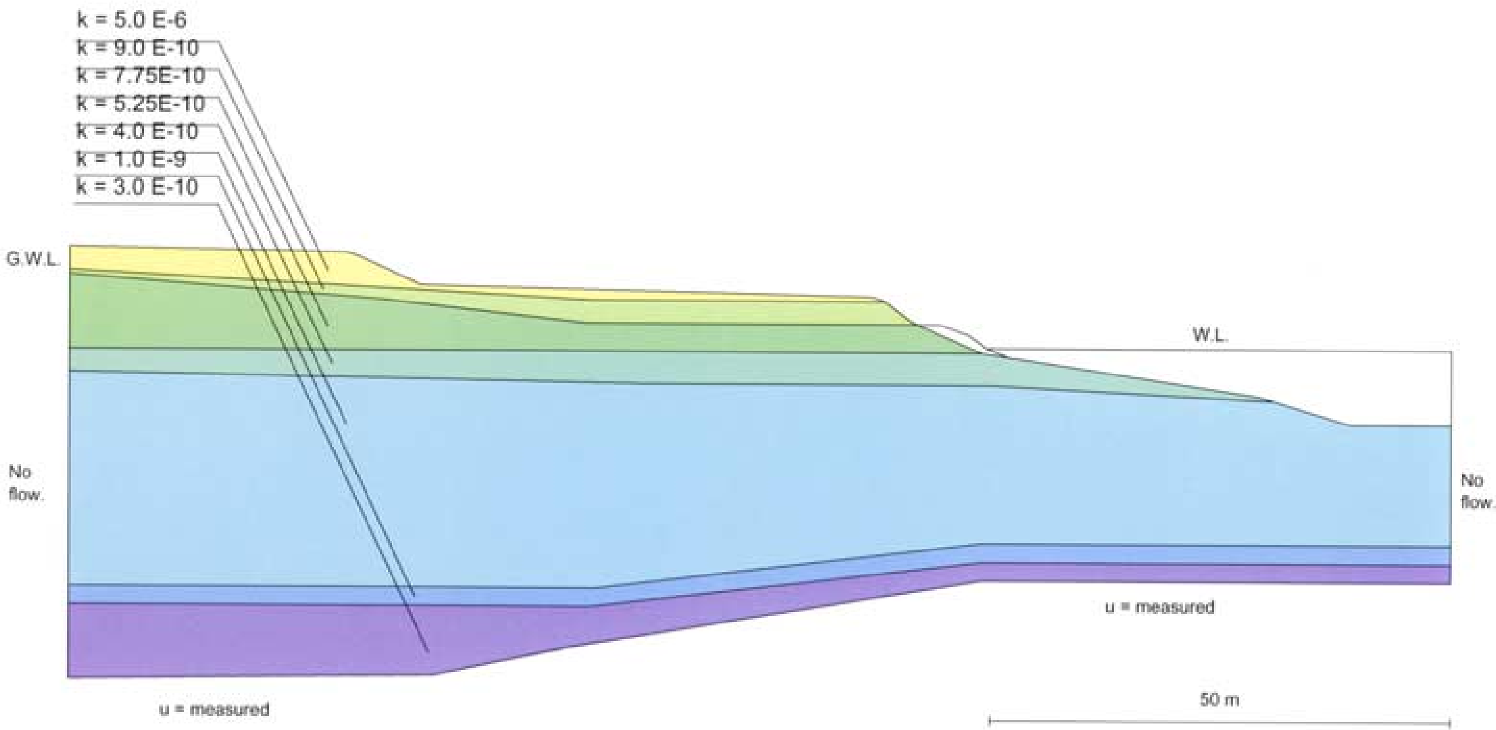


Fig. 165. Model for calculation of pore pressure distribution at Strandbacken in present conditions after the large excavation.

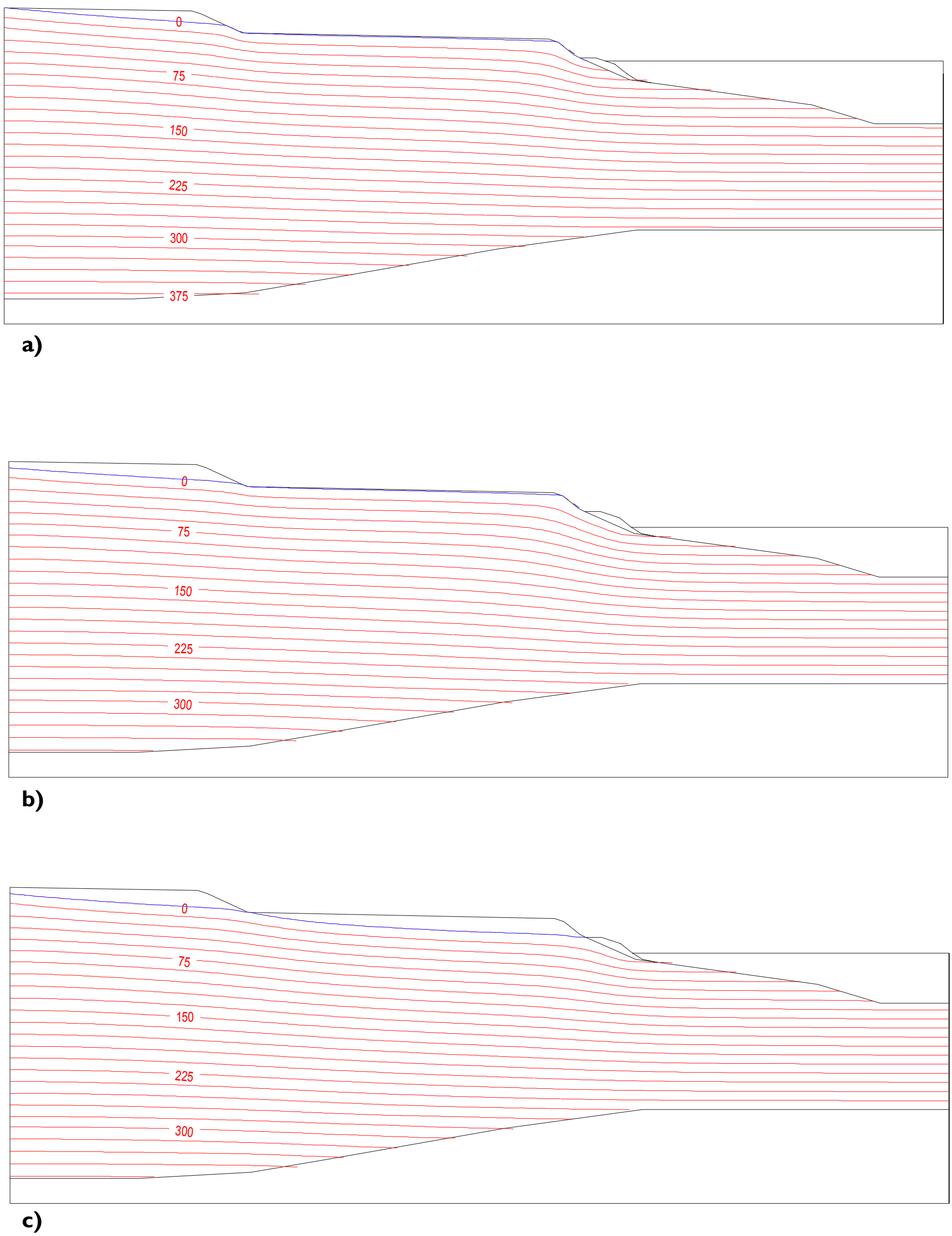


Fig. 166. Calculated pore pressure distribution after excavation. Pressure lines for every 15 kPa.
a) At high water pressures at the boundaries
b) At low water pressures at the boundaries
c) During a dry season.

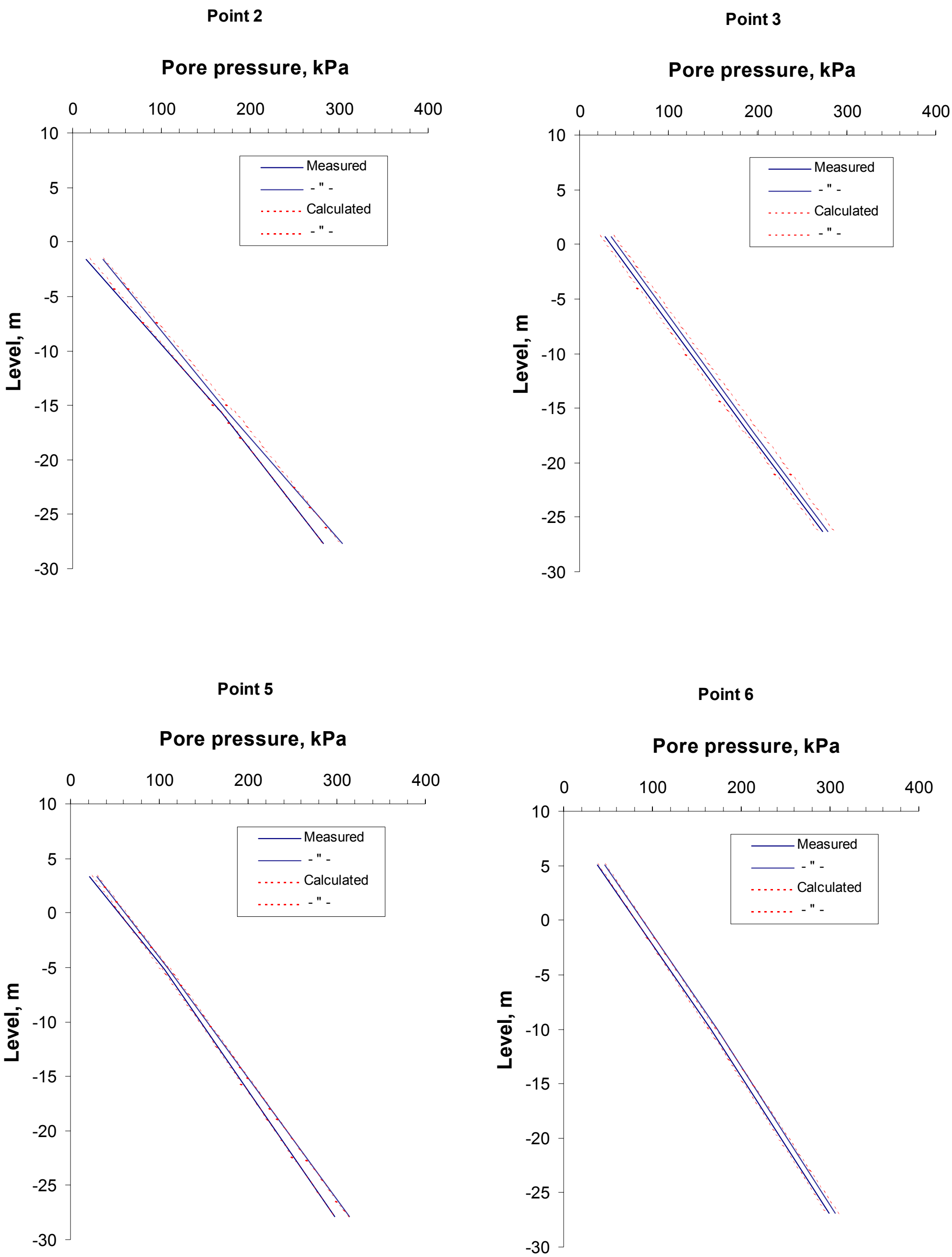


Fig. 167. Comparison between measured and calculated pore pressures after the excavation at Strandbacken. The span between the calculated pore pressures refers to variation between the cases of high water pressures at the boundaries and a dry season.

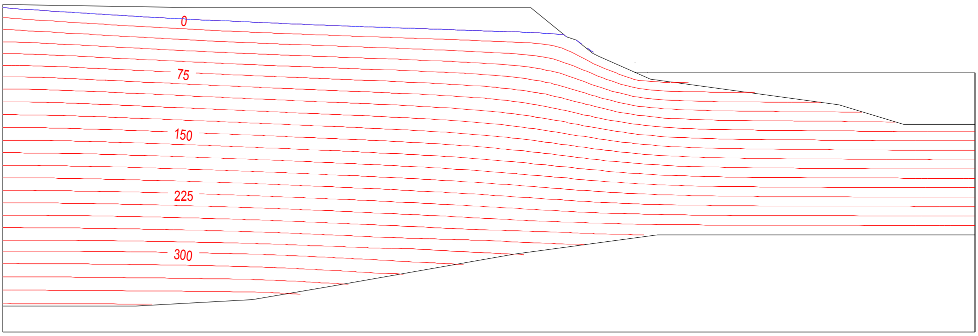
The previous conditions have been modelled for two cases, both before any excavation was made and after the first limited excavation. The calculations are schematic since the sections within the excavated area have varied somewhat and part of the area has been built-up with accompanying drainage systems. The calculations have been made for a "normal" condition with relatively low water pressures at the boundaries and a condition with prolonged rainfall. The results of these calculations are shown in Figs. 168a and b and 169a and b.

From the results it can be seen that the calculated groundwater level has been located at a much higher level than at present, particularly during wet periods, and that the first excavation should have brought a fairly small lowering of the groundwater level.

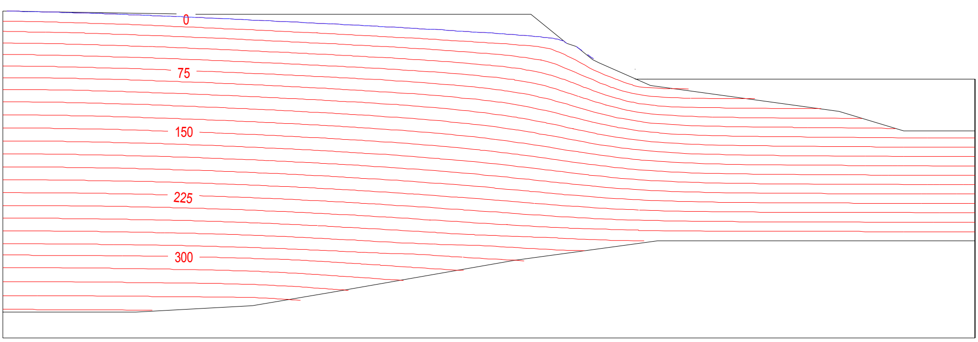
There are a few pore pressure observations made before the excavations, which can be compared, to the calculated values. In the Göta-älv investigation, pore pressure observations were made in open systems at a few points in the area between the lock at Ström and Strandbacken. These observations were made for a limited period of time and it cannot be estimated whether the read-off pressures were fully stabilised. All of these systems were not installed in or even close to the Strandbacken area and the sections and ground levels have varied. Six points with distances to the original crest of 20 to 40 metres have been selected for comparison. The ground level at one of these points differed by more than 2 metres in relation to the calculation section and the level for this value been adjusted accordingly when plotted for comparison.

In the investigation by SGI in 1988, two closed pore pressure measuring systems were installed in the clay and these were monitored for a relatively long period of time. They were placed about 10 metres behind the new upper crest after the first limited excavation that had been performed in 1959.

The comparison between the measured and calculated values is shown in Fig. 170. According to this comparison, the calculated values are of approximately the right size. However, the values from the Göta-älv investigation are mostly somewhat higher, possibly because the measured pore pressures have not been fully stabilised. The values measured by the investigation in 1988 show a larger variation than the calculated one, both upwards and downwards. This is plausible for the lower values since no dry season has been simulated. However, the highest measured values at this relatively close distance to the crest are difficult to fully explain and simulate.

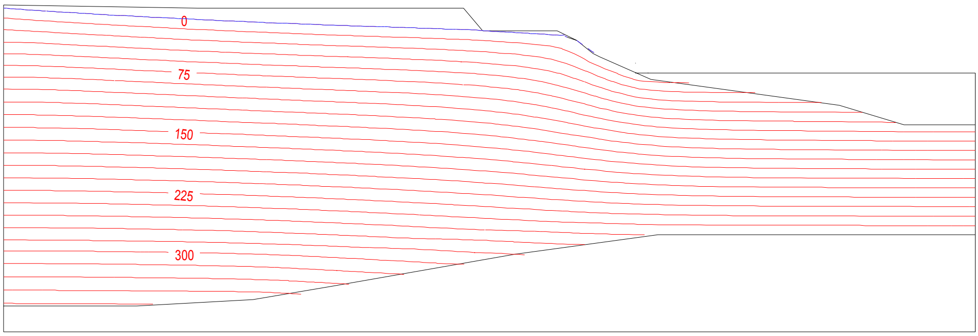


a)

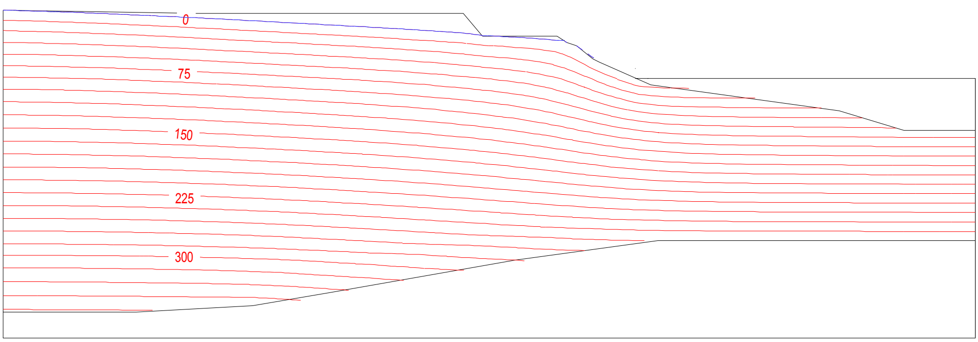


b)

Fig. 168. **Calculated pore pressure distribution before any excavation. Pressure lines for every 15 kPa.**
a) normal conditions
b) wet period



a)



b)

Fig. 169. **Calculated pore pressure distribution after the first excavation. Pressure lines for every 15 kPa.**
a) normal conditions
b) wet period

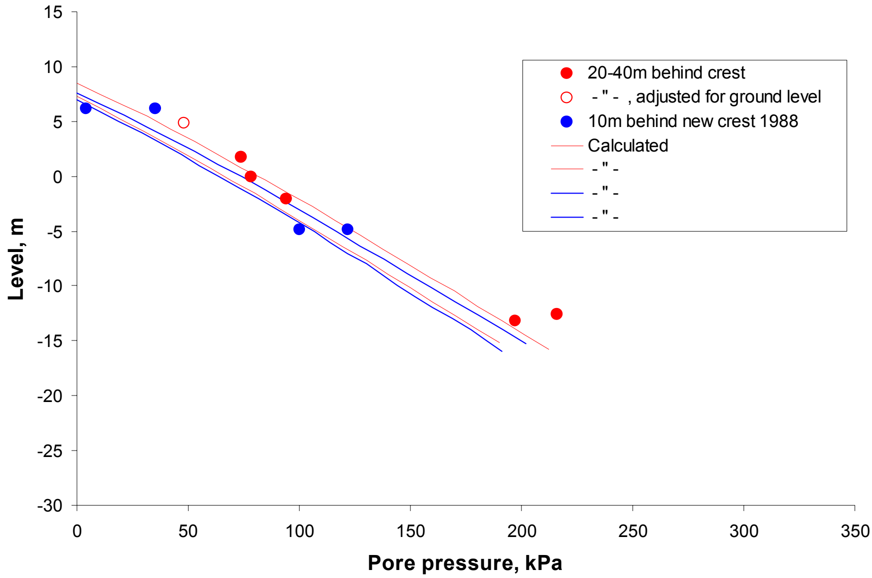


Fig. 170. **Comparison between measured and calculated pore pressures before any excavation and after the first limited excavation.**

5.5 MODELLING OF PORE PRESSURE CONDITIONS IN SUNDHOLMEN

The pore pressure conditions have been modelled in accordance with the stratigraphy and permeability values that were found in the investigations. The geometrical conditions have been selected as the same average section for the geometrical conditions on the nursing home lot that was also used in the stability calculations. On this lot, there is thus one part with a somewhat smaller width of the excavated terrace in front of the main building and one with a somewhat larger width close to the bridge.

