



STATENS GEOTEKNISKA INSTITUT  
SWEDISH GEOTECHNICAL INSTITUTE

# Settlements and shear strength increase below embankments

– long-term observations and measurement of shear  
strength increase by seismic cross-hole tomography

ROLF LARSSON  
HÅKAN MATTSSON

Report 63

LINKÖPING 2003





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Report **No 63**

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tomography

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***Rapport/Report***

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# Preface

This report deals with the long-term settlements of embankments and the accompanying increase in shear strength. The long-term settlements of embankments are important for the required maintenance during their service life and the accompanying increase in shear strength can be utilised both in the construction of the embankment and if it is to be raised or widened at a later stage and/or if the traffic load is to be increased.

Methods for prediction of long-term settlements are being developed all the time. This work requires that reference data are available in terms of long-term settlement records and other observations of constructions in areas with thoroughly investigated soil conditions. Two such areas are the geotechnical test fields at Lilla Mellösa and Skå-Edeby, where a number of test embankments were constructed 40 – 55 years ago. The test fields have been supervised by SGI since then and the long-term behaviour has been monitored regularly. A thorough investigation of the current state of the soil below the three embankments that were constructed without any kind of soil improvement or artificial drainage has been made in this project.

If the shear strength increase below an embankment is to be utilised in construction or to allow increasing loads or speeds, it has to be verified. In this project, a new method of assessing the increase in undrained shear strength below embankments using seismic cross-hole tomography has been tested. This method has the advantage of not requiring access to the embankment itself and provides a continuous mapping of the properties in the entire soil volume in the investigated section, not only in selected test points.

The report is intended for geotechnical engineers who deal with the construction and maintenance of embankments, who design and carry out field investigations and who develop methods for prediction of consolidation processes resulting in significant settlements and shear strength increases.

The part of the project concerning testing of the method of seismic cross-hole tomography for assessment of shear strength increase has been performed in co-operation between the Swedish Rail Administration, GeoVista AB and SGI, and the detailed investigation of the current state of the properties below the embankments has been financed by internal funds at SGI.

The authors wish to express their gratitude to colleagues at the Royal Institute of Technology for kind assistance with equipment and to Kjell Hidsjö at FM-Geo AB for well performed field work.

Linköping and Luleå, May 2003

The Authors

# Contents

## **Preface**

<b>Summary</b> .....	7
<b>Notations and symbols</b> .....	9
<b>1. Purpose and scope of the project</b> .....	10
<b>2. Introduction</b> .....	13
2.1 Settlements and shear strength increase .....	13
2.2 The test embankments .....	14
2.3 Verification of shear strength increase .....	15
<b>3. The test fields</b> .....	16
3.1 Lilla Mellösa, Upplands Väsby .....	16
3.1.1 History and soil conditions .....	16
3.1.2 Construction and instrumentation of the undrained test fill .....	21
3.1.3 Measured settlements, settlement distributions, excess pore pressures and changes in shear strength, preconsolidation pressure and other properties .....	21
3.2 Skå-Edeby .....	28
3.2.1 History and soil conditions .....	28
3.2.2 Construction and instrumentation of the undrained circular test fill .....	36
3.2.3 Measured settlements, settlement distributions, excess pore pressures and changes in shear strength, preconsolidation pressure and other properties .....	36

3.3	The test embankment .....	46
3.3.1	Construction and instrumentation of the test embankment .....	46
3.3.2	Soil conditions .....	48
3.3.3	Measured settlements, settlement distributions, excess pore pressures and changes in shear strength, preconsolidation pressure and other properties .....	48
<b>4.</b>	<b>Measurements with cross-hole tomography .....</b>	<b>60</b>
4.1	Background and hypothesis .....	60
4.2	The method .....	61
	Seismic waves .....	61
	Seismic tomography .....	62
	Software for tomography inversion .....	65
	Field equipment and procedure .....	65
4.3	Results .....	66
	Data analysis .....	66
	Tomography inversion .....	70
	Skå-Edeby .....	70
	Lilla Mellösa .....	72
4.4	Discussion of the results .....	73
<b>5.</b>	<b>Correlations between the evaluated results and measured increase in shear strength .....</b>	<b>75</b>
<b>6.</b>	<b>Demand for development .....</b>	<b>84</b>
<b>References</b>	.....	<b>85</b>

# Summary

This report contains updated records of the measurements performed below three test embankments constructed on soft soil between 40 and 55 years ago. These embankments are of particular interest since significant consolidation settlements are still in progress and significant excess pore pressures still remain below them. Since they have been constructed on natural soil without any soil improvement or installation of vertical drains, they have also been considered as best suited for the pilot tests concerning the use of seismic cross-hole tomography for estimation of shear strength below old embankments.

The report starts with a review of all measurements and observations that have been done since construction up to the present day. All data that are considered to be relevant for a description of the consolidation processes are included. These data include settlements, settlement distributions with depth and across the loaded areas, excess pore pressures, changes in water content, preconsolidation pressure, permeability and undrained shear strength, and in one case also horizontal movements in the soil.

No new settlement calculations or other predictions of the consolidation processes are presented herein. This was done in SGI Report No. 29 (Larsson 1986) and all later measurements are close to the courses that were predicted at that time. Detailed information about the compression characteristics of the soils can also be found in that report.

The method of seismic cross-hole tomography has been tried to estimate the increase in shear strength due to consolidation below the embankments. This study has been a pilot project in order to assess if the method can be used in practical applications and to what accuracy the shear strength increase can be estimated.

The measurements have been made with readily available commercial equipment and the evaluation of the shear wave arrival has been made with a built-in function in the recording instrument. The further evaluations have been made with existing

methods and programs as they are without any attempts of development with consideration to the new type of application.

The evaluation in terms of distribution of shear wave velocity is straightforward and the translation to distribution of undrained shear strength is fairly simple. However, certain aspects have to be observed and a procedure for this is presented in the report.

The results have provided good overall views of the shear strength increases and their distributions and the estimated values are also of the right sizes. They can also be considered reliable enough to be used in design if normal caution is taken for the spread in the results of shear strength estimations.

Since the application is new, there is no equipment that is designed for this particular purpose. Owing mainly to the source of the seismic waves, the installation of the measuring equipment was relatively laborious, costly and time demanding. The tests were also performed in homogeneous soil profiles in which the picking of arrival times was straightforward and did not require any filtering of the signals. The geometries of the measured sections were also favourable for this type of measurement.

The evaluation program employed is well tried and can be obtained free of charge from the Internet. It worked well in general, but certain anomalies could not be avoided in parts of the profiles with poor ray coverage being located close to borders between materials with very different stiffnesses.

Although the method has been shown to be useful, there is thus room for further development to make it a rational tool for routine investigations of the shear strength below old embankments.

# Notations and Symbols

$c_u$	Undrained shear strength
$CPT$	Cone penetration test
$CRS$	Constant rate of strain
$f$	Frequency
$G_0$	Initial shear modulus at small strains
$H$	Inclinometer tube
$P$	Piezometer
$S$	Settlement marker
$S_t$	Sensitivity
$SGI$	Swedish Geotechnical Institute
$V_s$	Shear wave velocity
$w_L$	Liquid limit
$w_N$	Water content
$w_P$	Plastic limit
$\rho$	Density
$\sigma'_0$	In situ vertical stress
$\sigma'_c$	Preconsolidation pressure
$\Delta\sigma$	Increase in vertical stress

# Chapter 1.

## Purpose and scope of the project

The purpose of the project was twofold. The first purpose was to test the method of seismic cross-hole tomography and investigate if and to what accuracy it can be used to determine the shear strength below existing embankments. The origin of this part of the project is the current upgrading of the Swedish railway lines for faster and heavier trains. In this context it will be of great benefit if the shear strength below old embankments on soft soils, which were constructed up to a hundred years ago, can be utilised. However, this requires that the shear strength increase in relation to the soil outside the embankment can be verified. Traditional geotechnical investigations require access to the ground at the investigation point and, since no interruption in the railway traffic can normally be accepted, these investigations are confined to the area outside the embankment. On the other hand, previous investigations have shown that there is a clear relation between the undrained shear strength in the soil and the shear wave velocity (e.g. Larsson and Mulabdic 1991). Measurement of shear wave velocity below an embankment of limited width can be made from one side of the embankment to the other by the cross-hole method. Consequently, the measurements do not require access to the embankment itself. However, the shear wave velocities in a soil mass below an embankment and in the embankment itself vary, and to estimate the shear strength variation in the soil, the more elaborate method of seismic cross-hole tomography has to be used. Equipment and evaluation software for this method is already at hand. The purpose of this part of the project was therefore to investigate whether the shear strength below existing embankments can be estimated accurately enough to provide verification of any increased shear strength and to be used in stability assessments.