The geometrical conditions are not quite the same as for the section where the pore pressure measurements were made, since the pore pressure measurement stations were placed to the north of this lot. The upper station is located on ground with the same level as the corresponding ground at the nursing home lot, but the lower station is located close to the riverbank within the area where a slide occurred in 1989. This means that the ground level here is even lower than on the excavated terrace in front of the nursing home.

The calculation model has been given an extent that reaches between the two roads running parallel to and on each side of the river. The given boundary conditions in these two vertical boundaries are that the groundwater level in the upper layer is located 1 metre below the ground surface and that the pore pressures in the underlying clay along these boundaries are hydrostatic from this level. The assumed groundwater level coincides with what has been observed in the investigations below the natural ground surface and is also natural with regard to the drainage systems along the roads. The lower boundary of the model has been placed at a level of –70 metres, i.e. 85 metres below the natural ground surface, in accordance with the information from the deep wells in the area. The pressure level at this boundary is also assumed to be at 1 metre below the natural ground surface which corresponds to hydrostatic water pressure from the assumed groundwater level at the boundaries. The excavation has been performed solely in the upper delta and lateral fluvial sediments and a layer of sand and silt still covers the excavated terrace. No effect of the installed drainage system has been taken into account. The calculations have been made for a mean water level in the river.

According to the results of the calculations, the upper ground water level barely touches the toe of the excavated slope both before and after the excavation, Fig. 171. This means that all of the excavated masses should have been located above the free groundwater level and that the latter has not been altered by the excavation works.

No influence of rainfall has been modelled. The area between the excavated terrace and the road consists of a built-up ground with accompanying paved surfaces and drainage systems. The infiltration of rain and melt-water should thus be very limited in this area. After the excavation, the groundwater level at the terrace is very close to the ground surface and the possible effect of infiltration is thereby very limited.

Heavy rainfall could have significantly affected the conditions in front of the built-up part of the lot before the excavation and thereby affected the stability of the steep riverbank. However, it is not known whether there was any effective drainage system at that time or not. High water levels in the river also affect the groundwater level and the pore pressures but do not constitute a stability problem except during very rapid drawdowns afterwards. The pore water pressures in the upper soil layers decrease during dry periods because of evaporation. This effect can be assumed to reach deeper after the excavation. The installed drainage system should also have brought a certain lowering of the groundwater level during those periods when it is functioning. According to the calculations, the excavation has not brought any

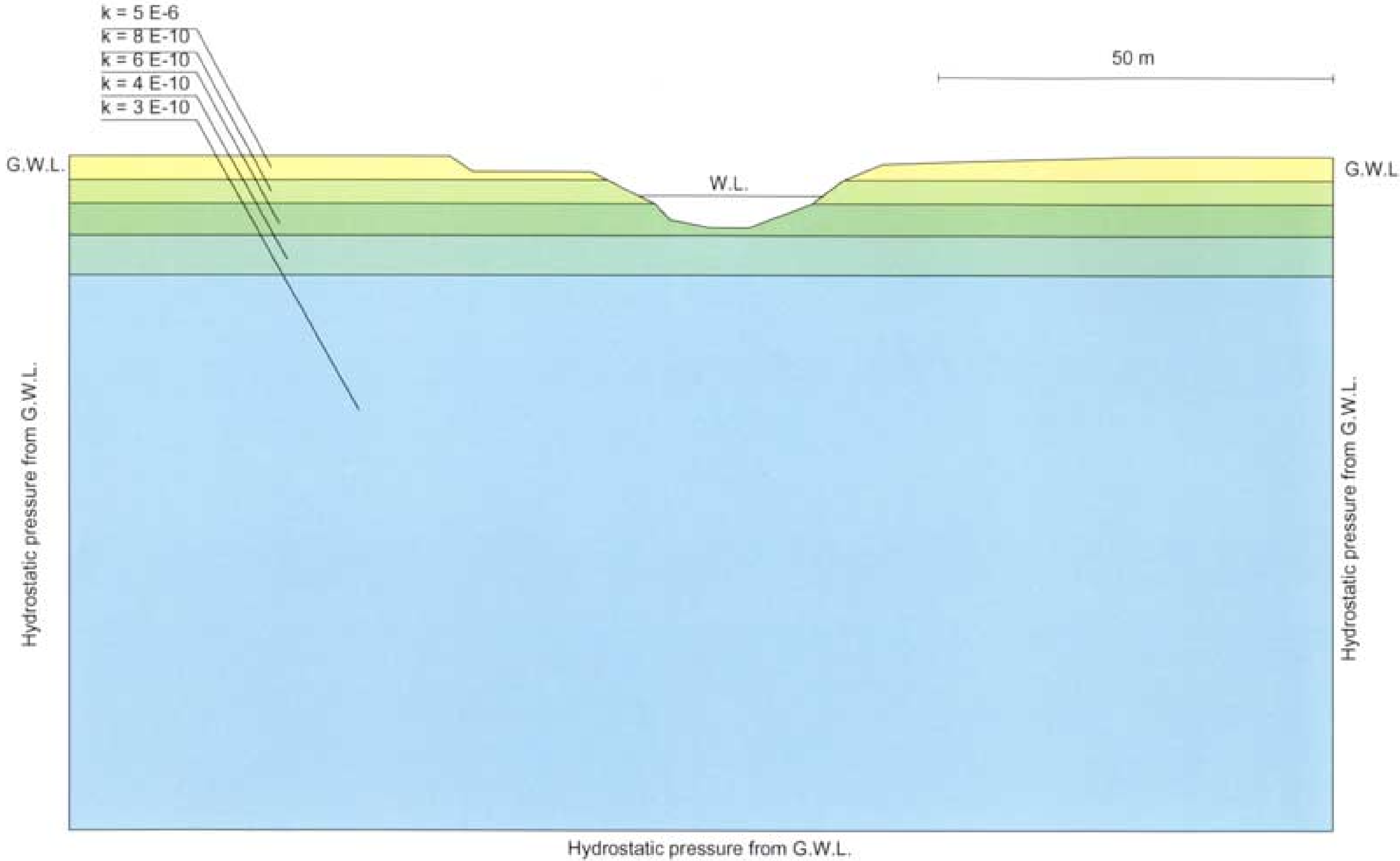


Fig. 171. **Calculation model and calculated pore pressure distribution within the upper 30 metres of the soil layers in Sundholmen.**
a) Calculation model

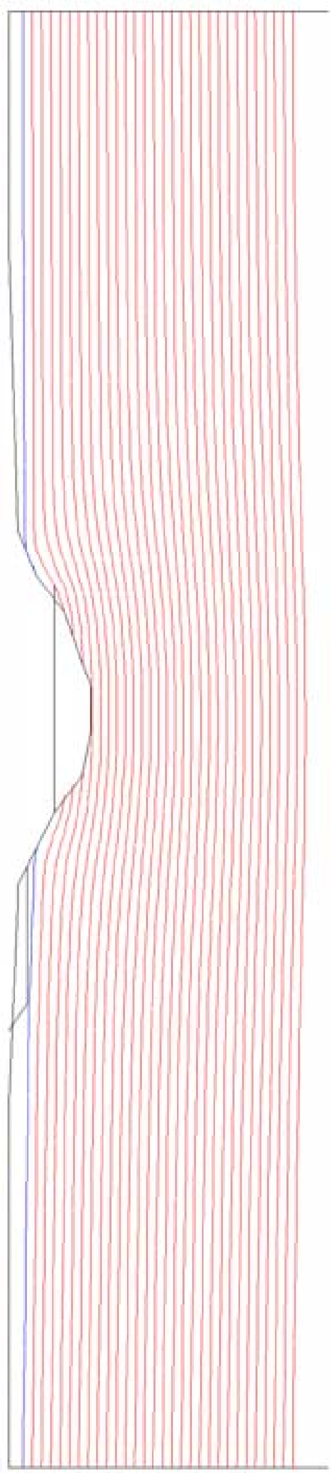


Fig. 171. Calculation model and calculated pore pressure distribution within the upper 30 metres of the soil layers in Sundholmen.
b) Calculated pore pressure distribution. Pressure lines for every 10 kPa.

significant changes to the pore pressure conditions apart from the aspects mentioned above, which are more difficult to simulate.

The calculated depression in pore pressures around and below the river fits very well with the picture obtained from the measurements. Considering the relatively simple soil and boundary conditions that prevail in this section, it is plausible that a good prognosis could have been made based on conditions known before the excavation. However, this does not apply to the effects of the installed drainage system.

Chapter 6.

Experience from and comments on the results of the investigations

6.1 INVESTIGATION METHODS

6.1.1 Determination of stratification and depth to firm bottom

The soil layers and stratification can be highly variable also in areas that in a general overview are characterised as clay areas. The thickness of the deposits of soft soils can also vary greatly. The clay layers are in many areas covered by coarser soils, and layers of coarser soil are also often found embedded in the clay layers. The shear strength in these layers differs from that in the surrounding clay and can be both higher and lower. If these layers are thick enough and continuous, they also become water transporting and affect the groundwater pressures in the entire soil mass. It is thus very important to use investigation methods that enable in a rational way the detection and mapping of such variations in the soil profiles. Among the various equipment used in current practice, only CPT tests and dilatometer tests fulfil this requirement.

The depths to "firm bottom" and to bedrock can vary strongly both sideways and along the investigated area. The depth to firm bottom has to be established since it constitutes the lower limit for possible slip surfaces. The firm bottom is also in many cases made up of friction soil or fissured rock that is permeable and thereby affects the pore pressure conditions in the overlying soil mass. The depth to firm bottom and its variation also affect the homogeneity of the overlying soil masses across and along the area. A strongly inclined firm bottom often means that the overlying soil layers are inclined and vary in thickness. The varying conditions for percolation of water and diffusion also entail that different parts of the soil mass may have been subjected to different degrees of chemical changes, such as leaching of salts and/or introduction of dispersive agents, and that the character and properties of the soil thereby vary.

The depth to firm bottom should preferably be known roughly before the detailed geotechnical investigations start. This facilitates a selection of proper equipment and a rational methodology for the investigations. General information about what type of soil layers can be found and what depths to firm bottom can be expected is often obtained by the initial "inspection" which should be performed according to the guidelines for slope stability investigations given by the Swedish Commission on Slope Stability (1995). This inspection comprises, among other things, studies of previous geotechnical investigations in the area, the geological map and the accompanying geological descriptions of the area. Great caution has to be observed when interpreting old investigations, because these were often performed with simpler equipment with less resolution in the results and ability to penetrate the soil layers. If the available information is insufficient, an initial geophysical investigation should be considered for clarification of the thickness of the soil layers in the area. Recommendations for when geophysical investigations should be performed and what methods should be used for different soil conditions and at different stages in a slope stability investigation have been given in SGI Report No. 62 (Dahlin et al. 2001).

The depths to firm bottom vary between the different investigated areas, even though they are fairly large in all cases, from about 20 to about 85 metres. In the Strandbacken area, there was a large amount of information available from previous investigations and also a detailed geological description of the area. Different investigation methods had been used and some of these could with confidence be assumed to have penetrated down to firm bottom. The maximum depths to firm bottom, about 40 metres, the main stratification and undrained shear strength of the soil were thereby largely known. The investigations could thereby be planned with already known requirements for penetration force, amount of equipment and its sensitivity and resolution. The purpose for the new investigations was thereby mainly limited to supplementing the known picture of stratification and soil properties in the area with more accurate results with higher degrees of detail and resolution.

In Sundholmen, the approximate depth to firm bottom was known from information from several deep wells in the area. It could thereby be estimated at about 85 metres, and there was no significant variation in the reported depths. The previous geotechnical investigations showed that the upper soil layers were also fairly homogeneous regarding thickness and properties. Because of the limited depth and width of the river-course, only the properties in the upper soil layers affect the stability unless any significantly weaker layer exists at greater depth. As in the previous investigations, the new ones were therefore concentrated in the upper 25 metres. However, one CPT test was continued to more than twice this depth in order to ascertain that the above-mentioned condition was fulfilled.

In the Torp area, there were also previous investigations available. However, these had mainly been performed with equipment with limited ability to penetrate and the achieved penetration depths had erroneously been designated as stops at firm bottom. This meant that the thickness of the soft soil deposits was underestimated before the new investigations and the preparations and equipment first brought to the site were insufficient. Supplementary investigations therefore had to be made in several stages and the investigations in the deepest profile were finally stopped at depths of 50 to 60 metres without having reached firm bottom. This could be done because it was estimated from control calculations that the then remaining underlying soil could not affect the stability. In this case, the investigation could have been made in a much more rational way if the later geophysical investigation had been performed prior to the geotechnical investigations.

CPT tests and dilatometer tests provide similar and often more or less equally good pictures of the stratification and classification of the soil layers. However, the CPT test is more cost-effective and yields a fully continuous profile. The dilatometer test is normally used only as a supplement and when some special property is to be measured. Both methods yield a coarse classification whereby significantly different soil types such as sand, silt and clay normally can be distinguished with fairly good accuracy. The CPT test can also be used for estimation of whether different layers can be considered as free-draining or not by studying the dissipation of the generated excess pore pressure during short stops in penetration in these layers. This should always be done at stops against presumed firm bottom layers to check whether these consist of free-draining coarse soil or fissured rock. A measure of the stabilised water pressure at these levels is then also obtained. The possibility to classify and differentiate between more similar soil types, such as organic clay, clay, silty clay and clayey silt is limited. Boundaries or transition zones between different types of soil are often clearly observed in the results of CPT tests in terms of changes in friction ratio, pore pressure ratio and development of tip resistance and generated excess pore pressure with depth. In the dilatometer tests this can be seen in the trends for measured pressures and evaluated material index. However, samples of the soil have to be taken and classified in the laboratory to enable a determination of the causes of these changes. The results of such classifications and the belonging routine tests are also required for making an optimal evaluation of properties in the soil from the field tests. The results of the CPT tests in this project illustrated these possibilities and limitations. All significant layers, both thin and thick, were registered, but an exact classification required that samples should be taken in all of these layers.

The investigations should therefore start with CPT tests and the sampling levels should be selected from these results in such way that all significant soil layers are included in the sampling. On the other hand, the amount of sampling in thick homogeneous layers can in the same way be reduced in many cases.

Static penetration tests, such as the CPT test, stop in very dense coarse soil. They therefore cannot be used to measure the thickness of underlying layers of dense friction soils. Also dynamic penetration tests usually stop somewhere within such layers, unless they are thin. Geophysical investigations provide a contour of the bedrock surface. The accuracy of this determination is often sufficient, but the measurement is not exact. In order to determine the bedrock surface more exactly, and if required obtain a better calibration of the geophysical measurements, soil-rock drilling with multi-channel registration of the drilling parameters should be performed, preferably by the Jb-3 method (SGF 1999). For an estimation of the drainage properties at the firm bottom, it is normally sufficient to study the equalised pore pressures at the stops of a large enough number of CPT tests in the area. This picture will also be clarified further when the results of the pore pressure measurements in piezometers become available.

6.1.2 Determination of shear strength

General

The *field vane test* is the traditional method of determining undrained shear strength in Sweden. *Fall-cone tests* are also performed as part of the routine testing of undisturbed soil samples in the laboratory. The results of the fall-cone tests often become of the same size as those of the field vane tests down to depths of 10 to 15 metres. At larger depths, they generally become lower, which is assumed to be a result of the large unloading that occurs in samples from great depths. The latter results are thus considered to be less relevant. This applies to homogeneous clay. In clay with numerous thin silt layers or with infusions of other coarser particles, the fall-cone test can give higher values because these objects stop the cone penetration. The fall-cone test should therefore not be used in such soils. The general picture with similar values from field vane tests and fall-cone tests down to 10 to 15 metres' depth and lower values from the fall-cone test below that was mainly obtained also in the present investigation. The exceptions were the investigations performed below the river beds, where the results of the fall-cone tests became lower throughout the profiles.

The results of *field vane tests* often indicate an improbably low increase in shear strength with depth. Among other things, this can be a result of the amounts of silt particles and thin silt layers increasing with depth, which entails an increased risk that the soil will be excessively disturbed when the vane is inserted at the test level. It is recommended that test equipment with protective rods and a vane casing be used in such soils, since this type of equipment has been found to cause less disturbance here. In this project, and in the special study of the field vane test that was performed in connection with it, it has also been found that a change from the normal vane size to a smaller one may result in a smaller shear strength value being measured (Åhnberg et al. 2001). Such a change is made when the maximum torque for the vane and/or the measuring instrument is reached. In soils that are easily disturbed, there is thus an obvious risk that there will be a jump in the measured shear strength at the level where the change is made and that the increase in shear strength then becomes lower at greater depths. This is a further reason to use equipment with casing since most of the rod friction, which otherwise constitutes a large part of the torque resistance, is thereby eliminated. The normal vane size can then often be used to considerably greater depths. These aspects were illustrated by the results from the Torp area in this investigation. Larger vanes than the normal are sometimes used in very soft soils. This can also affect the measured shear strength in some types of soils, as was shown in the special study of the field vane test that was initiated by the results from Torp (Åhnberg et al. 2001).

A further aspect that may influence the measured increase in shear strength with depth in field vane tests is that the overconsolidation ratio in the soil often varies with depth. How the overconsolidation ratio affects the results will be discussed further on in this Chapter.

The field vane test is a relatively time-consuming and costly method for determination of the undrained shear strength in situ. The *CPT test* is a considerably more rational method and it is therefore recommended that this test be used to a relatively large extent also for determination of the undrained shear strength, provided that the equipment is accurate enough. The results from initial CPT tests can then be used for rough calculations to estimate whether there is a possible stability problem and in that case how far the potential slip surfaces extend. The following field vane tests can then be concentrated in this zone where, apart from being a supplement to the CPT tests, they also constitute a local calibration of the latter against a better tried out method. However, the evaluation of both field vane tests and CPT test rely on empirical factors and both may need a calibration against, for example, direct simple shear tests, as has been shown in this project.

The shear strength can also be evaluated from *dilatometer tests*. These tests are normally only performed in order to determine some other parameter(s), as for example the coefficient of earth pressure. However, in some cases they may be used as supplements to estimate the shear strength in soils that are very easily disturbed and in which both field vane tests and CPT tests may be assumed to cause excessive disturbance and in which taking of undisturbed samples is extra difficult. In such soils, the dilatometer test has often proved to be the method whose results in terms of evaluated undrained shear strength have been least affected by the disturbance during penetration. In the investigations in this project, the shear strength evaluated from dilatometer tests often differed more than is normal from the results of the other methods in homogeneous clay layers. This applies primarily to the evaluation method proposed by Larsson (1989). Rankka (1994) has previously obtained similar results at investigations in slopes. The difference may be related, among other things, to the special stress conditions in a slope. In that case, such factors as the orientation of the dilatometer blade in relation to the direction of the slope could also influence the results. The internationally more commonly used method, which employs the evaluated overconsolidation ratio, the effective overburden pressure and empirical relations between these and the undrained shear strength, in most cases gave results in better agreement with the other test methods. However, this is provided that the evaluation method for the overconsolidation ratio that has been presented in this report is used.

Shear strength values obtained by field vane tests and fall-cone tests are corrected by empirical *correction factors*. In Sweden, these factors are related to the liquid limit of the soil (Larsson et al. 1983). The normal evaluation of undrained shear strength from CPT tests is also based on an empirical cone factor, which should be modified with respect to the liquid limit of the soil (Larsson and Mulabdic1991). An evaluation of the shear strength from field vane tests and fall-cone tests thereby requires that the liquid limit on the test level at the test point be determined. This has to be done on samples brought into the laboratory. A more qualified evaluation of the undrained shear strength from CPT test results also requires that the variation in liquid limit with depth in the test point be determined. Dilatometer tests are also evaluated by use of empirical correlations, However, the normally used evaluation methods do not require any other parameter than those determined in the test, apart from an estimate of the pore pressure distribution.

The basis for all empirical relations contains scatter, and a condition for the relevance of the evaluated shear strengths is that the tests have been performed according to established methods, i.e. according to the recommendations of the Swedish Geotechnical Society. Another condition is that the disturbance of the soil at the test is within normal limits for the particular test method in a soil with a corresponding composition and that all other factors have also been "normal". However, the only measure of the composition of the soil that is normally used is its liquid limit. Smaller or larger deviations are therefore often obtained in the results from some and sometimes for all of the methods mentioned above. In the soil profiles in the present investigation, the largest differences were obtained in the organic, silty and sandy delta and lateral fluvial sediments, whose compositions also differ from those soils on which the empirical factors are based.

A first way of checking whether the measured shear strength values are reasonable is to use *empirical relations* for how the shear strength normally varies with preconsolidation pressure, overconsolidation ratio and liquid limit (Larsson et al. 1983). This requires that undisturbed samples have been taken and that the variation in preconsolidation pressure with depth has been determined by oedometer tests. The empirical relations can be used to sort out obviously impossible values that are due to excessive disturbance during the test, improper execution of the test, measuring errors, damaged equipment, etc. However, also the empirical relations contain considerable scatter and they can only be used in this way and as support for measured values and trends.

A more direct calibration of the measured shear strength values can be made by performing undrained *direct simple shear tests* on samples of high quality, which have first been reconsolidated with guidance from measured preconsolidation pressures and then been allowed to adapt to the in situ effective overburden pressure. The results of such tests are not corrected but are used as direct shear strength determinations. They can thereby also be used for a local calibration of all the empirical factors and correlations mentioned above.

The *shear strength anisotropy* can be estimated roughly with the use of empirical relations (Larsson et al. 1983). However, the shear strength estimated in this way is only used in coarser estimates. Before being used in a final estimate of the stability, it has to be verified by active triaxial tests and when appropriate also by passive triaxial tests. These tests can be performed in a limited series that primarily aims at verifying that the existing empirical relations are also applicable in the specific case. If any significant deviations from these are measured in the tests, the test series has to be extended.

The *effective shear strength parameters* in clay are also normally estimated according to an empirical pattern (Larsson et al. 1983). The applicability of this pattern should be verified by a number of drained triaxial tests in all cases where the absolute values have a significant importance for the estimated stability. A control can also be made by studying the effective stress-paths in undrained triaxial tests. The test series should be expanded if any significant deviations from the empirical pattern are measured. Effective shear strength parameters in possible overlying or embedded silt and sand layers are normally estimated from empirical relations and results of CPT tests (e.g. Knutsson et al. 1998).

Examples of how the different determinations scatter, how empirical relations can support different trends and how the evaluation can be improved by local calibrations are found in the presentations of the test results from the different test areas. This also applies to the performed verifications of shear strength anisotropy and effective shear strength parameters.

The descriptions given above for how the shear strength can be determined are in principle in line with the recommendations for determination of shear strength in stability investigations given by the Swedish Committee on Slope Stability (1995). However, some additions and modifications have been made with regard to experience and results obtained in the present and other mentioned research projects carried out since then. Further aspects of the determination of shear strength in overconsolidated soil will be discussed in the next section.

Special effects in overconsolidated soils and slopes

The stress conditions in overconsolidated soils and slopes are different from those in normally consolidated soils under flat ground. The horizontal stresses have not decreased in the same degree as the vertical overburden pressure in overconsolidated soils in which the overconsolidation is due to a real unloading. When the unloading becomes large enough, a reversed situation with larger horizontal stresses than the overburden pressure will occur.

In natural slopes, the stress conditions will differ in different parts of the slope (e.g. Rankka 1994). The stress conditions far behind the crest of the slope are in principle the same as below ordinary flat ground and the soil is here often "normally consolidated or only slightly overconsolidated". The horizontal stresses in the soil here correspond to the earth pressure at rest. The soil closer to the crest is still often only slightly overconsolidated but often for slightly higher effective vertical stresses since the groundwater level is somewhat lower due to the seepage of water towards the lower lying parts of the slope. At the same time, the horizontal stresses are lower here towards the slope since the corresponding counteracting soil masses are less in this direction. The horizontal pressures here thereby correspond to values between earth pressure at rest and active earth pressure. The stress conditions below the slope then gradually change to the situation down at the toe of the slope, where the vertical stresses are low and the overconsolidation is correspondingly high. The coefficient of earth pressure is high because of both the overconsolidation ratio and the excess pressures transmitted from the overlying sloping ground, and the horizontal stresses lie somewhere between earth pressure at rest and passive earth pressure. The differences in stress conditions between normally consolidated soils and overconsolidated soils are thus enhanced in a slope.

Most of the test methods used in situ are sensitive to the acting horizontal stress. This has been shown for the case of field vane tests below embankments on clay (Law 1979 and 1985), and for CPT tests in sand (e.g. Schmertmann 1972). The dilatometer measures pressures in a horizontal direction and should thereby be sensitive to the in situ horizontal stress in that direction. The common evaluation methods are generally tried out for normal cases with normally consolidated to moderately overconsolidated soils and relatively flat ground and are therefore not necessarily valid for significantly different stress conditions. The influence of the horizontal stresses on the results of field vane tests and CPT tests in different parts of a clay slope is currently being investigated in a separate research project in co-operation between the Swedish Rescue Services Agency, Chalmers University of Technology and SGI (Löfroth 2002).

In most general models of how the undrained shear strength varies with preconsolidation pressure and overconsolidation ratio the shear strength is directly or indirectly described by the equation

$$c_u = a\,\sigma'_v\, OCR^b \quad alt.\ c_u = a\sigma'_c\, OCR^{b-1}$$

(e.g. Ladd and Foott 1974, Jamiolkowski et al. 1985, Schofield and Wroth 1968, Roscoe and Burland 1968, Wood 1991)

These relations are obtained in both direct simple shear tests and triaxial tests in the laboratory. Results presented by e.g. Larsson (1980), Jamiolkowski et al. (1985) and Mayne (1988) show that the factor *a* is about 0.33 for active triaxial tests on clay and up to 0.5 in the same type of test on organic soils, (Larsson 1986). For the cases of direct simple shear tests and passive triaxial tests, the factor *a* has been found to vary with the liquid limit (or plasticity index). An average value of 0.22 is often used for the case of direct simple shear, whereas considerably higher values should be used in organic soils. There may also be a possible slight variation in the *a*-factor in triaxial tests on clay with the plasticity of the soil (e.g. Westerberg 1999), but this is normally not taken into consideration.

The *b*-factor is normally between 0.75 and 0.85 in both triaxial tests and direct simple shear tests (e.g. Jamiolkowski et al. 1985 and Mayne 1988). It is normally set to 0.8 according to the empirical pattern (or more cautiously to 0.75). The values that have been measured in this investigation have been within the normal band. This means that a large unloading results in a considerable decrease in the undrained shear strength.

However, a corresponding reduction is not measured by field vane tests in situ. In such tests the measured reduction is far less, if any (e.g. Jamiolkowski et al. 1985). According to Swedish experience (Hansbo 1957), the preconsolidation pressure in clay can be estimated directly from the measured shear strength values in field vane tests by the equation

$$\sigma'_c = \frac{\tau_v}{0.45\, w_L}$$

where τ_v = measured uncorrected shear strength value from the field vane test
w_L = liquid limit

A corresponding relation expressed as a function of the plasticity index has been found for other clays by Mayne and Mitchell (1988). These correlations imply that no reduction in the measured shear strength value from field vane tests as a result of unloading can be expected.

The fact that a certain effect of the unloading and overconsolidation can be observed in the results of the field vane tests means that a certain modification of the Hansbo relation ought to be made in overconsolidated soil. However this modification is small. The results of this investigation have given the relation

$$OCR = \left[\frac{\tau_v}{0{,}45\, w_L\, \sigma'_v}\right]^{1{,}11}$$

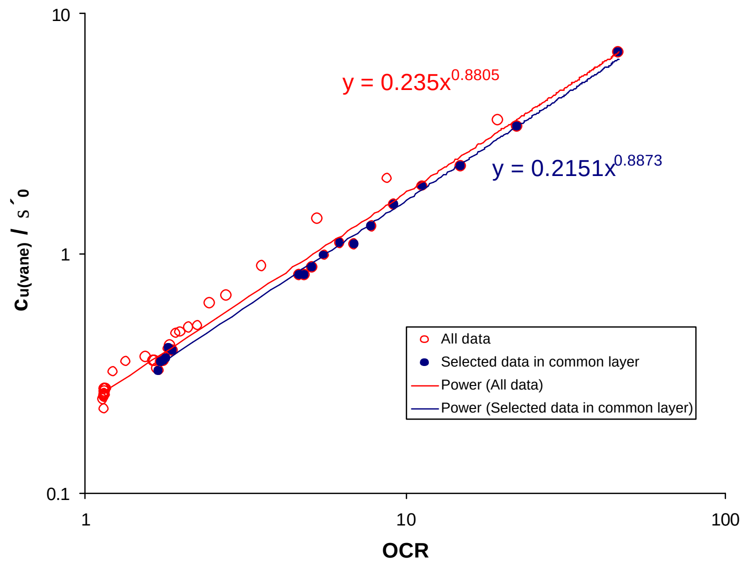
where OCR = overconsolidation ratio evaluated from preconsolidation pressures determined by oedometer tests
σ'_v = effective overburden pressure

An old rule of thumb for field vane tests below eroded water courses is that a reduction in the shear strength in relation to the measured values at the same levels in the surrounding ground can be detected within the uppermost 5 to 10 metres below the river bottom, but not further down in the profiles (Bergdahl 2002). Corresponding results were largely obtained in the present investigations and this also applied to the CPT tests according to the common methods of interpretation. It is hoped that the aforementioned ongoing project (Löfroth 2002) will show if, and to what extent, this is related to the stress conditions with relatively high remaining horizontal stresses in the unloaded areas at the toes of the slopes.

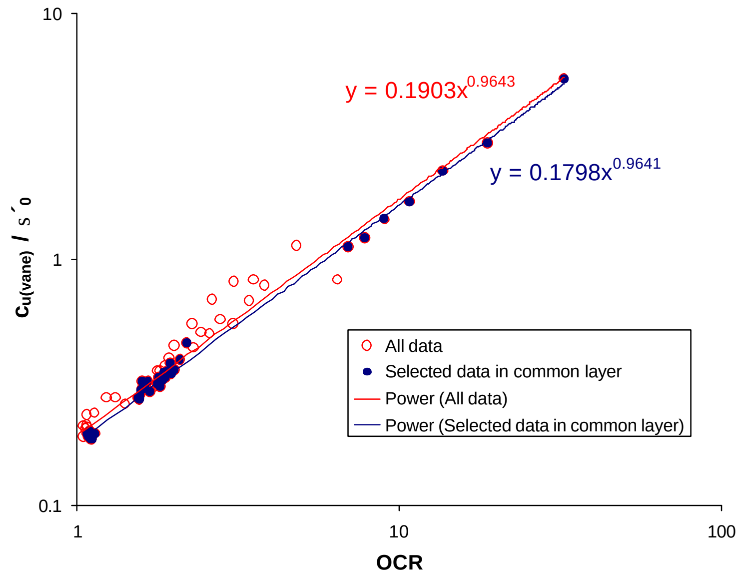
According to the accumulated values presented by Jamiolkowski et al. (1985), there is generally a small effect of an unloading on the shear strength values in uncemented clays which can be expressed with a *b*-value of about 0.95. This entails a certain but small reduction in measured undrained shear strength because of unloading. The results from the field vane tests in the present investigation have also been gathered for evaluation of the *b*-factor. However, it is not quite straightforward to obtain a correct value of this factor in a natural soil mass since not only the overconsolidation ratio but also the liquid limit (or plasticity index) vary. The relations between the effective overburden pressures and the measured shear strength values are thereby affected in more than one way and a certain scatter can be expected also after correction with respect to the liquid limit. The results from the different slopes have therefore been put together in different ways, both as a bulk and as results from a selected soil layer with relatively homogeneous properties in terms of plasticity, Fig. 172. In Sundholmen, only the results from the relatively homogeneous clay below the delta and lateral fluvial sediments have been incorporated.

As can be seen in the figure, b-values between mainly 0.89 and 1.04 could be evaluated from the different compilations with an average value of 0.97. This is approximately the same range and average value as those presented for similar clays by Jamiolkowski et al. (1985).