The second purpose was to obtain a new detailed check of the ongoing consolidation processes below three old test embankments constructed on natural ground without any vertical drains. The ordinary monitoring of the embankments is today limited to regular levelling of various settlement markers. During the first years after construction, the monitoring of settlements and pore pressures, and in one case horizontal displacements was very frequent and a number of sampling operations were performed to check changes in water content and density as well. However,

with time the pore pressure gauges and the inclinometer tubes ceased to function and the rate of settlement and other changes decreased. The readings thereby became fewer and their spacing in time increased. More thorough investigations with measurement of pore pressure, undrained shear strength and other soil properties have then been performed in a few separate research projects. However, the latest such project in any of the embankments was performed 20 years ago. In connection with the first part of this project, the shear strength increase below the embankments under which the seismic cross-hole tomography was performed had to be investigated also by ordinary geotechnical test methods. It was then decided to make a full investigation to check the current state of the soil below these embankments also in terms of remaining excess pore pressures, changes in water contents and densities, developed preconsolidation pressures and any other significant parameters. In this way, the determinations of the shear strength increase in the field could be supplemented by laboratory tests too. However, the main purpose of this part of the investigation was to provide new relevant data to check existing methods of prediction of settlements and shear strength below embankments and to provide extended references for new ones.

The investigations have comprised seismic cross-hole tomography below the three test embankments without vertical drains in Lilla Mellösa and Skå-Edeby and in the natural ground outside the embankments. The embankments are different in shape; one is circular, one is square and one is long and narrow like an ordinary embankment. Their central parts are instrumented with various iron rods and pipes penetrating the soil mass and, even if they are coated in order not to interact with the soil mass, they would most probably influence seismic measurements. The latter have therefore been performed in parts below the embankments well clear of any possible influence of these objects. The measurements have been performed with equipment that was readily available. The only such vibration source was a screw plate by which vertically polarised shear waves can be created. The receiving geophones were lowered in special plastic tubes and fixed at predetermined depths. It was apprehended that even slender straight-walled plastic tubes would be too stiff in their axial (i.e. vertical) direction to enable a proper measurement of the arrival of vertically polarised shear waves to the different levels, and therefore axially flexible bellows hoses were used. The bellows hoses were installed about one month before the actual measurements to let them become firmly fixed in the soil.

Ordinary soil investigations have been performed below the central parts of the embankments, below the outer parts of the embankments or directly outside and in natural soil well outside the embankments. The natural soil outside the embankments has been thoroughly investigated before and here only supplementary CPT tests

were performed. At the other points field vane tests, CPT tests and undisturbed sampling were performed at every metre depth. Ten pore pressure tips were also installed at equally spaced depths below the central parts of each embankment. These were monitored continuously until it could be verified that they had stabilised and were then withdrawn. The samples have been investigated concerning classification and routine testing of density, water content, liquid limit, fall-cone shear strength and sensitivity. CRS-oedometer tests have also been performed on all samples from all sampling points and levels to determine preconsolidation pressure and permeability. Furthermore, a number of direct simple shear tests have been performed to supplement the field vane tests and the CPT tests and a few triaxial tests have been performed to check the validity of empirical relations between the “active” shear strength and the preconsolidation pressure.

# Chapter 2.

## Introduction

### **2.1 SETTLEMENTS AND SHEAR STRENGTH INCREASE**

Construction of embankments on soft fine-grained soil deposits often entails both large settlements due to consolidation of the soil and stability problems due to insufficient shear strength of the soil in its natural state. However, when the soil consolidates the shear strength increases too. Much of the consolidation occurs fairly rapidly after the load application if the consolidation process is speeded up by use of vertical drains or facilitated by natural short drainage paths to coarse permeable layers. On the other hand, the process takes a considerable time in thick homogeneous clays layers. Also in a more long-term perspective, there is usually a slow but steady growth in shear strength with time because of creep deformations and load increases when the level of the embankment is adjusted to compensate for the settlements.

It has been shown that there is a direct relation between the preconsolidation pressure in the soil and the undrained shear strength (e.g. Ladd and Foott 1974). This relation is valid independent of whether the preconsolidation pressure has been created directly by an acting effective stress or as an indirect quasi preconsolidation pressure by creep effects. All types of consolidation that mean that the preconsolidation pressure increases thereby also mean that the undrained shear strength increases. An increase in the effective stress due to consolidation in overconsolidated soils also means that the undrained shear strength increases even if the preconsolidation pressure is not exceeded. This increase, however, is relatively small.

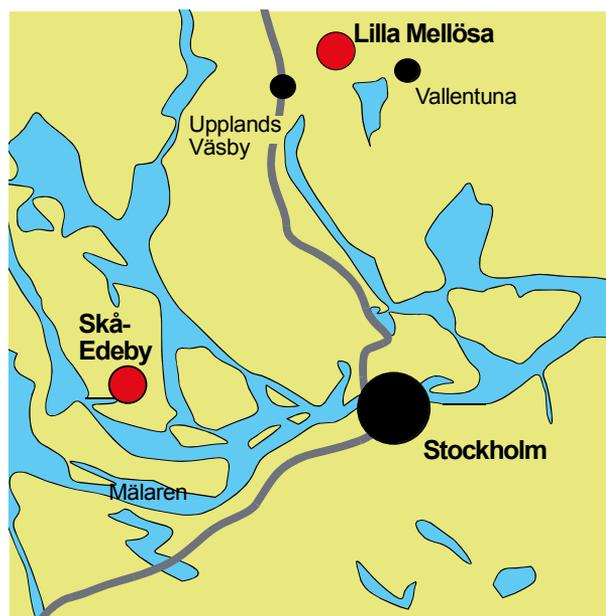
The increase in shear strength in soft fine-grained soils due to the consolidation under applied loads is often utilised in stage-construction, whereby the load is applied in steps with sufficient time intervals in between to let the soil consolidate for each applied load. In this way, embankments can often be constructed without supporting structures or soil improvement which would be required if the embankment was constructed in a single stage. However, installation of vertical

drains is often required to speed up the process. The increase in shear strength can also be used when old embankments are to be raised and widened, when the traffic load is to be increased, when the traffic speed is to be increased and at any combination of these factors.

## 2.2 THE TEST EMBANKMENTS

Calculation methods that take into account both so-called primary consolidation and creep effects have to be used in order to accurately predict the consolidation process in terms of both settlements and shear strength increase. Such methods, in which the compressive parameters of the soil are functions of time (or rather strain rate), have been successively developed for the last about fifty years. In order to calibrate these methods, a number of well-documented reference objects have to be available with full-scale constructions in the field which have been monitored in detail over a long time. Such reference objects are available at the test fields at Lilla Mellösa and Skå-Edeby, Fig. 1, where altogether eight test embankments have been constructed on soft clay. Most of the embankments were constructed 40–55 years ago and vertical drains of different types and spacing were installed below five of them. The remaining three embankments were constructed directly on the natural ground and for these the consolidation process still continues with excess pore pressures created by the applied load still remaining. The results from these test fields are fairly unique and they have been used for calibration of calculation methods all over the world. In fact, the latest results obtained in this investigation

were used in one thesis concerning calculation of settlements even before they were published (Claesson 2003).



*Fig.1  
Location of the test fields at  
Lilla Mellösa and Skå-Edeby.*

### **2.3 VERIFICATION OF SHEAR STRENGTH INCREASE**

If the shear strength increase below an embankment is to be utilised in construction or to allow increasing loads or speeds, it has to be verified. In stage construction, this is normally done by a control programme with traditional geotechnical investigations of the soil below the construction. Special inspection wells are then installed through the embankment during construction, and the tests can be performed from the bottom of these. Verification of shear strength increase below old road and railway embankments that are in use constitutes a more difficult task. Normal geotechnical investigations require that holes be taken through the embankment, which is laborious and often entails that part of the soil below is disturbed. They also require that the traffic be interrupted during the time for the field tests and sampling operations. Particularly for heavily trafficked railway embankments, this can seldom be accepted.

In recent years, various geophysical non-intrusive methods of soil investigations have been tried and implemented in geotechnical investigations. Some of these methods, such as seismic refraction and reflection investigations and measurements of shear wave and compression wave velocities, are well-established whereas methods such as resistivity measurements and surface wave analyses are still in a trial state. In this project, a new method of assessing the increase in undrained shear strength below embankments using seismic cross-hole tomography has been tested. This method has the advantage of not requiring access to the embankment itself and also provides a continuous mapping of the properties in the entire soil volume in the investigated section, not only in selected test points. The method itself is not new but has previously among other things been used for location of buried objects in the ground. However, the application of trying to map the shear strength in a soil mass has to the authors' knowledge not been reported before.

# Chapter 3.

## The test fields

### 3.1 LILLA MELLÖSA, UPPLANDS VÄSBY

#### 3.1.1 History and soil conditions

##### *History*

The test fills at the farm of Lilla Mellösa near Upplands Väsby were constructed by SGI shortly after the institute was founded. They were constructed in connection with the search for a suitable site for a new airfield outside Stockholm. At Lilla Mellösa there was a large, almost flat, area with only scattered farms. The subsoil conditions, however, were less ideal with ten to fifteen metres of soft highly compressible soils.

Three test fills were constructed in 1945 – 1947, first one with vertical drains installed below it and one on natural ground and later a low fill consisting of the removed surcharge material from the drained fill. Only the first fill without drains will be considered in this report, Fig. 2. The fills were instrumented and measurements were made periodically, but after new test fills had been constructed at Skå-Edeby with improved vertical drains and instrumentation, the interest in the fills at Lilla Mellösa waned for some time.

In 1966, the test field at Lilla Mellösa came into focus again as Chang (1969) compiled the available earlier data and started new investigations regarding soil compressibility, pore pressures and settlements using newer and more accurate equipment. This investigation was completed ten years later when Chang (1981) was invited as a guest researcher to SGI. The report from 1969 was then updated with the accumulated measurements, new pore pressure measurements and further laboratory investigations. From these investigations, Chang concluded that the observations in terms of settlements, decrease in water content, pore pressures and undrained shear strength were incompatible with each other on the basis of then current concepts of the process of consolidation.



*Fig. 2. The undrained test fill at Lilla Mellösa in 2002.*

The investigations continued with new sampling techniques and oedometer test methods to accurately determine the compressibility and permeability characteristics of the soil, and the results from the test field were used in the development of a new calculation method of consolidation below embankments including creep effects (Larsson 1986).

The measurements at Mellösa have continued, even if much of the original instrumentation has ceased to function and the regular measurements are therefore confined to total settlements and those settlement markers at various depths that still appear to function.

The investigation in this project was performed in May – June 2002 and comprised levelling of all markers, sampling, field vane tests and CPT tests at the outer crest of the fill and at its central part and a CPT test well outside the area influenced by the fill. The pore pressures were measured in a series of piezometers that were monitored continuously for more than a week in order to verify that stabilised pore pressures were read off. The piezometers were then left in place for two more months before a final reading was taken and the systems were retracted, Fig. 3. The



*Fig. 3. Decommissioning of the pore pressure systems in August 2002.*

system for pore pressure measurements was selected to obtain a check of a new automatic data acquisition system for long term observations in the field. The atmospheric pressure was monitored along with the pore pressure readings and the measured values were corrected accordingly. When estimating small excess pore pressures, there are many factors that are important, such as the accuracy of the measuring system, the atmospheric pressure, any zero drift in the transducers, the exact levels of the filter tips and the reference pore pressures in the natural ground. Taking all these factors into account, the accuracy of the measurements is estimated to be within  $\pm 2$  kPa.

The test field has also been used for a number of other investigations concerning properties of soft clays. It was thus one of the sites where the new field vane apparatus was tried out (Cadling and Odenstad 1950). Later research concerning the vane shear test (Wiesel 1975), undrained shear strength and creep (Larsson 1977), quality of undisturbed samples (Larsson 1981), permeability of clay (Carlsten and Eskilsson 1984), dilatometer tests (Larsson and Eskilsson 1988), shear wave velocity and seismic cone tests (Larsson and Mulabdic 1990), CPT tests in clay (Larsson and Mulabdic 1991) and properties of lime and cement columns (Åhnberg et al. 1995) has been carried out at the Lilla Mellösa test field. Samples from Lilla Mellösa were also included in e.g. a study on the permeability of natural clays performed at Laval University (Tavenas et al. 1983).

## *Soil conditions*

A generalised soil profile in natural ground at Lilla Mellösa is shown in [Fig. 4](#).

At the top, there is a layer of about 0.3 metres of organic topsoil which was scraped off at the areas for the fills before they were constructed. The dry crust is unusually thin and consists of organic soil. The desiccated dry crust is limited to 0.5 metres and is underlain with soft clay. The clay has an organic content of about 5 % just under the crust which decreases with depth and is less than 2 % from 6 – 7 metres' depth and downwards. The colour changes from green to black and becomes grey with depth. The black colour is the result of presence of iron-sulphides which between 2.5 and 6.5 metres depth amount to about 0.5 % of the dry weight of the soil. The natural water content is about equal to the liquid limit and decreases from a maximum of about 130 % to about 70 % in the bottom layers. The bulk density increases from about 1.3 t/m<sup>3</sup> to about 1.8 t/m<sup>3</sup> at the bottom. The undrained shear strength has a minimum of about 8 kPa at 3 metres' depth and increases thereafter with depth.

The shear strength values in the natural ground were determined by field vane tests in 1964 and 1967 (the present test method was not developed until long after the construction of the embankment). The measured values have been corrected according to the present SGI recommendation (Larsson et al. 1984).

The clay becomes varved below 10 metres' depth. The varves are first diffuse but become more and more pronounced with depth. At 14 metres' depth, there is a thin layer of sand on top of the bedrock.

The pore water pressure in the ground outside the fill is hydrostatic with a ground water level about 0.8 metres below the ground surface. Very little seasonal variation has been measured throughout the years.

The soil in the upper two metres is overconsolidated due to dry crust effects. The rest of the soil profile is only slightly overconsolidated with an overconsolidation ratio of about 1.15. Further details of the compressibility characteristics are given in SGI Report No. 29 (Larsson 1986).

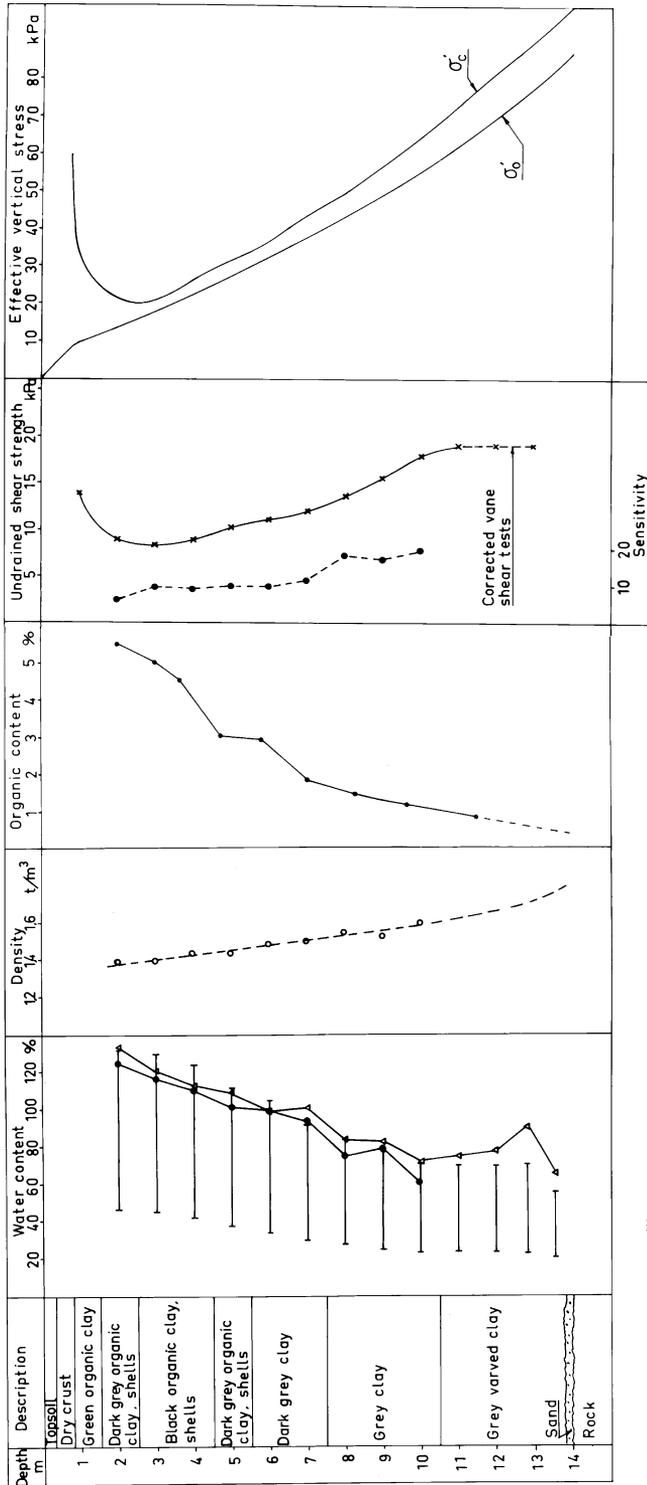


Fig. 4. Soil profile at Lilla Mellösa.