The Swedish experience, which forms the basis for the normally employed correction factors for field vane tests, is based on normally consolidated and only slightly overconsolidated soils. A certain caution is therefore called for when applying the results of field vane tests in more overconsolidated soils, for example at the toes and under the rivers in slope stability investigations of eroded watercourses. A correction in heavily overconsolidated soil is obtained automatically when a combined analyses is used since the drained shear strength becomes governing at high overconsolidation ratios. However, there is a certain span with over

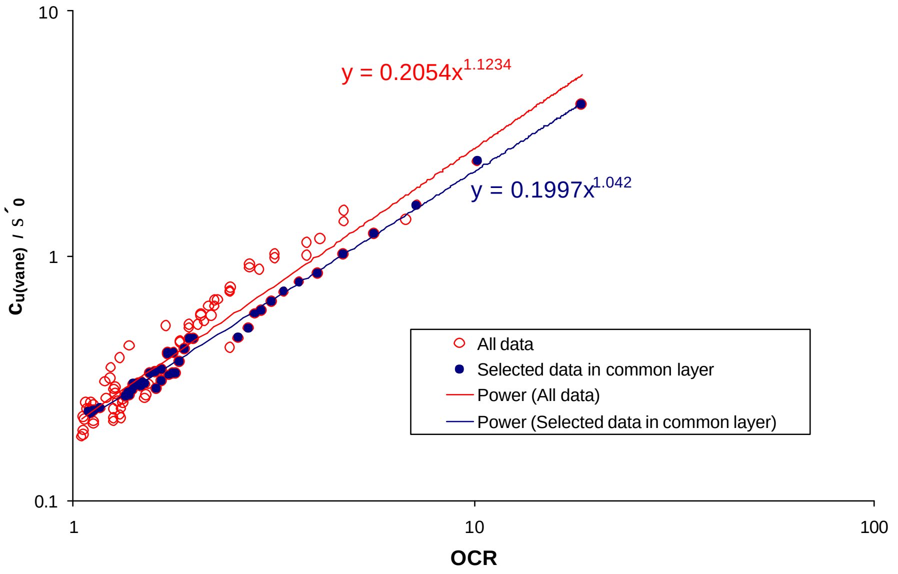


a)

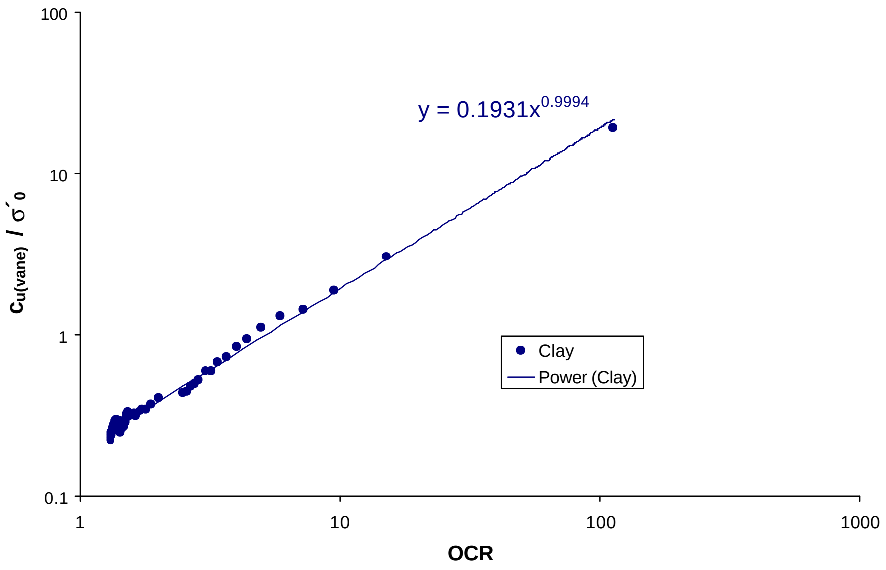


b)

Fig. 172. **Evaluated undrained shear strengths from the field vane tests in the different slopes as functions of effective overburden pressure and overconsolidation ratio.**
a) Torp, Section A
b) Torp, Section C



c)



d)

Fig. 172. Evaluated undrained shear strengths from the field vane tests in the different slopes as functions of effective overburden pressure and overconsolidation ratio.
c) Strandbacken, Lilla Edet
d) Sundholmen

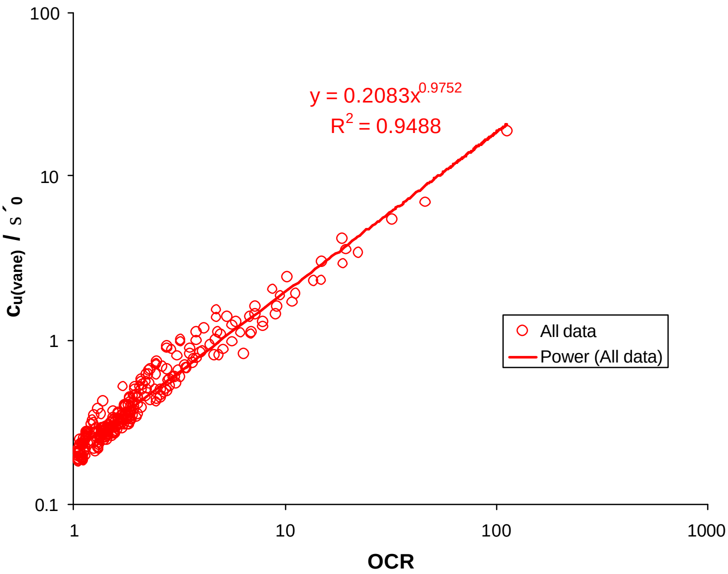


Fig. 172. **Evaluated undrained shear strengths from the field vane tests in the different slopes as functions of effective overburden pressure and overconsolidation ratio.**
e) All data

consolidation ratios that are higher than normal, for which the correction factors have been determined, but lower than the level at which the drained shear strength becomes governing. This means that an extra correction should be required in this interval also when combined analyses are used.

The need for a correction of the undrained shear strengths measured by field vane tests for overconsolidation has been mentioned only sparsely in the literature. The first observation about this, to the authors' knowledge, was made by Leroueil et al. (1983). They pointed out that the shear strength values measured by field vane tests would require a correction factor of about 1/(0.9 OCR) to correspond to the back-calculated shear strength at slides in overconsolidated Champlain clay. However, this was not intended as a real proposal for a correction factor but rather intended to illustrate that the field vane tests, which had been found to be a useful tool in normally or only slightly overconsolidated Scandinavian clays, was less suited for overconsolidated Canadian clays where use of effective stress parameters and analyses was considered more appropriate.

The first and, to the authors' knowledge, hitherto only real proposal for a correction of the results of field vane tests for overconsolidation was presented by Aas et al. (1986). In this proposal, the soil is divided into the three categories “Young, Aged and Overconsolidated”. “Young” here stands for normally consolidated soils and “Aged” represents soils which with time have attained a certain overconsolidation because of creep effects and possibly other processes without having been subjected to a corresponding load. These two categories are merged into a soil group designated “normally consolidated, NC”, which corresponds to what in the Swedish classification system would be designated “normally consolidated or only slightly overconsolidated soil”. In order to separate these soils from overconsolidated soils, “OC”, the quotient between the measured shear strength value and the effective overburden pressure in relation to the plasticity index is studied, Fig. 173.

From the given chart, it can be concluded that “normally consolidated, NC, clays” refers to clays with an overconsolidation ratio less than about 1.5. The correction factors, μ, for the two types of soil are then determined from the curves in Fig. 174 on the basis of the quotient between the measured shear strength value and the effective overburden pressure. No guidance for a possible interpolation between the two curves is given, but the full correction for overconsolidation is to be applied to all soils that are classified as overconsolidated in the first chart.

The correction factors that are determined in this way for normally consolidated clay are usually very similar to those obtained according to the rules for correction factors used in Sweden (Larsson et al. 1984). This is to be expected since both methods are based on much of the same data. However, they differ for very high-plastic soils because the Norwegian database for such soils includes results from South-east Asia whereas the Swedish database relates to the types of organic clay, gyttja and highly decomposed peat that are found in Sweden and Finland.

The correction factors differ in overconsolidated soils since the Swedish database only includes normally consolidated and only slightly overconsolidated soils, whereas the Norwegian database includes also overconsolidated soils. No correction of the results from field vane tests for overconsolidation has hitherto been made in Sweden except for that which is made indirectly when using a combined analysis. In principle, it therefore appears that for evaluation of a relevant undrained shear strength an extra correction, μ_{OCR}, should be applied in addition to that which in Sweden is made on the basis of the liquid limit of the soil. According to the empirical values for how the results of field vane test are affected by overconsolidation, this correction should be approximately

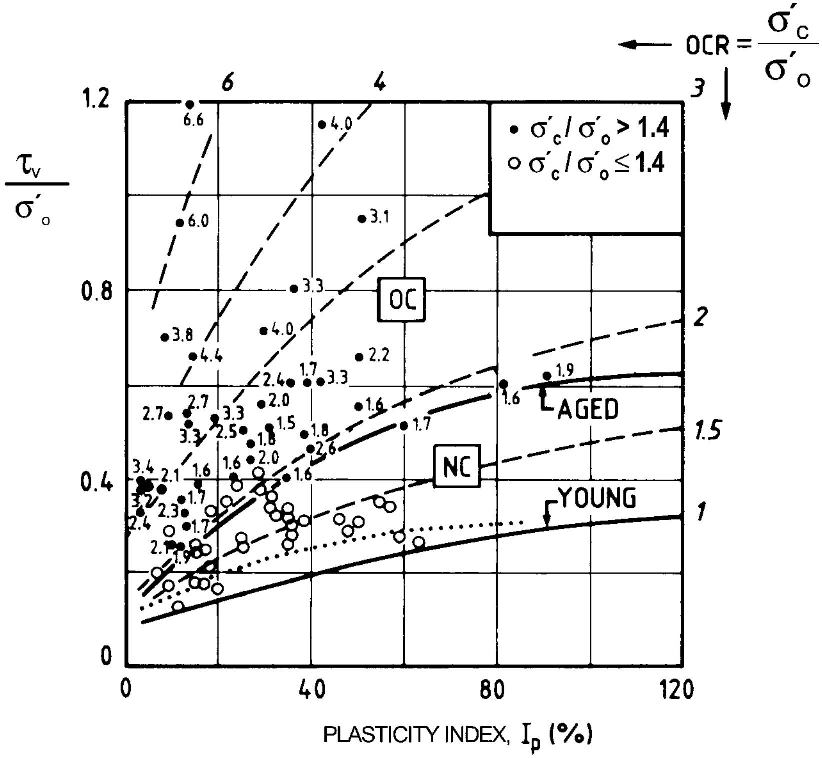


Fig. 173. **Chart for separation of normally consolidated and overconsolidated clays on the basis of the quotient between the measured shear strength value and the effective overburden pressure in relation to the plasticity index (Aas et al. 1986).**

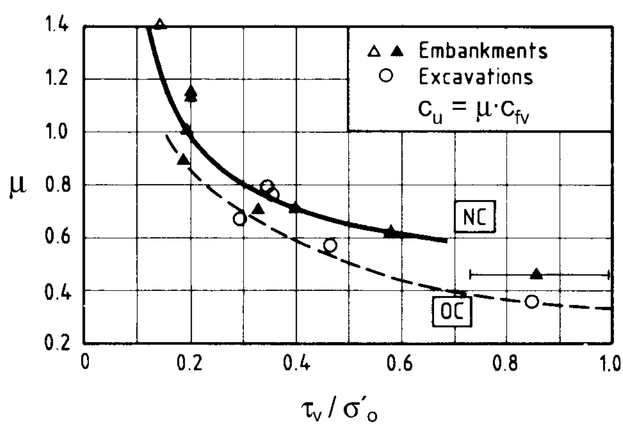


Fig. 174. **Determination of correction factors for field vane tests in "normally consolidated", NC, soils and overconsolidated, OC, soils respectively (Aas et al. 1986)**

$$\mu_{OCR} \approx OCR^{-0,15}$$

However, the ordinary correction with respect to liquid limit is based on soils with an overconsolidation ratio of about 1.3, and corrections for overconsolidation up to this value can be assumed to be included. The evaluation of undrained shear strength from from results of field vane tests with combined corrections thereby becomes

$$c_u = \tau_v \; \mu_{w_L} \; \mu_{OCR}$$

which according to the considerations described becomes

$$c_u = \tau_v \left[\frac{0.43}{w_L} \right]^{0.45} \left[\frac{OCR}{1.3} \right]^{-0.15}$$

A determination of the overconsolidation ratio *OCR* is not always available. A preliminary estimation with a normally adequate accuracy for this purpose can then be performed using the Hansbo (1957) relation, with or without modification for overconsolidation effects.

The effect of this correction for overconsolidation is small or non-existent for normally consolidated and only slightly overconsolidated soils but considerable in heavily overconsolidated soils. The effect on the calculated safety factor in an eroded slope where the soil behind the crest is normally consolidated or only slightly overconsolidated is normally small since the strength in only a limited part of the slip surface will be significantly affected.

For CPT tests, it is proposed that a revised evaluation method be used based mainly on the results of the present project. In this method, the overconsolidation ratio is considered by using the equation

$$c_u = \frac{q_T - \sigma_{v0}}{13.4 + 6.65 w_L} \left[\frac{\sigma'_c}{1.3\, \sigma'_{v0}} \right]^{b-1}$$

No directly corresponding proposal for correction of the evaluated shear strength from CPT-tests for overconsolidation has been found in the literature. However, proposals have been made for a varying of the cone factor with respect to the pore pressure parameter *Bq* in such a way that a lower *Bq*-value should correspond to a higher cone factor and thereby a lower evaluated undrained shear strength (e.g. Karlsrud et al. 1996). Since a lower *Bq*-value indicates a higher overconsolidation ratio (e.g. Robertson et al. 1986, Mayne and Holtz 1988, Larsson and Mulabdic 1991), this entails an indirect correction for overconsolidation ratio. A general use of the proposed method is therefore to be recommended.

A correspondingly limited effect of unloading on the evaluated undrained shear strength is found in the results of dilatometer tests if the evaluation method recommended in SGI Information No. 10 (Larsson 1989) is used. Internationally, the general equation for the variation of the undrained shear strength with effective overburden pressure and overconsolidation is more commonly used. In this evaluation, the values $a = 0.22$ and $b = 0.8$ taken from empirical pattern for clays are normally used and the equation thus becomes

$$c_u = 0.22\,\sigma'_{v0}\ OCR^{0.8}$$

This evaluation is indirect and uses the overconsolidation ratio that is evaluated from the dilatometer test results together with the in situ effective overburden pressure (Marchetti 1980). The equation implies that the expected effect of an overconsolidation is evaluated. A condition for the applicability of the method is that a relevant method for evaluation of the overconsolidation ratio is used. For Swedish conditions, it should thus only be used in connection with the method for the latter evaluation that is proposed in this report (see next section “Determination of other properties”). The factor $a = 0.22$ is an average value for clays. The corresponding factor for organic soils is generally higher, which means that the undrained shear strength in organic soils is underestimated by this method. An alternative generally applicable factor for organic soil is difficult to give since it has been found to vary between 0.22 and 0.45 depending on the type of organic soil and the organic content (Larsson 1990). In the examples given in this report, this evaluation method thus generally provides fairly good estimates of the undrained shear strength in the clays in the soil profiles, whereas it generally underestimates the shear strength in the upper layers containing organic matter unless an *a*-factor which has been determined by direct simple shear tests is used.

The new evaluation methods are intended to provide more relevant shear strength determinations, which should also bring the results from the different test methods into better concordance. This improved concordance is illustrated when the undrained shear strengths determined by direct simple shear tests and CPT tests, field vane tests and dilatometer tests evaluated by the new methods are shown together in those investigation points where such results are available, Figs. 175-178. In these presentations, the results are much closer than before in all parts where the soil is overconsolidated (compared with the previously presented test results from each investigated slope). However, certain significant differences still remain. This in particular relates to the field vane tests and the dilatometer tests in the upper sandy, silty and organic delta and lateral fluvial sediments, even when these results have also been brought into better concordance with the others by the new evaluation methods. This is related to the fact that other factors in the evaluations, such as the correction factor with respect to the liquid limit and the a-factor, are not relevant for this type of soil. For the dilatometer tests in Sundholmen, this was remedied by inserting the a-factor determined by direct simple shear tests in the evaluation. Significant scatter in the test results, particularly for field vane tests, can occur in layers where the clay contains layers of coarser material of embedded coarser objects, such as e.g. shells and gravel particles.

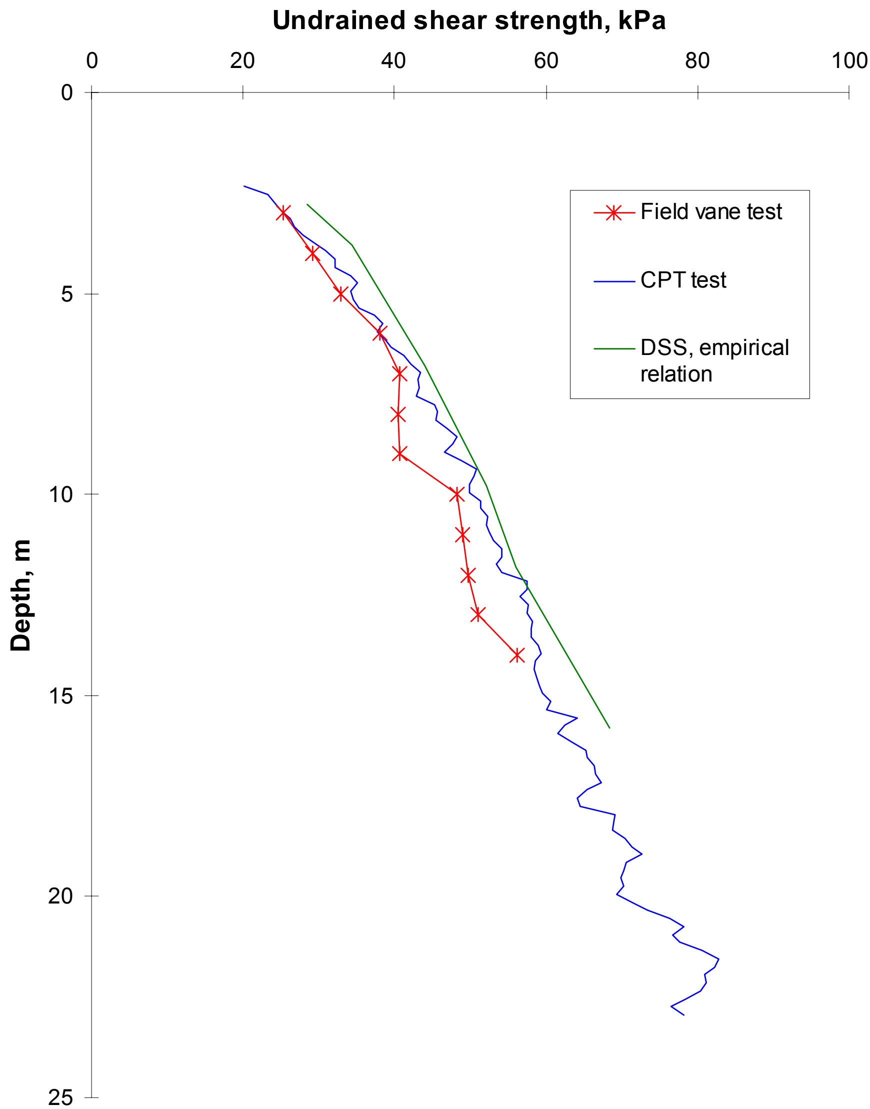


Fig. 175. Evaluated undrained shear strengths after correction for overconsolidation ratio in Torp, Section A.
a) Point S13 below the river

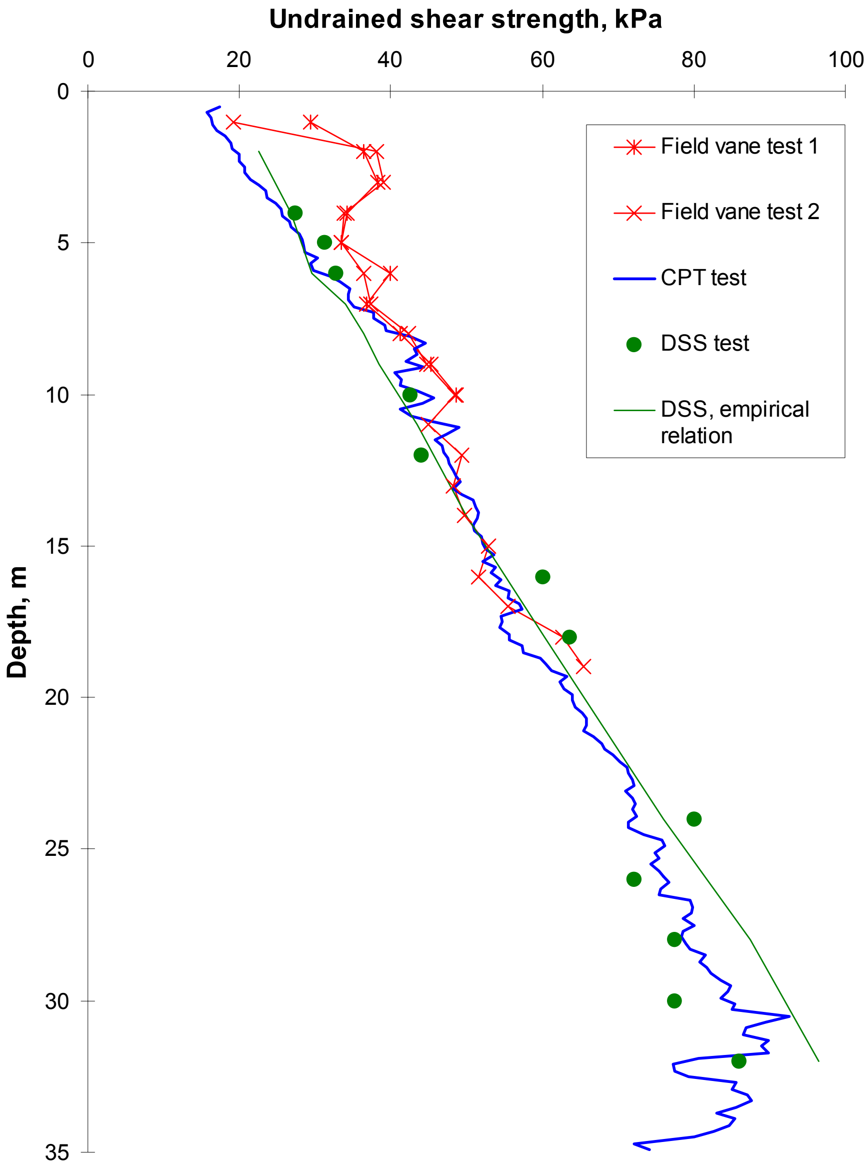


Fig. 175. Evaluated undrained shear strengths after correction for overconsolidation ratio in Torp, Section A.
b) Point S2 on the excavated terrace

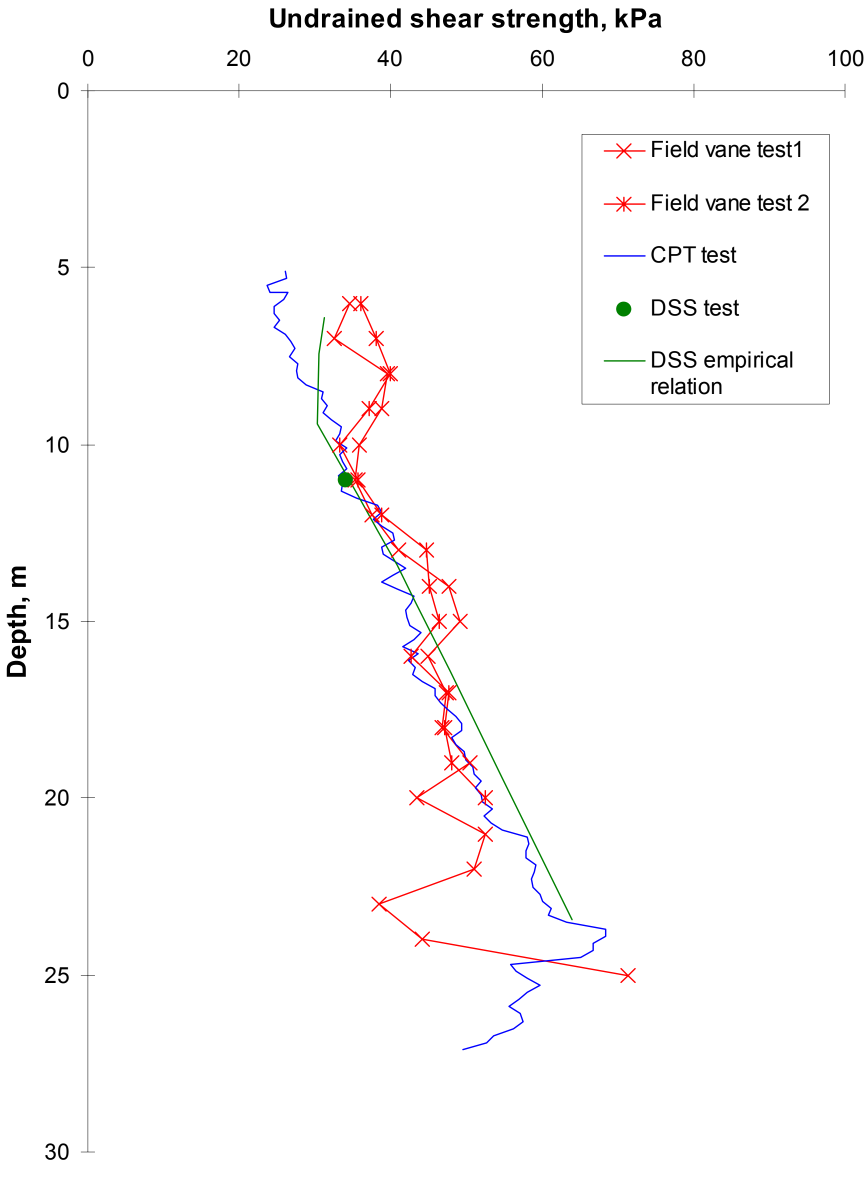


Fig. 175. Evaluated undrained shear strengths after correction for overconsolidation ratio in Torp, Section A.
c) Point S3 behind the upper crest

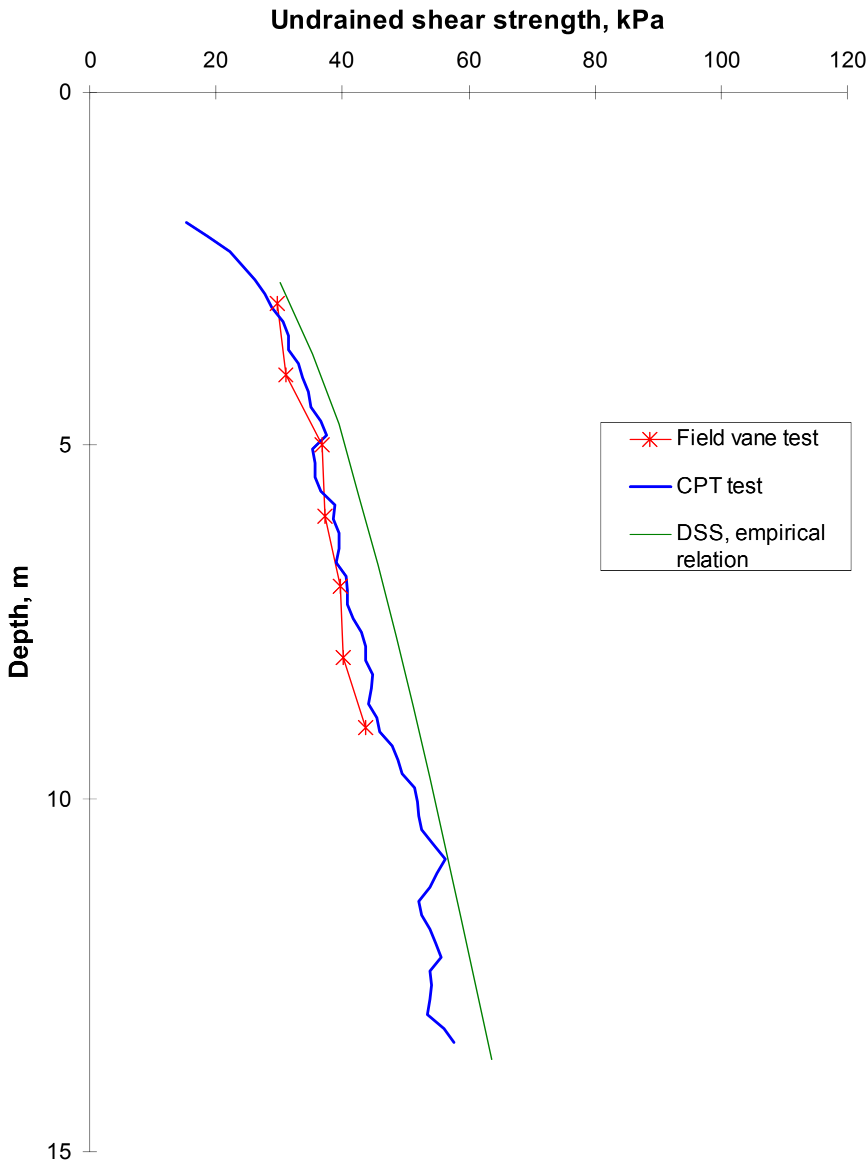


Fig. 176. Evaluated undrained shear strengths after correction for overconsolidation ratio in Torp, Section C.
a) Point S7 below the river

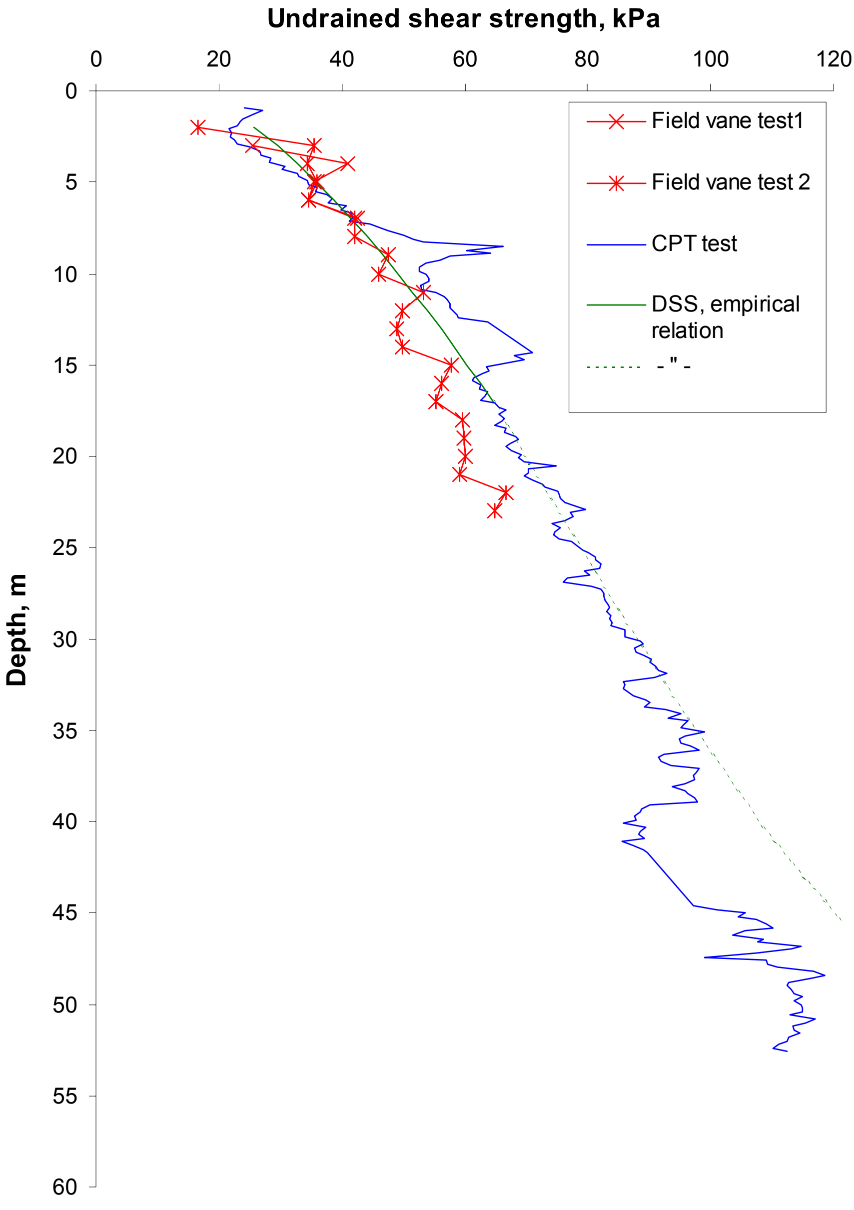


Fig. 176. Evaluated undrained shear strengths after correction for overconsolidation ratio in Torp, Section C.
b) Point S8 on the lower excavated terrace

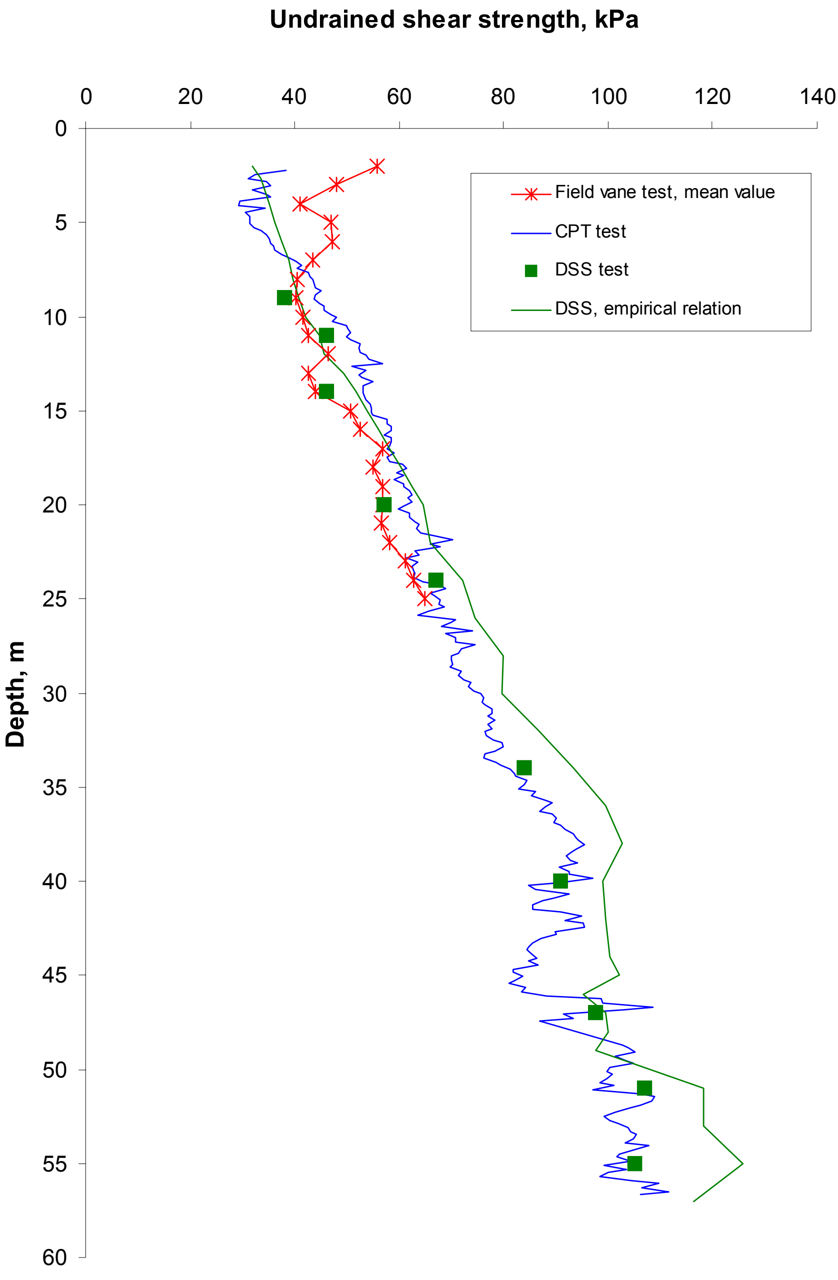


Fig. 176. Evaluated undrained shear strengths after correction for overconsolidation ratio in Torp, Section C.
c) Point S9 on the upper excavated terrace