### ***3.1.2 Construction and instrumentation of the undrained test fill***

Before the fill was placed, the 0.3 metres of loose organic topsoil were removed. A 2.5 metre high fill of gravel with a density of  $1.7 \text{ t/m}^3$  was then constructed. The fill had bottom dimensions of 30 x 30 metres and slopes of 1:1.5. Time for construction was 25 days and no change in load other than natural variations has been made after that. The net increase in vertical stress was calculated to 40.6 kPa.

A number of settlement markers and piezometers were installed at various depths before the fill was placed. The piezometers readings proved to be unreliable. Also the original settlement markers installed at various depths ceased to function with time and were replaced with newer models, which in turn have in some cases ceased to function with time. The settlement distribution with depth has therefore been checked by settlement distributions calculated from the measured changes in water content. This has been done for the changes estimated in 1967, 22 years after construction, and 2002, 57 years after construction.

New piezometers were installed were installed in 1968. Also these ceased to function after a certain period of time and the pore pressures have then been measured by retractable piezometers in 1979 and 2002.

### ***3.1.3 Measured settlements, settlement distributions, excess pore pressures and changes in shear strength, preconsolidation pressure and other properties***

The initial settlement during the time of construction amounted to 0.065 metres. The computed “final” settlement when creep effects were disregarded was just below 1.4 metres. This amount of settlement was reached in 1966, [Fig. 5](#). At that time, there were still remaining excess pore pressures in the order of 30 kPa, indicating that almost no increase in effective stress had occurred in large parts of the profile, [Fig. 6](#). In 1979, the total settlement was 1.65 metres and the remaining excess pore pressures were over 20 kPa. The settlements have then continued and were just over 2.0 metres at the investigation in 2002. The maximum remaining excess pore pressure at that time is estimated at 12 kPa. The scatter in the latter readings reflects the inaccuracy of the estimations. The settlements continue, but at a continuously decreasing rate. The current rate of settlement is about 10 mm/year, [Fig. 7](#).

The measured and calculated settlement distributions indicate that the consolidation during the first years after construction mainly occurred in the uppermost layers and in the layer close to the draining bottom layer, [Figs 5 and 8](#). Due to the large

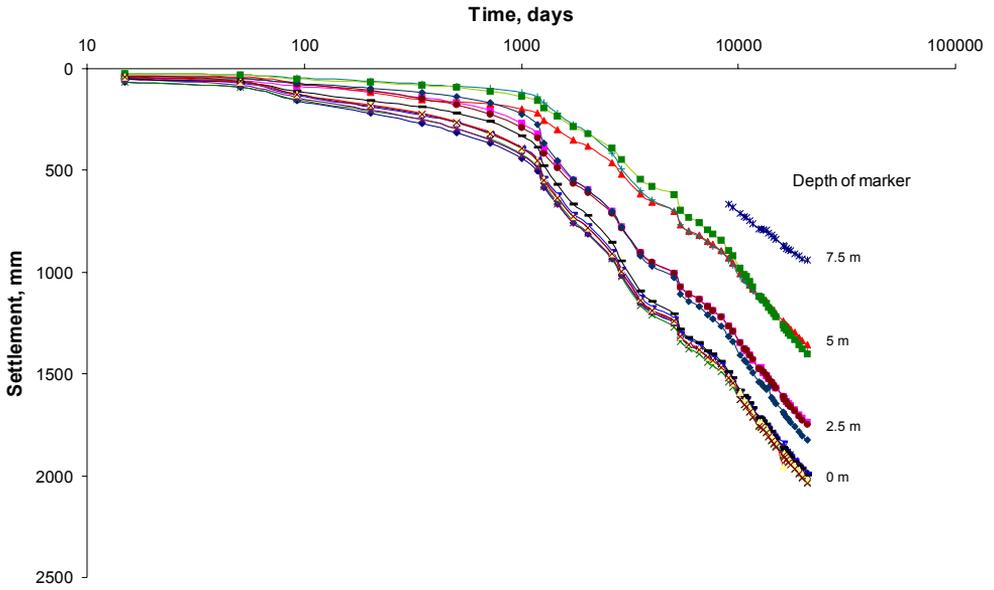


Fig. 5. Measured settlements below the undrained test fill at Lilla Mellösa, time in logarithmic scale.

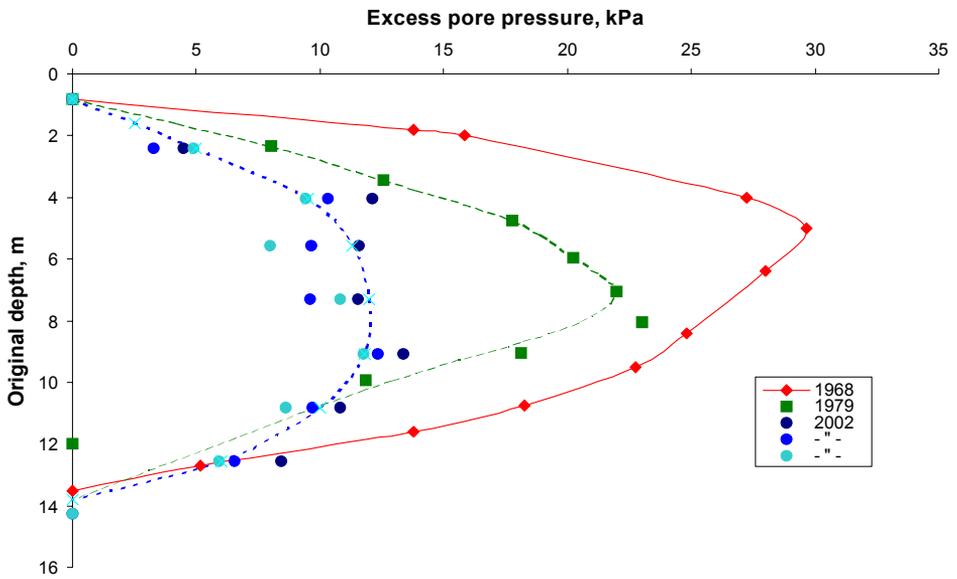
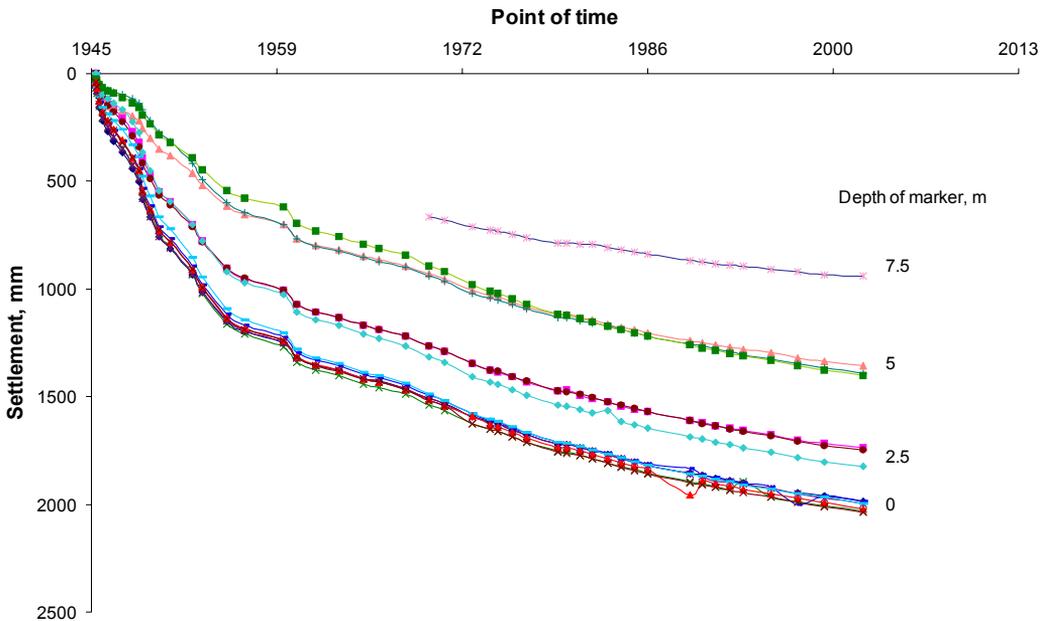


Fig. 6. Excess pore pressures below the undrained test fill at Lilla Mellösa at various times.



**Fig. 7.** *Measured settlements below the undrained test fill at Lilla Mellösa, time in linear scale.*

compressions in the top layers, the soil here has become overconsolidated and because of the reduction in effective stress when large parts of the crust and fill become submerged below the groundwater table, the upper layers have for some time been in a state of unloading. The compression of the upper 5 metres or so has therefore ceased and the more recent settlements are all related to compression of the central and lower layers in the profile. This is also reflected in the shear strengths measured by field vane tests, Fig. 9. The results scatter somewhat but they still clearly show that the shear strength during the first period increased only in the upper part of the profile and possibly somewhat in the lowest part and that the increase in shear strength that has occurred during the last period of time is solely located in the central parts of the profile.

The settlements and shear strength increases vary across the embankment. However, the settlements are fairly uniform below the central parts of the fill, and it is only at the outer 2 metres or so of the once flat upper surface that they are noticeably smaller. The maximum difference in settlement below the central part and the crest of the slope is just below 0.4 metres. The settlements at the crest are thus about 1.6 metres compared to 2.0 at the centre or 80 % of the maximum settlements. The increase in shear strength should thereby be expected to be somewhat lower just

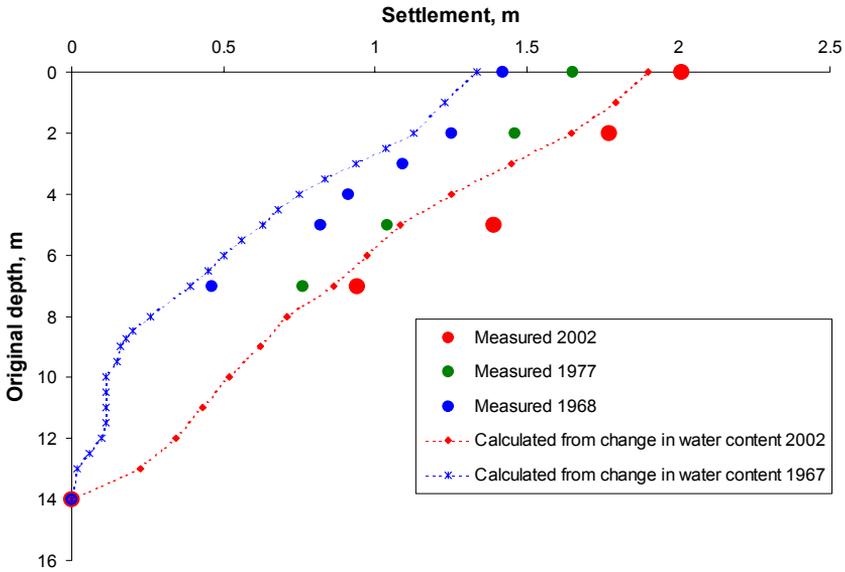


Fig. 8. Settlement distribution with depth below the undrained test fill at Lilla Mellösa at various times.

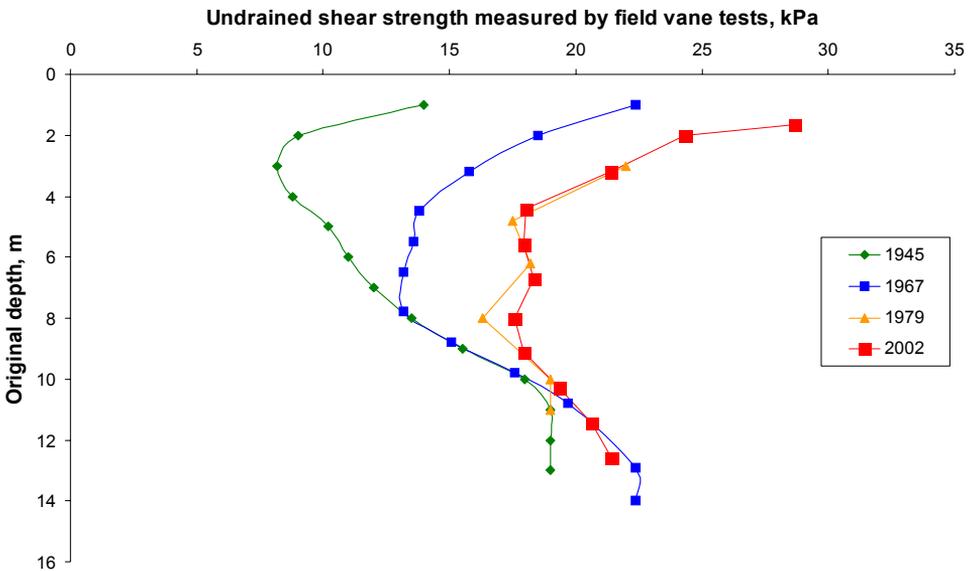


Fig. 9. Measured undrained shear strength below the undrained test fill at Lilla Mellösa at various times.

below the crest, which can also be observed in the results from the field vane tests, Fig. 10. Approximately the same relation can be observed in the results from the CPT tests. However, the evaluation of these tests involves the effect of the overburden pressure, which is more difficult to estimate at the border of the loaded area, and thereby becomes more uncertain.

The new investigations in the laboratory showed that the liquid limits were approximately the same as before the loading whereas the water contents had decreased significantly throughout the profile, Fig. 11. There was no significant difference between the samples taken at the centre of the fill and those taken at the crest.

The pre-drilling through the 2.5 metre thick embankment of gravel appears to have disturbed the soil below, particularly at the crest. This may have resulted in some too low values of the determined preconsolidation pressures. Nevertheless, it is obvious that the preconsolidation pressures have increased and in the upper layers even more than the maximum increase in effective stress, Fig. 12. It also appears that the increase is similar below the crest and below the central part of the fill except for the uppermost layers, where it appears to be lower below the crest. It can

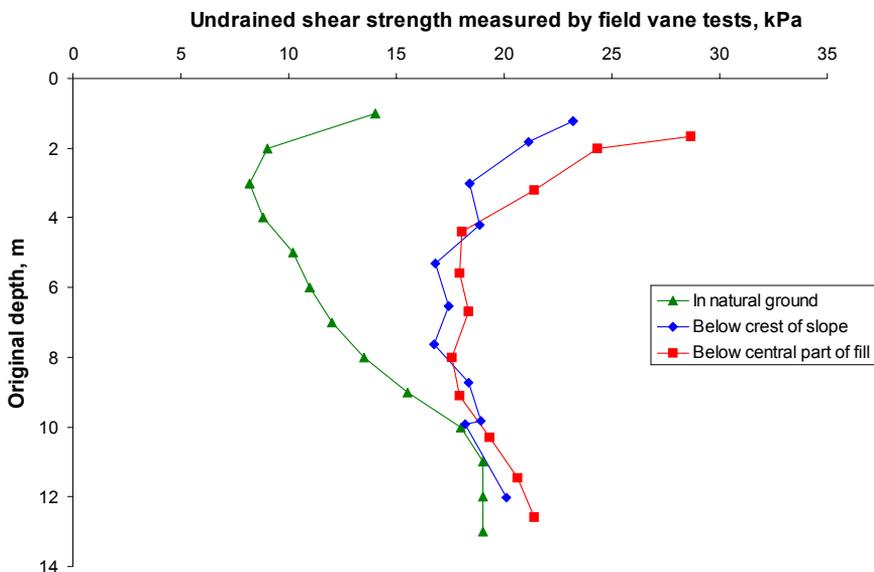


Fig 10. Distribution of undrained shear strength below different parts of the area measured by field vane tests.

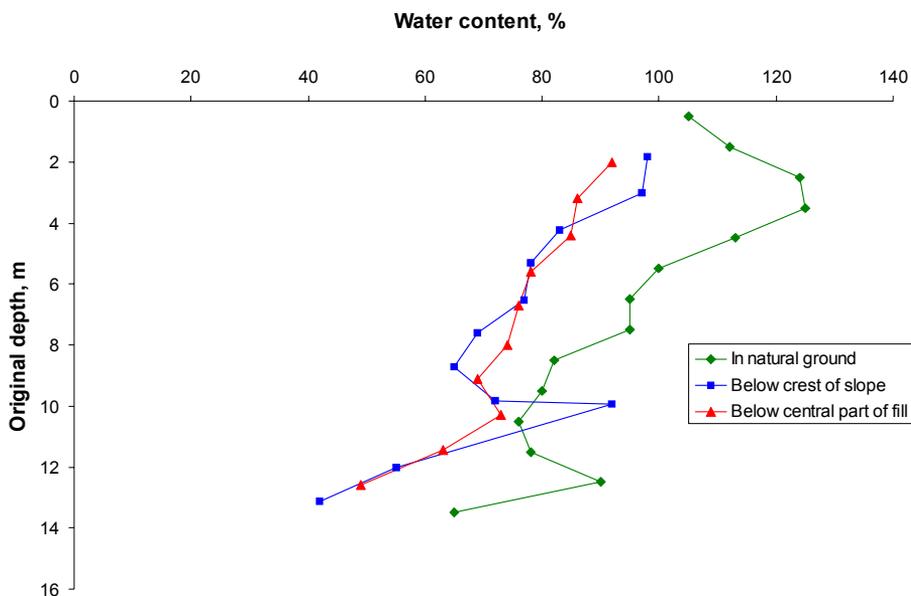


Fig. 11. Water contents in natural ground and below the fill in 2002.