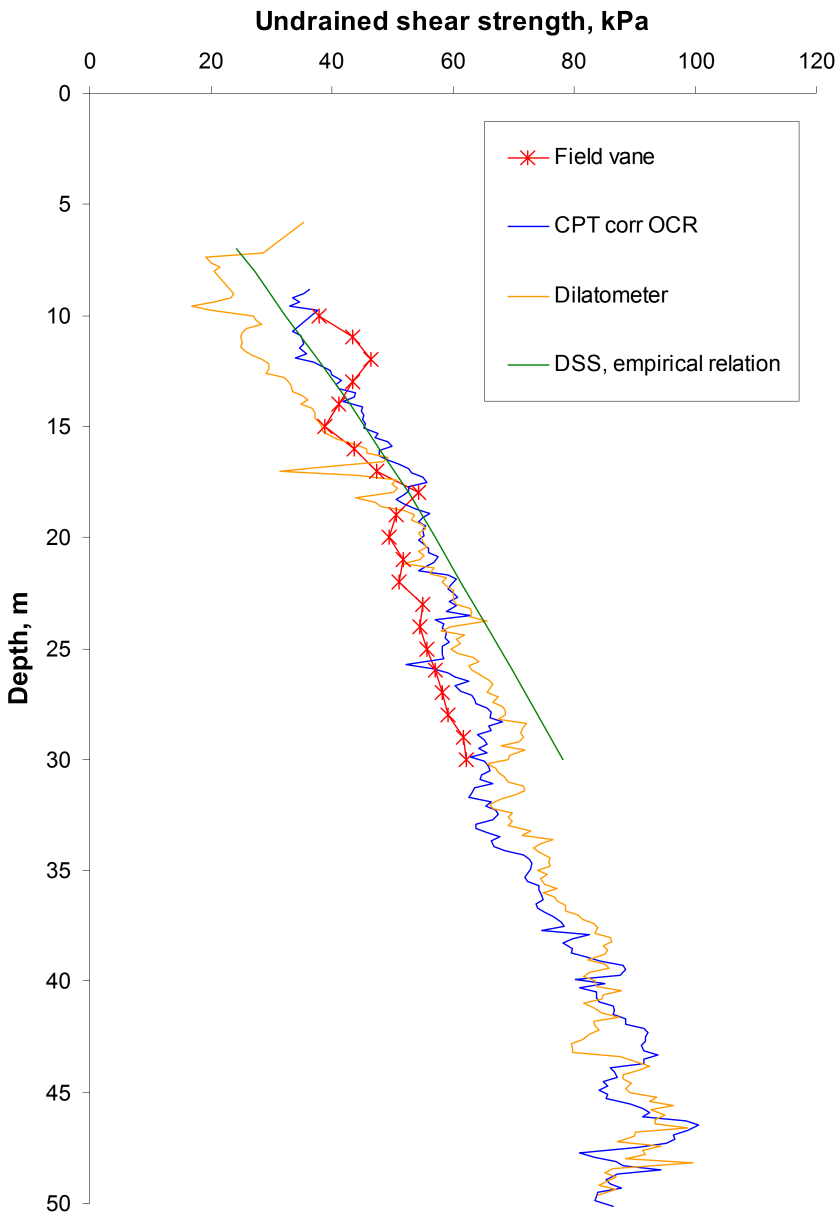
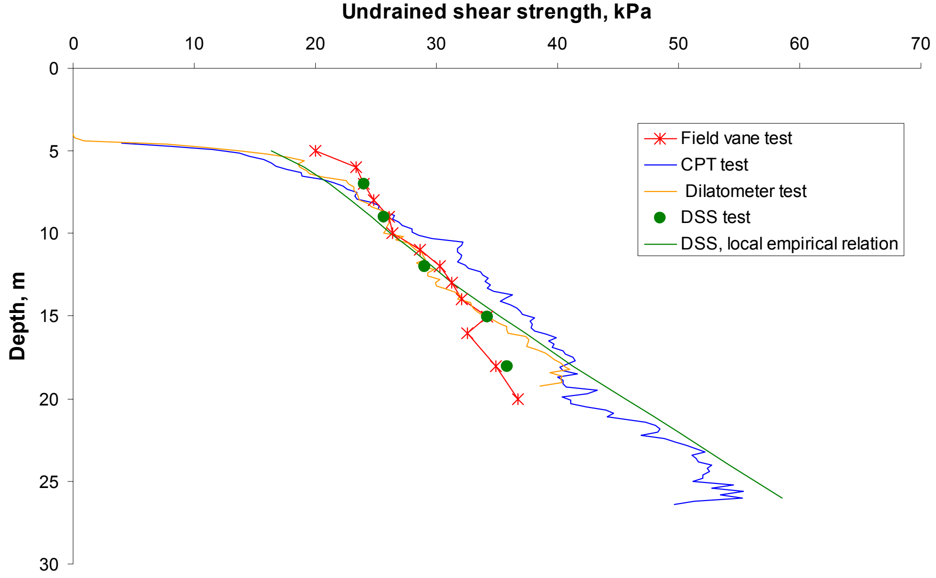
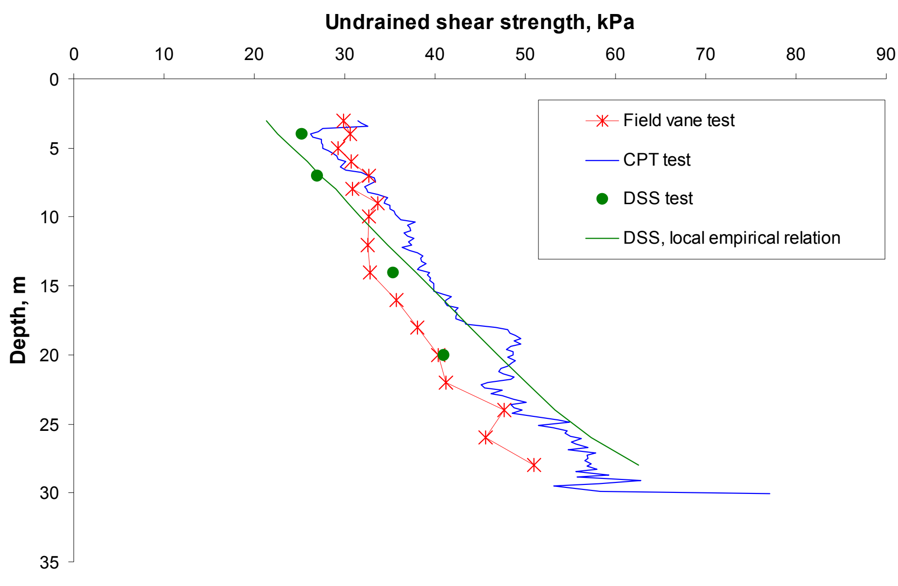


Fig. 176. **Evaluated undrained shear strengths after correction for overconsolidation ratio in Torp, Section C.**
d) Point S11 behind the upper crest



a)



b)

Fig. 177. Evaluated undrained shear strengths after correction for overconsolidation ratio in Strandbacken.
a) Point 1 below the river
b) Point 2 on the riverbank

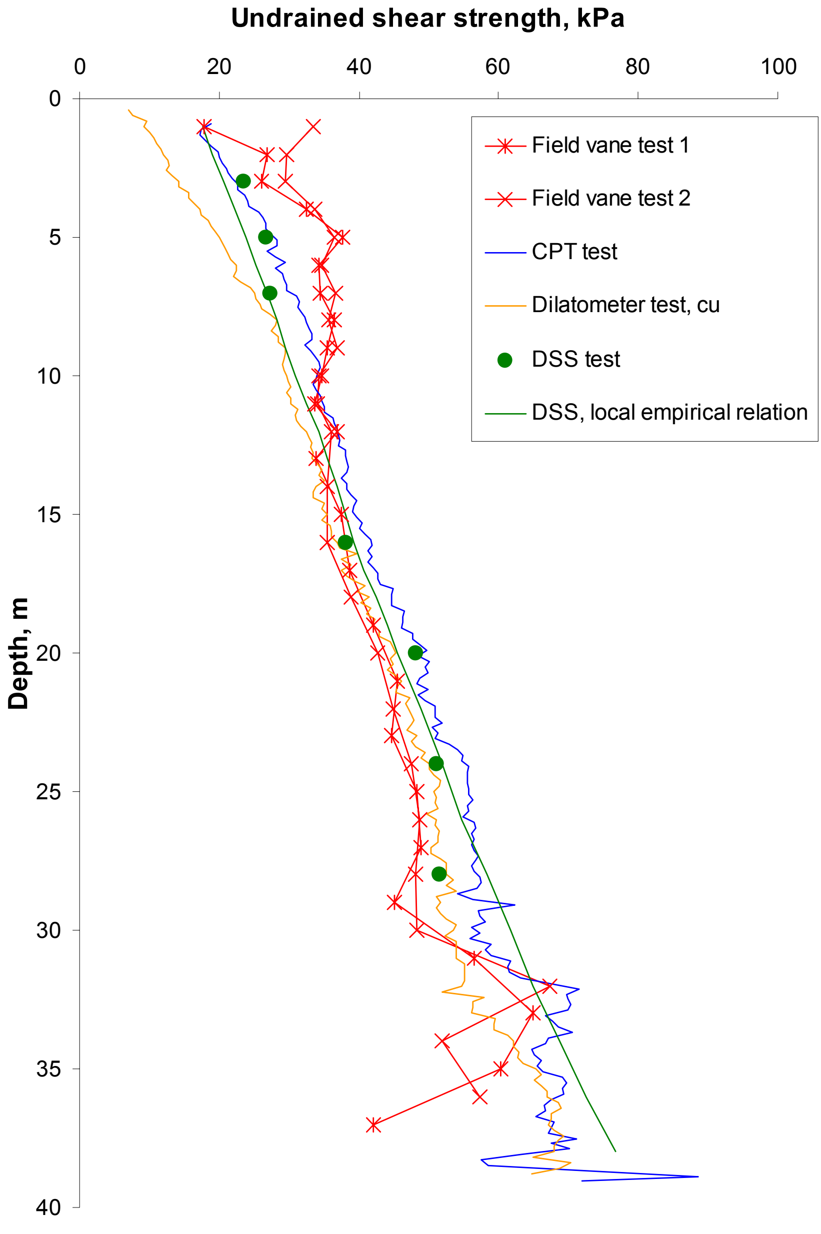


Fig. 177. Evaluated undrained shear strengths after correction for overconsolidation ratio in Strandbacken.
c) Point 4 on the excavated terrace

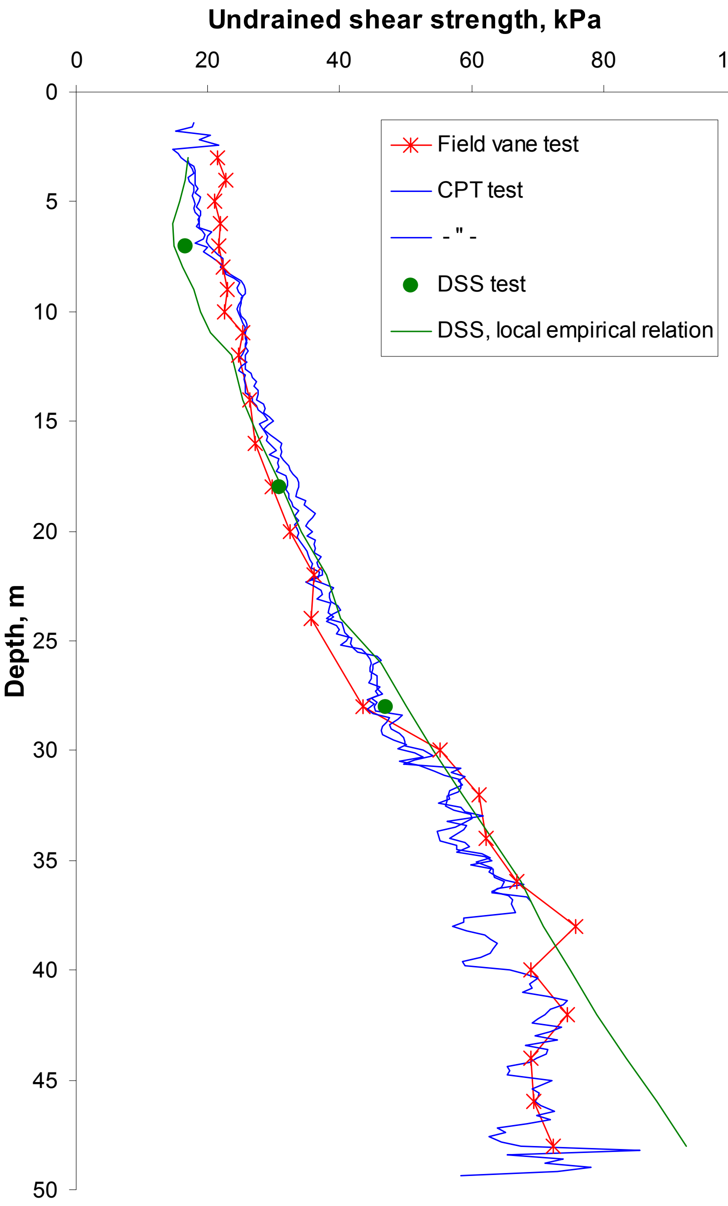


Fig. 177. **Evaluated undrained shear strengths after correction for overconsolidation ratio in Strandbacken.**
d) Point 6 behind the upper crest

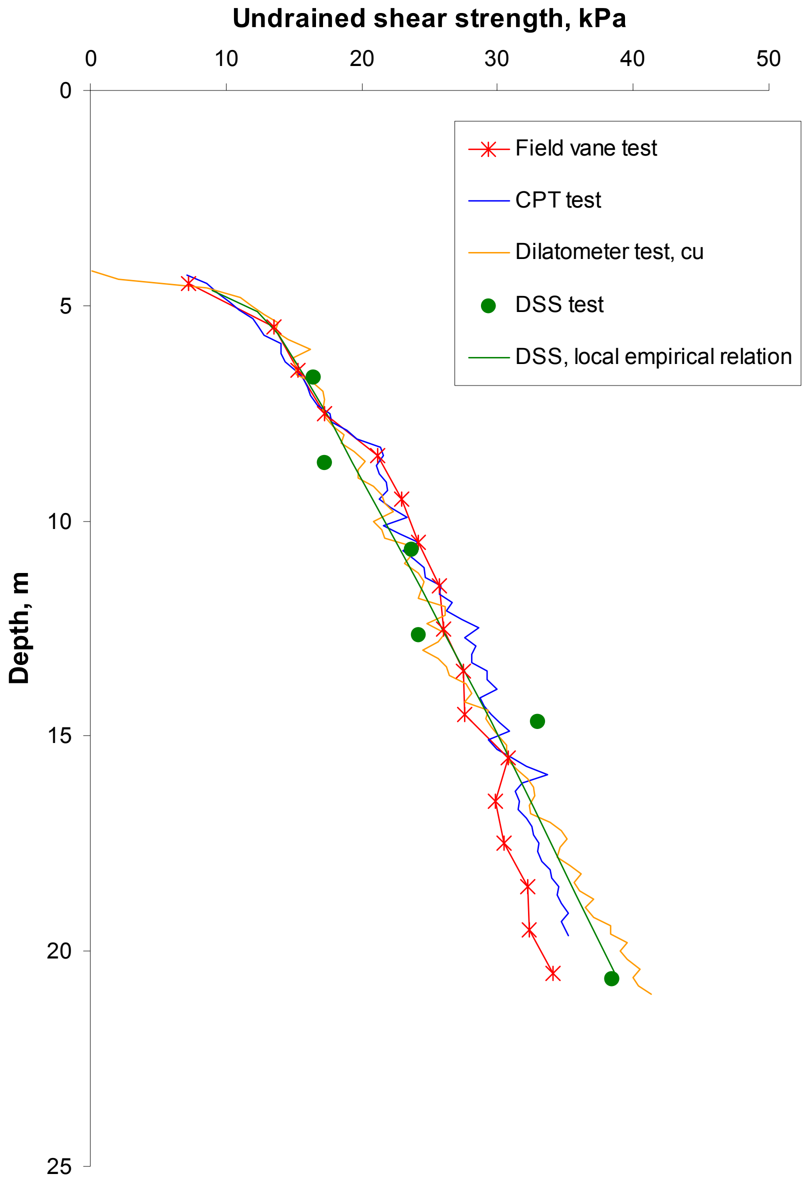


Fig. 178. **Evaluated undrained shear strengths after correction for overconsolidation ratio in Sundholmen.**
a) Below the river