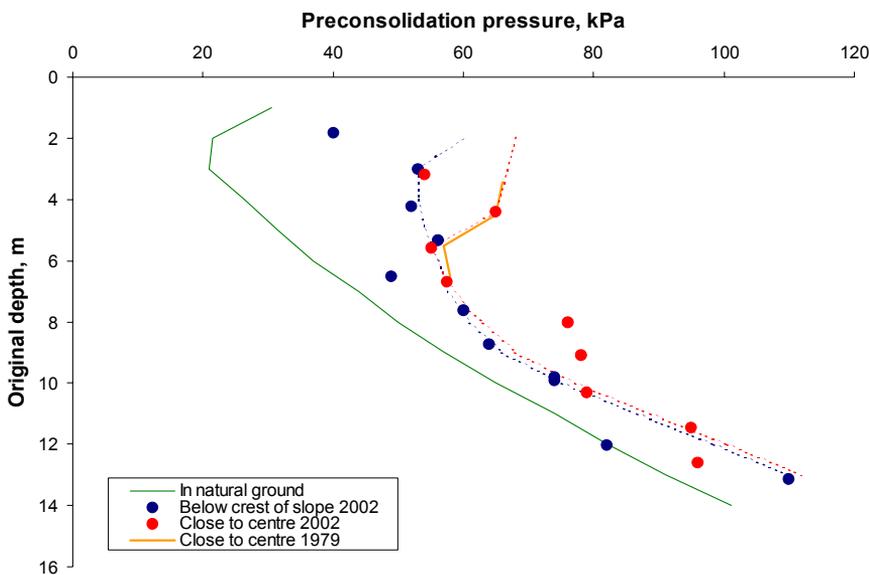


Fig. 12. Evaluated preconsolidation pressures below the fill at Lilla Mellösa.

also be observed that the quasi preconsolidation pressures below the centre are practically the same as those measured here in 1979, indicating that no significant further compression has occurred in these layers since then. A further indication of that some of the preconsolidation pressures evaluated from the oedometer tests are too low can be obtained from the results of the field vane tests and the CPT tests. The empirical correlations for evaluation of preconsolidation pressures from these tests in general give very good and consistent results in this soil, and the comparison clearly indicates that some of the lowest preconsolidation pressures from the CRS test are too low, particularly in the upper part of the profile.

The permeabilities have also been evaluated from the results of the CRS-oedometer tests. The results show that there has been a considerable decrease in permeability due to the compression of the soil, particularly in the top and bottom layers where the relative compression is largest, Fig. 13. The average decrease from about  $9 \cdot 10^{-10}$  to about  $2.5 \cdot 10^{-10}$  m/s corresponds well with the decrease that should be caused by a relative compression of 14 – 15 %.

The results obtained until the early 80s were used in calibration of the new calculation model elaborated at SGI at that time (Larsson 1986). The now observed settlements, pore pressures, settlement distribution and shear strengths in general agree with the predictions made with this model.

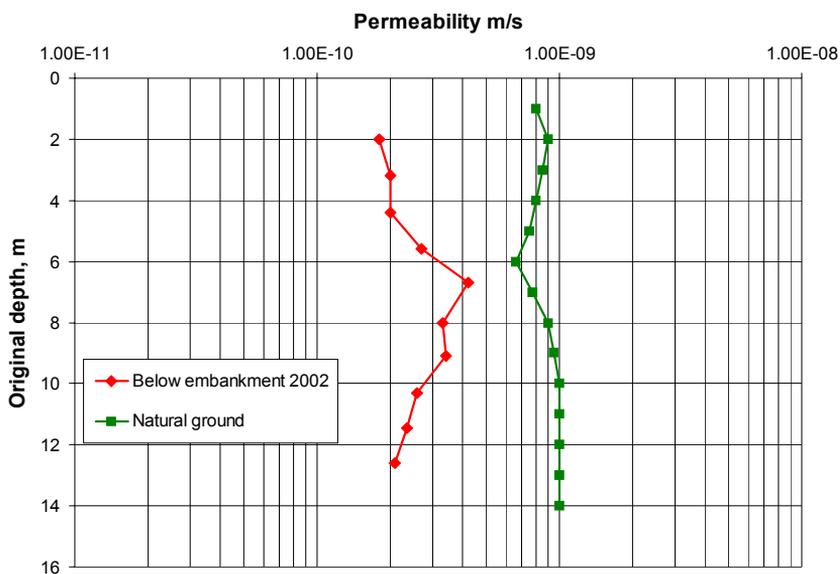


Fig. 13. Measured permeabilities in natural ground and below the fill at Lilla Mellösa.

## **3.2 SKÅ-EDEBY**

### **3.2.1 History and soil conditions**

#### **History**

After the initial results from the test embankments at Mellösa were available, this site was disregarded for the new airfield. Construction was instead started at a place called Halmsjön (today Arlanda) in 1946. However, this construction was halted after a while as the new airfield was postponed.

In 1956 Scandinavian Airlines System placed an order for new Douglas DC8 jet planes. The runways at the old airfield at Bromma could not be extended and the construction of a new airfield became urgent. At that time, however, it was thought that Halmsjön, which is situated about 40 kilometres north of Stockholm, was too far away. Skå-Edeby, situated on an island about 25 kilometres west of Stockholm, was a possible alternative. While the conditions at Halmsjön were well known, too little was known about the soil conditions at Skå-Edeby to make a decision.

In spring 1957, the Swedish Geotechnical Institute was therefore commissioned by the Government to carry out field tests to investigate the possibilities for construction of an airfield at Skå-Edeby. The construction time had to be short and as the soil consisted of up to 15 metres of soft clay the only practical solution would have been to use vertical drains and preloading.

The investigations thus became a close study of consolidation of soft clay and the effect of vertical drains. For this purpose, four circular test fills with diameters from 70 to 35 metres were constructed. One test fill was undrained (i.e. without vertical drains) while vertical sand drains with varying spacings were installed under the other fills. The surcharges for the drained fills were also varied.

Work was started immediately with field investigations, sampling, installation of measuring devices, installation of drains and construction of the fills. This work was completed three months later at the end of July 1957. In these operations, the experience from Lilla Mellösa was very valuable. New types of settlement markers and piezometers (Kallstenius and Wallgren 1956) were constructed and installed for the first time and an attempt was made to measure the horizontal movements by measurement of the change in inclination of flexible pipes installed vertically at the toes of the slopes.

The location of the test area was not quite ideal as the depth to firm bottom varied from 12 to 15 metres for the fill locations. The area was selected bearing in mind that if an airfield was to be constructed the test area had to be out of the way. The layout of the test area is shown in Fig. 14a and a view of the area today can be seen in Fig. 14b.