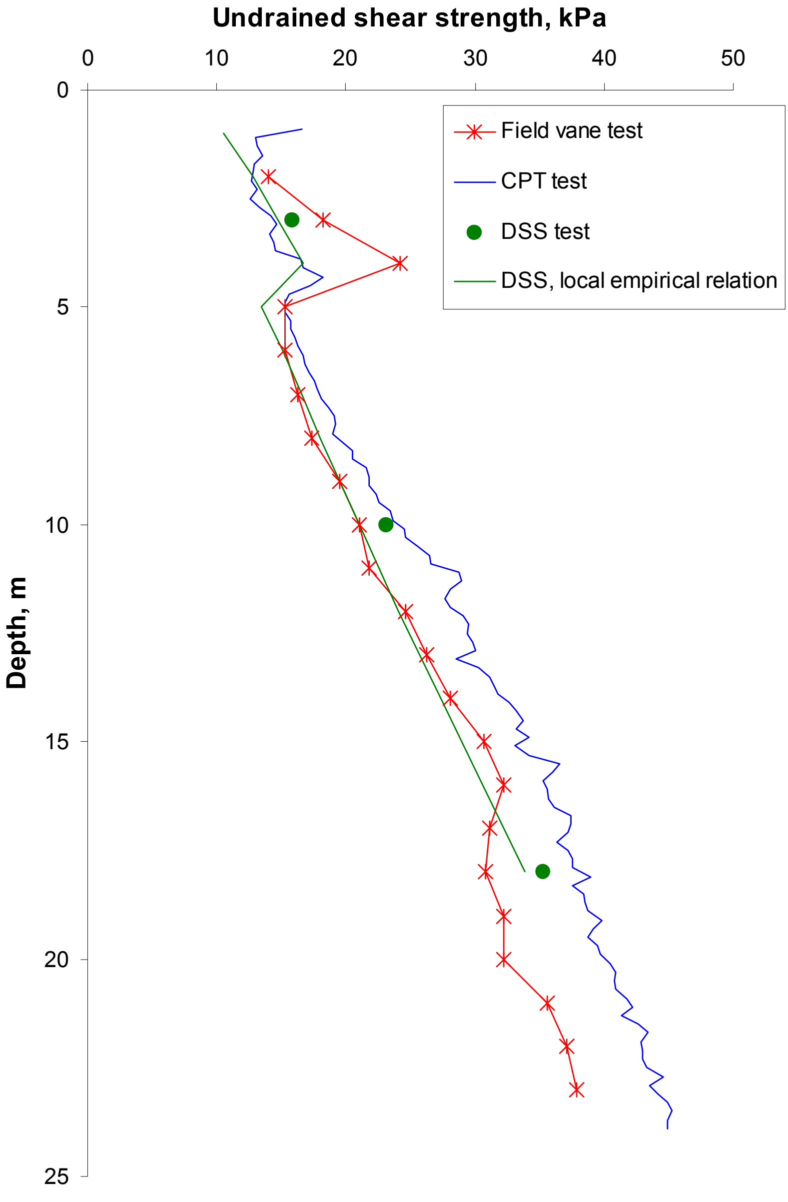


Fig. 178. **Evaluated undrained shear strengths after correction for overconsolidation ratio in Sundholmen.**
b) On the excavated terrace

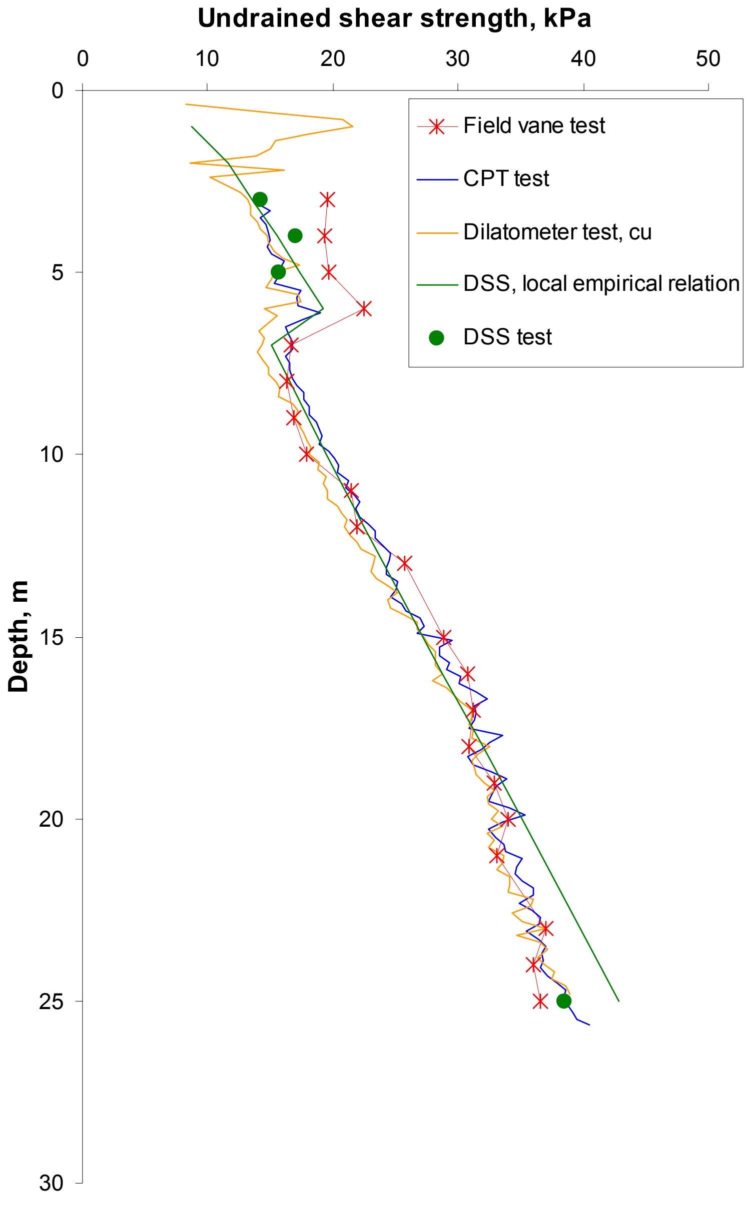


Fig. 178. Evaluated undrained shear strengths after correction for overconsolidation ratio in Sundholmen.
c) On natural ground behind the upper crest

6.1.3 Determination of other properties

The soil properties in terms of density, natural water content, liquid limit and sensitivity are determined in routine tests in the laboratory.

The approximate determination of the *bulk density*, which is obtained from CPT tests and dilatometer tests, is often adequate for a preliminary evaluation of these tests but not for estimation of the self-weight of the soil in calculation of slope stability. For the latter purpose, the density should be determined on undisturbed samples in the laboratory. This is also valid for the density in all thicker layers of silty and sandy soil.

The *natural water content* and the liquid limit in clay can be determined on both disturbed and undisturbed samples, but a closed sampler should be used. A determination of the *sensitivity* normally requires undisturbed samples but can alternatively be done using undisturbed shear strength from field tests and remoulded shear strength determined on disturbed samples. In the investigated slopes, which are all located in western Sweden, it was found that the value of 1.1 for the quotient between natural water content and liquid limit proposed by the Göta-älv Committee (1962) as a lower limit for the possible occurrence of *quick clay* was applicable. The results of the geophysical investigation, which was carried out in the Torp area, also indicate that the lower limit for the resistivity of 5–10 Ωm for a possible occurrence of quick clay reported by Söderblom (1969) is also applicable. The corresponding limit in Torp was about 7 Ωm. The limits can be used to exclude the possibility of quick clays within those parts of the soil masses that fall outside the limiting values. However, a possible occurrence of quick clay in areas within the limits always has to be verified by sampling.

The presence of quick clays normally results in enhanced pore pressure ratios and very low friction ratios in CPT tests. This can be used as support for the estimation of the extent of the quick clay formation, but the occurrence of the quick clay, like all other more detailed soil classifications of different soil layers, must always be verified by sampling.

The most rational way of determining the *permeability* of the soil is by CRS oedometer tests on undisturbed samples in the laboratory. The *preconsolidation pressure* of the soil is also determined by these tests. CRS tests that are performed in slope stability investigations normally only aim at determining these two parameters. They can therefore be stopped after a shorter time than is required for determination of all compressibility parameters and require less evaluation.

The preconsolidation pressure in the soil can be estimated fairly well by CPT tests and dilatometer tests. This has been shown by the test results presented. However, conditions for this are in both cases that the presented evaluation methods are employed. This means that the preconsolidation pressure is evaluated from CPT tests using the equation

$$\sigma'_c = \frac{q_T - \sigma_{v0}}{1.21 + 4.4\, w_L}$$

The overconsolidation ratio is evaluated from the dilatometer tests by the equations

$$OCR = 10^{0.16(K_D - 2.5)} \qquad K_D \leq 5$$
$$OCR = 2.51 + 0.368(K_D - 5) \qquad 5 > K_D \leq 7.5$$
$$OCR = 0.24 K_D{}^{1.32} \qquad K_D > 7.5$$

whereupon the preconsolidation pressure can be calculated with knowledge of the effective overburden pressure.

The preconsolidation pressures evaluated from CPT tests and dilatometer tests have to be verified by oedometer tests, but the number of oedometer tests can be reduced in relation to the case when the former data are missing. The results of the field tests are a very useful guidance for estimation of the trends with depth and when connecting the oedometer data to a profile with depth, even if there may be some discrepancies in absolute values between the field and laboratory tests.

6.1.4 Pore pressure measurements and modelling of pore pressure distribution

All pore pressure measurements in fine-grained soils should be performed with closed systems. The material in the systems and all connecting tubes, pipes and rods should be selected carefully in order to avoid as far as possible evolution of gas. However, equipment for removal of gas and saturation of the systems should always be at hand to enable control and remedy of possible gas formation. The location of the pore pressure systems should be selected with guidance from the CPT test results. Systems should be placed in all presumed water transmitting layers in such way that the continuity and possible pressure differences along the layers can be observed. A reference system should also be installed to monitor the water levels in any connecting watercourses.

These data together provide the boundary conditions required for a calculation of the groundwater seepage and for outlining a model of the pore pressure distribution in the slope. The pore pressure systems and the water levels should be recorded over a sufficient time to allow a study of the natural variations and how these occur within different parts of the slope and affect each other. A prognosis of the maximum variations should be made, and the risk of possible remaining high pore pressures within the soil mass in the event of rapid lowering of the water levels in the watercourses should also be estimated.

The results of this project indicate that fairly good general models can be created for the pore pressure distribution and variation in the soil masses within a slope provided that the soil profiles are not too complicated. They also indicate that a reasonably accurate prognosis can be made for the changes in pore pressure conditions after a stabilising measure that changes the geometry of the slope. Some details can be difficult to model, such as the effects of precipitation and particularly evaporation. These effects would need further studies to provide better guidelines. However, it is normally the maximum pore pressures that are of chief interest for the stability. If the maximum infiltration in the calculation models is exaggerated, this often becomes obvious from improbable effects in the calculation results. The results of the calculations therefore have to be carefully scrutinised. Depending on how the calculation model is designed, fictitious surface layers may have to be introduced in order to model seepage water streaming out at some parts in the slope and then being re-infiltrated in other parts. The use of different calculation programs is thus often not quite straightforward but requires a certain experience. A detailed modelling of all different conditions with different intensity and duration of precipitation, evaporation, water levels in watercourses under different circumstances etc. would also be very laborious.

Whether or not the effect of different drainage systems can also be adequately modelled cannot be judged from the results of this investigation. This has not been tested since such measures were only made to a limited and unspecified extent in one of the investigated slopes.

Some additional pore pressure systems installed at the centre of thick clay layers may be required to verify the calculated model. However, supplementary pore pressure systems are primarily required in those parts of the slope where the created model and the measured shear strength properties show that the drained shear strength may be dimensioning. This normally applies to the outer parts of the slope where the overburden pressures are relatively low. A detailed knowledge of the properties of the superficial soil layers is required to accurately model the pore pressure situation in this part of the slope. This is often difficult to obtain because large parts of this soil volume are often heterogeneous with a character of a dry crust with cracks and fissures, root systems of trees and other plants etc. For this reason, it is often more rational to install a larger number of pore pressure systems within this zone and to make a cautious manual estimate of the most critical pore pressure conditions.

The results of this project show clearly that the pore pressures are of great importance also in areas with "soft normally consolidated" clay, particularly for shallower slip surfaces in the outer parts of the slopes.

6.2 ENVIRONMENT AFTER EXCAVATION OF SLOPE CRESTS

The observations within the three areas show that the environment after an excavation according to the hitherto most commonly used method for excavations at slope crests is often not quite like what would have been desirable. Such excavations are mostly performed within or near built-up areas and there is often a strong demand to use the excavated areas for gardens, parks or other recreation purposes. Their common location above watercourses with a natural view over these means that they have very good preconditions to become nice environments for many forms of human recreation. However, this purpose requires that they are provided with sufficient inclinations and drainage systems to lead off both surface water and any water seeping out from the rear excavation slope and to prevent any waterlogged areas from occurring. This requirement is valid both if open areas are desired and if it is desired that any other growth should be established than that which occurs naturally in marshy areas.

This was clearly illustrated in the investigated areas, and the contrast between the environments on the drained lots in Sundholmen and the undrained areas there and in the other areas is striking. This is in spite of the fact that the drainage systems in Sundholmen are fairly shallow and not quite functioning all the year around.

The designs of the excavations have also entailed that the increase in the stability of the outer parts of the slopes has not been quite as large as would have been desirable. The improvement that has been achieved is also not primarily due to the excavations but to the fact that erosion protection structures have been constructed at the toes and decreased the average inclinations of the remaining slopes. The overall stability together with the environment would benefit if the design of the excavation involves a larger and preferably also varied inclination and if an effective drainage system is installed to permanently lower the groundwater level below the excavated areas.

The erosion protection structures, apart from contributing to the improved stability, have entailed that the previously ongoing erosion processes have stopped. This in turn has meant that the previous scars in the slopes from superficial slips have healed and at the same time that the littery impression created by trees that had fallen in the slopes or slid out into the watercourses has ceased. The construction of the erosion protection in Torp and Strandbacken has also had the result that shelves have been created above the highest water level, which constitute walking paths along the river and which can be used by fishermen. These are benefits for both the environment and for the possibility to use the areas for recreation.

6.3 SOIL DEPOSITION AND STRESS HISTORY

All the investigated areas are located in western Sweden and the loose soil layers have mainly been deposited in connection with the melting of the inland ice and the sedimentation that occurred in the sea outside the front of the retreating ice. The areas are located in valleys with surrounding ridges of rocky hills and they have risen above the sea because of the land-elevation. After the areas emerged above the sea level, rivers were formed and have eroded their courses down through previously deposited sediments, which have then been redeposited outside the new coastline. Coarser sediments have been deposited as delta or lateral fluvial sediments close to the river mouths and have in turn successively been eroded and redeposited when the coastline has moved out towards the present coastline.

It can be assumed that the areas were rather flat and horizontal just as the coastline passed over them and that the groundwater level was then located at the ground surface. All soil layers in the area can be assumed to have consolidated for the stress conditions prevailing at that time. Thereafter, the rivers have eroded their courses down through the upper soil layers and the overburden pressures below them have thereby decreased. This has been a very slow process that still continues. Also the soils below the watercourses may therefore have attained a certain overconsolidation in relation to the maximum effective overburden pressures because of creep processes at times when the stresses still were high. Such an effect appears to be detected in most of the investigated areas, but the exact size is more difficult to judge since the previous stress history can only be assumed.

The erosion process and the consequent successive lowering of the water level in the created river-course have entailed that the groundwater level has been lowered in the adjacent ground. This has resulted in increased effective stresses and the soil has consolidated for these at the same time as creep processes have brought a further increase in preconsolidation. The largest groundwater lowering in the areas outside the slopes has occurred close to their crests. As the watercourses have eroded down through the sediments, they have also been widened because of erosion and slides in the slopes and a successively larger volume of soil has thereby become unloaded and overconsolidated. All soil masses below the watercourses and their slopes are thus overconsolidated because of the stress history. This also means that the highest preconsolidation pressures, and thereby also the highest shear strengths, are normally found just behind the natural crests of the slopes. The groundwater lowering is normally less far behind the crests and the preconsolidation pressures and shear strengths are thereby somewhat lower.

The differences in stress history for the soil masses behind the slope crests which are related to the distance from this can be marginal or highly significant depending on the size of the groundwater lowering, how long the different stress conditions have prevailed etc. They have been clearly traceable in many cases in the present project.

A further change in properties can occur when clay which has been deposited in salt water is leached. This may mean that preconsolidation effects created by previous stresses and creep effects over a long time are broken down and the soil turns into a state that is more normally consolidated in relation to the current stresses. Thereby the undrained shear strength also decreases. Such effects are indicated by the test results in points far behind the crests in both Torp and Strandbacken.

The stress history outlined above is common in western Sweden, but can of course vary because of anomalies during the deposition, larger landslides that have occurred during the time from deposition to the present day, human activities etc. In the investigated areas, the stress history could be inferred from the test results and provided a logical explanation for the observed variations in these. It was also a support for the creation of overall models for the variation of the strength within the areas.

A plausible model for deposition and stress history is a prerequisite for the creation of a reliable model for the shear strength properties. Even if the stress history may vary between different areas, it is reflected in the test results and can normally be described with support from knowledge of the geological history of the area and the test results. The variation of the preconsolidation in the area is most rationally mapped by performing CPT tests with a sufficient accuracy and a verification and calibration of the results by an adequate number of oedometer tests.

6.4 VARIATIONS IN SOIL CONDITIONS

As a rule, there is a certain variation in the soil profiles also in areas with so-called "homogeneous" soil conditions. A relatively large variation could thus be observed in the upper layers consisting of delta and lateral fluvial sediments with sand, silt and organic material in the three areas that have been studied. The thickness of these layers varied from one or two metres to about 10 metres and the variation was not necessarily regular but could vary pretty much at random. This in turn can be related to the varying shape of the delta during the period of deposition. Nor is the composition of these upper sediments uniform; it varies with different grain size distributions and organic contents. These variations may be gradual and relatively small, but can also, for example, constitute embedded clay layers creating separate ground water aquifers with different groundwater heads, as in Section C in Torp, or different soil layering in different parts in the area, as in Sundholmen, where a three metre thick gyttja layer was found on one side of the river but not on the other. These anomalies could likewise be related to the varying geometry and currents in the previous river deltas.

The thickness of the underlying clay layers may vary as well, partly related to the varying thickness of the fluvial sediments under the mainly flat ground surface, partly related to the variation in the underlying firm bottom. This was most significant in Section A in Torp, where the depth to firm bottom was smallest and where the latter was heavily inclined. On the other hand, no variation was observed in the clay layers in Sundholmen where no significant variation in the thickness of the upper sediments was measured and the depth to firm bottom was very large within all the investigated area.