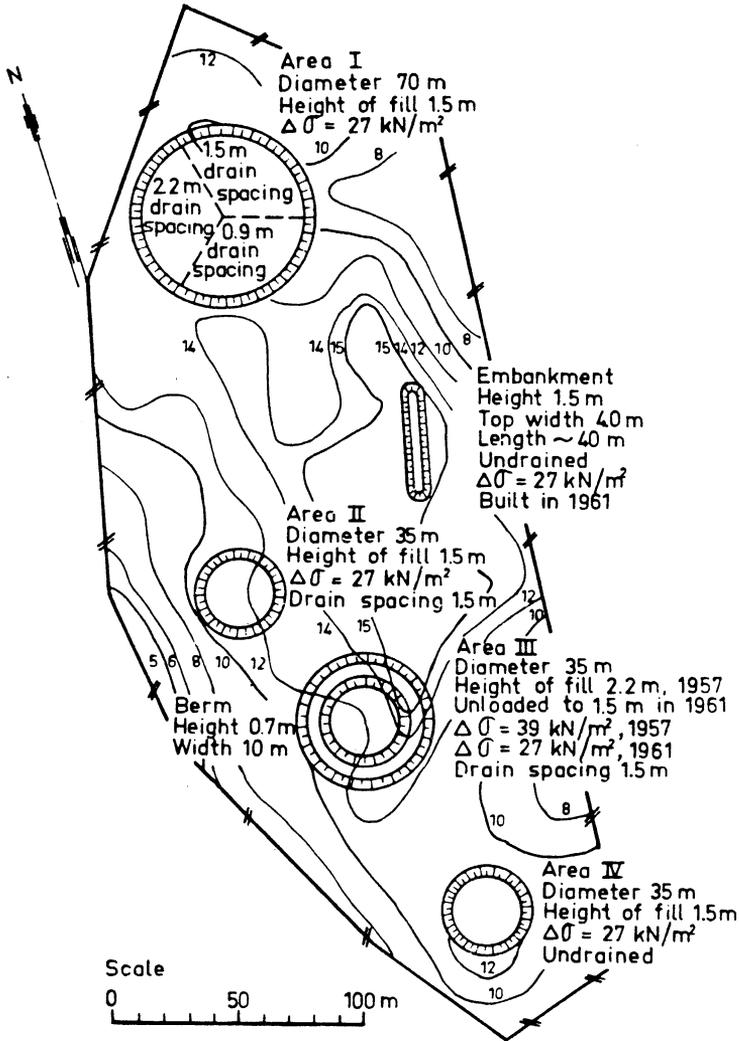


Fig. 14. The test field at Skå-Edeby.  
 a) Main test areas and depth from ground surface to firm bottom in the test field at Skå-Edeby. (Holtz and Broms 1972)



Fig. 14. The test field at Skå-Edeby.  
b) View of the area from the west in 2002. The fill in the front with a car parked on it is Test area II. The other fills can only be surmised since they are now almost level with the ground and hidden behind the grass.

Parallel to the following field observations, supplementary field investigations and an extensive laboratory investigation were carried out.

The first results were published in September 1957 (Utlåtande angående Stockholms storflygplats 1957) but were too preliminary to form a basis for a definite conclusion. Later, however, the Skå-Edeby alternative had to be abandoned for economic reasons. In spite of this, the investigations were continued as the results were important for future similar projects and particularly for road construction.

A full report on the results of the investigations at Skå-Edeby including the first years' measurements and a revised theory for consolidation of clays with vertical drains was presented by Hansbo in 1960.

In 1961, the most heavily loaded test fill was partially unloaded. The surplus gravel was used to construct an additional test fill which was given the shape of an embankment. The test embankment was instrumented with the usual settlement markers and piezometers. New flexible inclinometer tubes were also installed to be used together with a new inclinometer specially designed for measurement in soft clays (Kallstenius and Bergau 1961).

Preliminary results from this test fill were reported by Osterman and Lindskog in 1963. Readings of the instrumentation in all the test fills were then taken periodically. A new thorough investigation, including measurements of changes in properties under the fills and measurements of pore pressures with more modern piezometers, was started by Holtz in 1970. The results were reported at the Purdue Conference in 1972 (Holtz and Lindskog 1972, Holtz and Broms 1972). Further measurements have been made regularly since then, but at longer time intervals. The pore pressures under the undrained fill were measured in 1982 in connection with testing of a new measuring system. New samples were taken outside and under this fill in 1984 and 1985 in connection with the development of a new calculation method of settlements (Larsson 1986).

A large number of other research activities have been carried out at the test field at Skå-Edeby. This was one of the sites where the effects of various factors on the quality of clay samples were studied in connection with the standardization of piston sampling in Sweden (Kallstenius 1963). A new test fill has been constructed to compare the effects of sand drains and prefabricated drains (Torstensson 1976). Other test fills have been constructed to evaluate the effect of lime columns. The stabilizing effect of lime columns at excavations has also been tested (Boman and Broms 1975). At one of the old drained fills, a part of the fill has been removed and

the sand drains partly excavated. The condition of the drains, the distribution of water content and zones affected by the installation of the drains were studied (Holtz and Holm 1972). A large study of the effect of size of piston samplers on the quality of the samples was performed at Skå-Edeby after Norwegian and Canadian tests had shown large size effects. No significant practical differences between the Swedish standard piston sampler and piston samplers with larger diameters were found, however (Holm and Holtz 1977).

The test field has also been used in research concerning the vane shear test (Wiesel 1975) and in testing various piezometers and the piezometer sounding method. It has also been used in projects concerning CPT tests, seismic CPT tests and initial shear modulus as well as dilatometer tests in soft clays (Larsson and Mulabdic 1991, Larsson and Eskilsson 1988).

### ***Soil conditions***

The soil under the test fills consists of soft clay with a thickness of 12 to 15 metres on top of till or rock. Also in other respects the conditions are not quite uniform since most of the soil profile is significantly varved. The water contents and liquid limits in the upper two metres vary between the different test locations, as does the level of a high-plastic clay layer in the upper clay profile. These variations are, however, not much larger than the scatter of results within the separate test areas when different investigations for the test fields are collated. A large part of the scatter is probably related to the varved nature of the clay and that the determinations of various properties are made on small specimens of soil with very limited volumes. The same observation is valid also for strength and deformation characteristics, so that a general soil profile is probably most representative for the entire area. A geological profile and grain size distribution determined at the largest test fill is shown in [Fig. 15](#).

The desiccated dry crust is only about half a metre thick. Below the crust, there is a layer of grey-green organic clay which in spite of the relatively low clay content is very high-plastic due to the organic content. This layer is affected by the closeness to the ground surface and is thus overconsolidated with a relatively high shear strength and water contents lower than the liquid limits.

The underlying postglacial clay is slightly organic and high-plastic. The postglacial clay, as well as the glacial clay beneath it, is coloured by or banded by iron sulphides. The contents of iron sulphide may be assumed to be of the same order as at Lilla Mellösa.

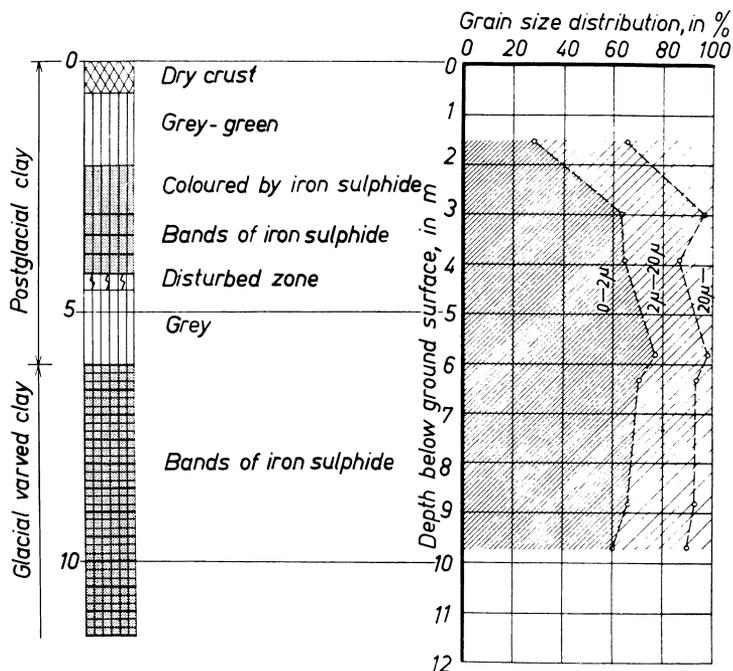


Fig. 15. Geological profile and average grain size distribution in a core taken in Test Area No. 1. (Hansbo 1960)

The glacial clay is varved. The varves are thin at the top but become thicker with depth. Near the bottom, occasional seams of silt and sand are found. Bedrock or dense till is found below the clay. Both can be considered as free draining. The geology of the site has been discussed in detail by Pusch (1970), who investigated the microstructure and chemistry of the clay. Some leaching of salts has occurred in the profile. The general soil properties are shown in Fig. 16.

The water contents in the clay are well above the liquid limit except for the upper two metres which are affected by dry crust effects. The water contents decrease from about 100 % at the top to about 60 % in the lower layers. The bulk density of the clay increases from 1.3 t/m<sup>3</sup> at the top to about 1.7 t/m<sup>3</sup> at 12 metres' depth.