The natural water content related to the level varies depending on the thickness of the clay layers within the area. It may to some extent be related to differences in preconsolidation pressure at different distances from the watercourses, but this is of marginal importance in the studied areas. If large areas are studied, like the 1.5 km long distance south of the Ström lock in Lilla Edet that was studied in the Göta-älv investigation in 1962, there will usually be fairly large variations. This depends, among other things, on differences in grain size distributions, use of the land and topographical variations. As a result, the larger the studied area is, the larger will the scatter be in different compilations of parameter data and the greater will the uncertainties be about the relevance for the specific area of interest.

The clays in the areas were deposited in salt seawater and have then been leached to varying extents. This has in some areas entailed that quick clay has been created in some parts. A low salt content does not automatically mean that the clay becomes quick, but that the salt content becomes lower than a certain limit is a prerequisite for this to happen (e.g. Söderblom 1969). Besides leaching there are a number of other factors that affect this process such as void ratio, oxidation, weathering, ion precipitation, ion composition, other chemical substances in the pore water etc. Leaching can occur both by percolation (water seepage through the soil) and diffusion (equalisation of chemical concentrations by ion migration). In both cases, this is facilitated if the thicknesses of the clay layers are small and if these are underlain by water transporting coarse layers. Quick clay was found in Section A in Torp within parts where the thickness of the clay layer was moderate but not where they were thick. High sensitivity-values were also found at the test point located furthest away from the river in Strandbacken, although no quick clay was found within the area now investigated. However, further up towards the valley sides where the thickness of the clay layers decreases, there are large areas with quick clay according to the previous investigations.

Leaching normally entails that the liquid limit of the clay is reduced. It can also mean that the structure of the soil becomes weaker and that both the preconsolidation pressure and the undrained shear strength thereby become reduced. Indications of this could be observed in both Torp and Strandbacken within those parts where the sensitivity was enhanced.

There can thus be considerable variations in soil properties due to chemical changes that have occurred during the time after the deposition also when the soil layers have the same history of deposition and stresses.

Modelling of the soil properties within an area is usually made with reference to depth below the ground surface or to the level. For areas with watercourses that have eroded down from an originally approximately flat ground surface, the method of presenting properties versus level gives the best model.

6.5 EFFECT OF UNLOADING ON THE SHEAR STRENGTH

That an unloading affects the undrained shear strength in such way that it decreases after the soil has been given time to swell and adapt to the new stress condition can be regarded as an established fact. This is also clearly illustrated by the results in Torp and Strandbacken, where erosion has brought very large unloading below the watercourses.

However, the measured reduction varied depending on what method was used for the measurement when the hitherto commonly used evaluation methods were applied. The reduction that was measured in more advanced laboratory tests, such as direct simple shear tests, corresponded fully to the reduction that is normally estimated from the soil models that have been elaborated by primarily advanced laboratory testing in triaxial and direct simple shear apparatuses. The reduction that was measured by field vane tests was considerably smaller. This is also in line with previous Swedish experience. The same was found for the results of CPT tests and dilatometer tests. However, the evaluation methods that have hitherto been used for field vane tests and CPT tests in Sweden are mainly based on experience from normally consolidated and only slightly overconsolidated soils. When using the revised evaluation methods that have been presented earlier in this chapter, the corresponding shear strength reductions are obtained as in the advanced laboratory tests. The same reduction was obtained with the revised method for dilatometer tests that has also been presented here.

The evaluation methods for the field tests are empirical and no theoretical framework for the response of the various methods and the necessary corrections in overconsolidated soils has been elaborated.

The excavations varied in size from fairly large, up to 9 metres deep, to fairly moderate, 1 to 2 metres deep. The excavation work was mainly performed within the upper delta and lateral fluvial sediments and, in most cases, such sediments or coarser silty clay constituted the upper soil layers after the excavation as well. For this reason and because of the test methods employed, the changes measured after the excavations were very moderate. However, the only available values from the conditions before the excavations to compare with had been obtained by field vane tests. Considering the experience of how the results of this type of test are affected by an unloading, there is nothing in these results to disprove that the expected reduction in shear strength really has occurred.

Experience from other types of excavations in clay has shown that it can take very long time, sometimes several decades, before the full shear strength reduction has evolved (e.g. Skempton 1964). In the studied cases, where free water has kept streaming out on the excavated terraces, the 10 to 15 years that have elapsed should be a sufficient time for all major changes to occur.

Another aspect of how unloading affects the shear strength is that the load reduction and the pore pressure equalisation entail that the effective stresses decrease in all of the unloaded soil volume. As a result, the drained shear strength becomes lower than the undrained shear strength and thereby governing in a successively larger part of the soil volume. This means that the reduction in available shear strength becomes larger than is the case for the undrained shear strength alone, and this often becomes the dominating factor.

6.6 STABILITY AFTER EXCAVATION OF THE SLOPE CREST

The calculated stability factors after the excavations in principle reach the values that are required according to the guidelines of the Swedish Commission on Slope Stability (1965) for the current use of the land. All potential slip surfaces which affect existing buildings have calculated safety factors of at least 1.3 and the calculated critical slip surfaces in what is classified as “other ground” have minimum safety factors of mainly 1.2 according to combined analyses. The only exception is the outermost strip in front of the Brosätter nursing home, where the calculations still show very low stability. This strip is fenced off and is normally not entered. There is also no obvious risk that a local slide in this riverbank will continue backwards within a short time after a such event. The requirements set by the guidelines for existing buildings and current land use can thereby be considered to be met.

The achieved result is not solely an effect of the excavations but for shallower slip surfaces the constructed erosion protections have also provided a considerable improvement of the calculated stability.

6.7 ACHIEVED INCREASE IN STABILITY IN RELATION TO EXPECTED RESULTS

The excavations had in all cases been designed to provide a certain minimum calculated safety factor for the remaining buildings according to undrained analyses. However, the ambitions differed between the different cases. In Strandbacken the aim was to reach a calculated safety factor of 1.5. In the Torp area, designs were made for two alternatives, one with 30% increase in calculated stability and one with a 50 % increase. The safety factor before the excavation was in this case assumed to be 1.0. The calculations without consideration for anisotropy showed lower safety factors, but in reality these cannot be lower than unity. The final design of the excavation and the calculated safety factors varied for different reasons, among them a desire to preserve the existing cement works and varying geometrical conditions along the roughly 600 metre long distance. The variation in calculated safety factor mainly remained within the two given alternatives. In Sundholmen, a lowest calculated safety factor of 1.3 for slip surfaces reaching the existing buildings was aimed for. However, certain compromises had to be made with consideration for the risk of flooding and the requirement that the excavated ground should be possible to use as gardens also after the excavation.

The demands thus set have in principle been fulfilled. The main discrepancies refer to the stability of the excavated areas and shallower slip surfaces at the outer slope from the excavated terraces down to the watercourses. The calculated safety factors are lower in these parts than was assumed in the designs. However, this mainly refers to calculated safety factors in combined analyses, for which lower requirements of their sizes are generally set. The general assessment of the stability of large and medium size slip surfaces is thereby not significantly affected.

However, the excavations performed have only entailed minor improvements in the stability of shallow slip surfaces in the outer slope from the excavated area down to the watercourses. The calculated safety factors in these parts are mainly governed by the drained shear strength. The lowering of the pore water pressures after the excavations is very limited, whereas the steep slopes remain after this operation and the increase in calculated safety factor is thereby very limited. In some cases, the stabilising works also entailed that the vegetation in the slopes had to be cut down, which resulted in stability problems in steeper parts. A number of superficial slides are thus reported to have occurred in the upper parts of the remaining steep outer slopes shortly after the excavation works, which among other things brought down some of the replanted trees. However, the erosion protection structures have normally entailed that also the outer slopes have become more level and in some cases they also constitute counterweights for the shallow slip surfaces. The exception is the riverbank in Sundholmen, which has been commented upon above. The combined effects mean that the calculation factors normally reach acceptable levels. However, as stated before, this is not primarily a result of the excavation, and the effects of the erosion protection structures were not considered in the designs.

Chapter 7.
Recommendations

Investigation methods

On the basis of the results of this project and previous experience it is recommended that a slope stability investigation should be started with a thorough study of existing geotechnical investigations and geological information and other observations that can be used for estimation of the layering and depths of the soil deposits in the area. This is prescribed in the guidelines of the Swedish Commission on Slope Stability (1995) and is here called ”Geotechnical inspection and rough estimations”. If the existing information is insufficient for an estimate of the thickness of the soil layers, a geophysical survey should be considered. The geophysical methods that are most likely to be used at this stage are refraction- or reflection- seismic surveys.

The subsequent geotechnical investigations can then be performed in a most rational way and it can be assured that the best-suited and adequate equipment is brought and used in the field. All soundings that are aimed at detailed mapping of the soil layering and measuring properties in the soft or medium-stiff soil layers that are of primary interest should be performed as CPT tests. The pore pressures should be measured during the tests and the most accurate equipment should preferably be used, i.e. Class CPT-3 according to the standard recommended in Sweden (SGF 1993). The tests should be continued to depths well below those that can be involved in a potential stability problem. This often entails that they should be continued to firm bottom. In extremely deep soil profiles, an initial pilot test should be continued to large depths to ascertain that no weaker or water-transporting layers, which can affect the stability, occur at larger depths. After this has been ensured, the subsequent penetration tests can be stopped at more moderate levels where the criteria given above are fulfilled.

In cases where there is a requirement to determine the thickness of underlying denser layers of coarse soil and till and the level of the bedrock surface by sounding tests, soil-rock drilling with multi-channel registration of drilling parameters should be used. This should preferably be performed as Jb-3 sounding according to the description of the method by SGF (1999).

After obtaining supplementary data from routine tests on samples in the laboratory, a preliminary estimation of both preconsolidation pressures and shear strengths can be made from CPT tests performed with high accuracy. This should be made according to the revised methods presented in this report. If the investigations are supplemented by dilatometer tests, these too should be evaluated according to the revised method presented here. However, in soils with significant organic contents, which are often found behind the crests of the slopes, the previous evaluation method should be used unless relevant factors for the new method have been determined for the particular soil in the laboratory.

The undrained shear strength is traditionally determined by field vane tests performed according to the standard recommended by SGF (1993). This should normally be done with the normal-size vane. The tests should be performed with equipment with casings for the vane and rods in soils that are susceptible to disturbance and at large depths, where the rod friction against the turning rods otherwise becomes too large. The extent of the required field vane testing depends, among other things, on the extent to which high-accuracy CPT tests have been performed. The field vane is so far the most well-tried method of determination of undrained shear strength in the field and can partly serve as a calibration of determinations by CPT tests. The hitherto commonly used evaluation method for the field vane test appears to yield too high values of the undrained shear strength in overconsolidated soils, for example, at the toes of slopes and below eroded watercourses. A reduction for overconsolidation should therefore be made when the overconsolidation ratio exceeds 1.3 according to the method for this presented in this report.

Supplementary direct simple shear tests should be performed whenever the accuracy of the results and/or the relevance for the test and evaluation method is uncertain and also when the scatter in the test results is large. The results of direct simple shear tests can be used for calibration of undrained shear strengths obtained from both field vane tests and CPT tests provided that the tests have been performed on samples of high quality. This is also stated in the guidelines of the Swedish Commission on Slope Stability and has been exemplified in this report.

The guidelines of the Commission also state that in cases where the shear strength anisotropy is used and is of significant importance for the assessment of the stability, these effects have to be verified by triaxial tests. This too has been exemplified in the report.

The stress history in the area should be clarified to enable a relevant modelling of the shear strength properties in the soil mass. This can be achieved by a sufficient number of oedometer tests on undisturbed soil samples supported by the results of the CPT tests.

Measuring of pore pressure variations in fine-grained soils should only be done with closed systems. Problems with evolution of gas may occur with time and equipment to remedy this problem should be at hand. The pore pressures in the outer part of a slope, i.e. close to the crest and below the steep slope below this, are often of great importance. At the same time they are normally most difficult to model properly. Several systems for pore pressure measurements should therefore be placed in these parts.

Calculation methods

The pore pressure conditions in a slope can be modelled fairly accurately with existing calculation programs. This is provided that the soil layering and its variation within the area are not too complicated. The modelling can mainly be done for the pore pressure conditions at large, whereas important details such as the exact pore pressure distribution in the outer parts of the slope are more difficult to calculate. The effect of precipitation and evaporation is also difficult to calculate accurately. With these reservations, it should be possible to make an adequate prognosis beforehand of how the pore pressure conditions will change after a change in the geometry of the slope, such as an excavation of the crest.

A relatively good prognosis for the changes in shear strength after an excavation can be made on the basis of established soil models and empirical parameters. The general equation $c_u = a \cdot \sigma'_0 \cdot OCR^b$ is then used. The b-factor is then normally set to 0.8 and the a-factor can be calculated from measured undrained shear strength, effective overburden pressure and overconsolidation ratio before the excavation. A more refined analysis can be made if these parameters are calibrated for the particular soil by direct simple shear tests in the laboratory. Stability calculation in connection with slope stability analyses and stabilising measures should be made with "rigorous" analyses. It shall be possible to model in detail the shear strength of the soil both concerning drained and undrained shear strength, the shear strength anisotropy and the pore pressure variations in the soil mass, and combined analyses which take all these factors into account should be performed. If these requirements are not fulfilled, there is risk that the necessary stabilising measures become both under- and overestimated and that their design will not be suitable in every respect. Different existing calculation programs can handle different parts of these demands, but there is so far no really good "self-seeking" program, which can handle all these aspects in a satisfactory way without compromises or manipulations. One way to overcome this, is to use two programs, one of which is self-seeking, as has been done in this project.

Design of excavations for increase of slope stability

More aspects than only the calculated factor of safety in undrained analyses have to be considered in the design of an excavation. From an overall stability point of view, the stability according to combined analyses, which also include drained analyses, also has to be considered. This often has the result that the remaining slope from the excavated area down to the toe of the slope also has to be changed in some way.

The stabilising measures in all slopes towards running watercourses should be combined with construction of permanent erosion protection, unless this is already in place. The construction of a substantial erosion protection structure normally entails that the average inclination of the lower slope is decreased, which improves the stability. However, the stability for shallow slips in the upper part of the remaining slope is not necessarily affected by this measure. There is therefore often cause for a rounding-off of the crest of the slope. This should also be considered from an aesthetic point of view.

The design of the rear excavated slope, or slopes if the excavation is made in stages, should also be considered, particularly when the excavation is performed in silt and fine sand. Slopes that are given normal fairly steep inclinations are often not stable in these types of soil because of high pore pressures and they are often also subject to internal erosion by the water seeping out through them. This can necessitate further remedies, such as the construction of erosion protection already during execution of the works, or it can entail a degradation of the slope with time. In the latter case, cavities and small slips will occur with attendant risks for those who live in or visit the area, and there is also a risk that the slope with time will retrogress towards gardens, buildings or other constructions behind it. There are also aesthetic aspects concerning the design of the rear excavation slope, where a more rounded geometry is often considered to be more natural and appealing.

The excavated areas should always be given sufficient inclinations to ensure adequate run-off of surface water. This requirement applies both when the area is intended for recreation and when natural or planted vegetation is to be re-established. In many cases deeper drainage systems should also be installed, which will lead away any water flowing in from higher elevated ground at the rear of the excavated area and which will permanently lower the groundwater level below the excavated area and its outer crest.

Chapter 8.

Need for further research and development

The investigations in this project have been performed in slopes with somewhat different types of clay and organic soils ranging from low-plastic clay through high-plastic clay, organic clay to gyttja. However, they have been limited to the same type of stabilising measure at, in principle, the same type of layering and stress history in the soil profiles. The main differences between the different slopes have been the heights of the slopes and the sizes of the excavations. Similar measures have been used in other types of soils, such as more overconsolidated clays and clay deposits without overlying delta and lateral fluvial sediments. There may thus be further aspects of the method and the design of excavations than those put forward in this report. Furthermore, even if the investigated method is one of the most commonly used ones, it is only one of many whose long-term functions have not been investigated. There is thus a need for a continuation with studies of the effects of other methods and in other types of soil.

The results of the investigations, together with previous experience, have raised serious questions about the previous methods of evaluation of undrained shear strength in overconsolidated soils in general and in particular in slopes. This refers to all commonly used field methods for this determination. Revised methods for evaluation of the shear strength from field vane tests, CPT tests and dilatometer tests have been proposed in this report. However, these methods are based solely on empirical experience and so far no theoretical basis has been elaborated for them. Research is currently going on in co-operation between the Swedish Rescue Services Agency, Chalmers University of Technology and SGI regarding how the stress conditions in the soil affect what is measured in field vane tests and CPT tests and how this affects the interpretation (Löfroth 2002).

The field vane test is also used in other types of overconsolidated soil such as clay till, both to estimate the undrained shear strength and to estimate the preconsolidation pressure. It is also sometimes used for calibration of other methods as CPT tests (e.g. Foged and Steenfelt 1992, Kammer Mortensen et al. 1991). It is thereby normally not considered that the overconsolidation affects the results of different tests in different ways. The results of this investigation and what has been reported elsewhere about results of field vane tests in overconsolidated soil (e.g. Leroueil et al. 1983, Jamiolkowski et al. 1985, Aas et al. 1986), raise questions regarding this practice. A review of the experience of all different field methods to determine the properties in all types of overconsolidated fine-grained soils is therefore required.

The current methods of prognostication of pore pressure variation would also need to be improved. More series of continuous measurements with closed systems of the pore pressure variations in soil masses are required. This also applies to how the pore pressures vary in rather short time perspectives with climatic changes in terms of temperature, wind, precipitation, humidity, water levels in adjacent watercourses and their duration. Better knowledge is also required about how the pore pressures vary in the outer parts of slopes. Better guidelines are thereafter required for how the pore pressures in slopes should be modelled when using different calculation programs.

Finally, new or further developed "self-seeking" calculation programs are needed, which in all repects can take into account all important factors and variations that should be consideared according to the guidelines given by the Swedish Commission on Slope Stability (1995).

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