The shear strength has been determined by field vane tests, fall cone tests and unconfined compression tests. Field vane tests and fall cone tests normally yield compatible results at depths down to 10 – 15 metres. The averages of the results are in good agreement but the scatter in the results is unusually high for this type of clay. In the original stability calculations a cautious value of 5 kPa was used. The compiled shear strength measurements show, however, that a minimum shear

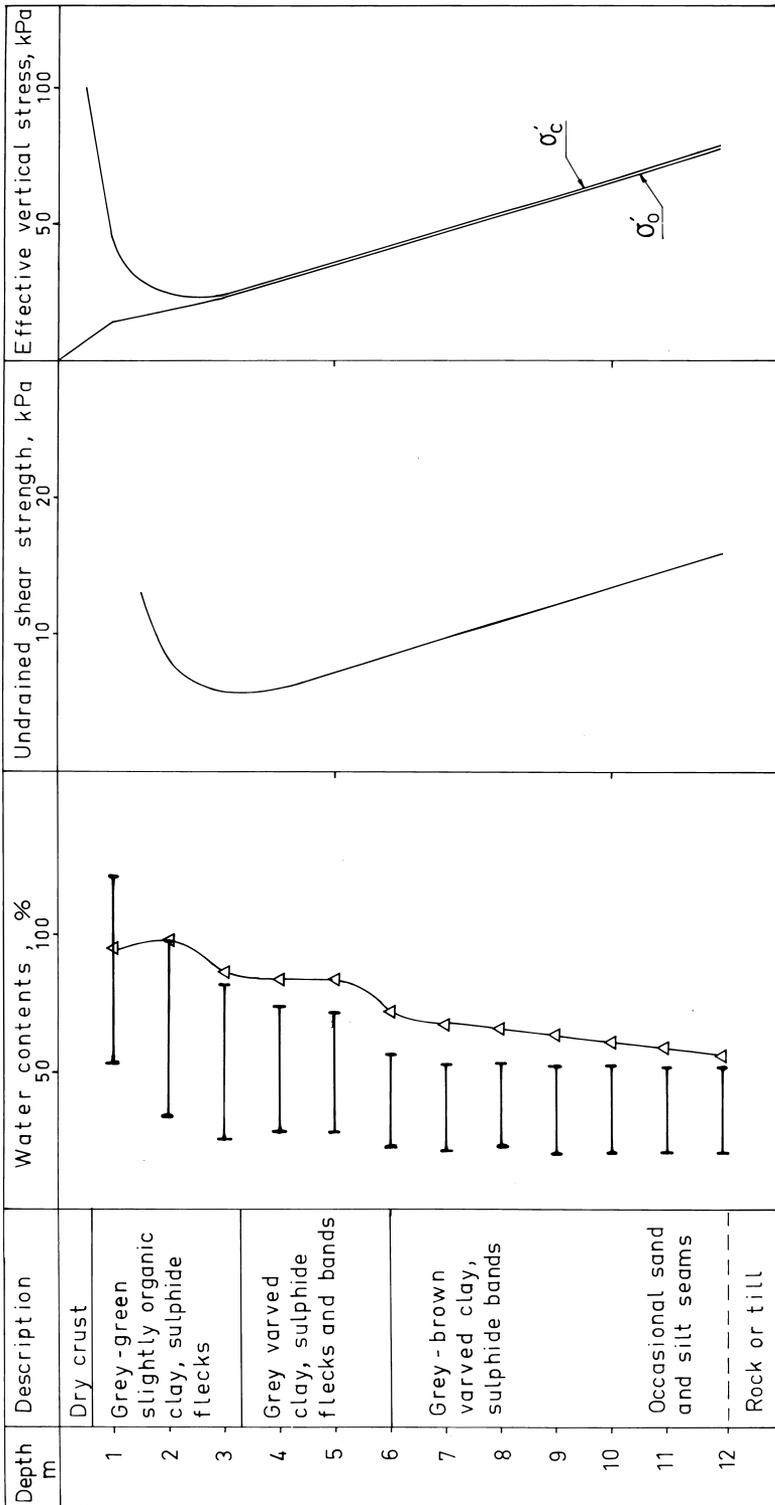


Fig. 16. General soil properties at the Skå-Edeby test field. (Larsson 1986)

strength of 6 kPa at a depth of 3.5 metres and an increase with depth of 1.2 kPa per metre thereafter is a more realistic value. Still, the clay is very soft even for Swedish conditions.

The sensitivity was found to vary from about 5 in the upper layers to about 15 at the bottom in the early investigations. In the investigations in 1971, the sensitivity in the ground outside the fills was found to be about twice as high as in the early investigations, or about 10 – 30. Both sets of values are compatible with normal values for clays of this type and no obvious explanation for the discrepancy has been found as all other parameters from the different testing times agree quite closely. Experience has shown that factors such as the time and temperature at storage before testing can affect the measured sensitivity, and this may be an explanation.

The groundwater level and pore pressure in the ground have been found to vary seasonally. The maximum measured variation is  $\pm 0.5$  metres. The pore pressure in natural ground can generally be assumed to be approximately hydrostatic for a groundwater level which can vary from the ground surface to 1 metre below.

The preconsolidation pressures have been measured in incremental oedometer tests; originally by Hansbo (1961) then in 1971 by Holtz and Broms and again in a large number of tests by Holm and Holtz (1977). Finally, the preconsolidation pressures were measured in CRS tests in 1984 in connection with the study of consolidation processes including creep effects (Larsson 1986). The results are unanimous in showing overconsolidation effects in and just under the dry crust, after which the soil becomes only slightly overconsolidated for a groundwater level located one metre below the ground surface and hydrostatic pore pressures below. The overconsolidation ratio at average pore pressure conditions is at least 1.15.

The permeability of the clay has been determined in a comprehensive series of tests with varying gradients and degrees of compression by Hansbo (1960). Some deviations from Darcy's law were observed in this investigation. Supplementary determinations of the permeability were made by CRS tests in the study in 1984. The results are in good agreement, but the CRS tests comprise more levels.

A detailed description of the compressibility and permeability characteristics of the soil in the Skå-Edeby test field is given in SGI Report No. 29 (Larsson 1986).

### ***3.2.2 Construction and instrumentation of the undrained circular test fill***

The undrained test fill was constructed in June-July 1957. It was made up of gravel and given a bottom diameter of 35 metres and slopes 1:1.5. The density of the gravel fill was checked continuously and was 1.79 t/m<sup>3</sup> on average. The total height of the fill was 1.5 metres and the imposed load intensity thus 27 kPa. No changes of load except for natural variations have been made after that.

Before construction, newly constructed settlement markers were placed on and at various depths below the ground surface. Most of them were placed at the centre of the fill, but some were also placed at various distances towards the perimeter to enable measurement of the settlement distribution across the fill. Piezometers for pore pressure measurements were placed in a similar pattern. The instrumentation also included vertical pipes at the perimeter of the loaded area for measurement of horizontal movements by inclinometers and some surface settlement markers outside the area for measurement of possible heave. No relevant or significant measurements appear to have been obtained from these last two sets of instrumentation.

The settlement markers below the test fill appear to be still functioning, but the piezometers stopped working after the first years of frequent measurements and maintenance. The pore pressures have later been measured by retractable piezometers at a few instances, the last being in the investigation in 2002. Sampling and field vane tests below the fill were performed in 1971 and 2002 whereas only sampling was performed in 1985. In the investigation in 2002, CPT tests were also performed.

### ***3.2.3 Measured settlements, settlement distributions, excess pore pressures and changes in shear strength, preconsolidation pressure and other properties***

The settlements during the construction period were measured and found to be about 0.06 metres. The computed “final” settlements disregarding creep effects are about 0.75 metres. This amount of settlement was obtained in 1972. At that time, there were still excess pore pressures in the order of 20 kPa. This means that fifteen years after load application there was practically no increase in effective vertical stress in large parts of the clay profile. Twenty-five years after the load application, the settlements amounted to 0.95 metres and there were still excess pore pressures in the order of 12 kPa in the middle of the clay profile. Because of the settlements, the dry crust and parts of the fill had become submerged and the initially applied

effective stress increase had thereby been reduced by about one third due to the settlements after 25 years. In 2002, 45 years after construction, the settlements amounted to 1.10 metres and the maximum remaining excess pore pressures were about 8 kPa. A view of the fill in 2002 is shown in [Fig.17](#).



*Fig. 17. The undrained circular test fill in 2002.*

The distribution of settlements from the centre of the fill outwards shows that the initial deformations were evenly spaced and decreased towards the edges of the fill. There were thus no excessive shear deformations at the edges. The distribution of settlements with time showed, however, that there was some effect of three-dimensional consolidation and horizontal water flow. The maximum settlement rate thus occurred at a distance of about 10 metres from the centre of the fill where the load concentration is still high but the horizontal drainage path relatively short, [Fig. 18](#).

Another observation during the first years of observation was that there was a pronounced seasonal variation in settlement rate. These variations in themselves varied for different layers. The external factors causing the variations are varying ground water levels, varying water contents in the fill, varying temperatures, snow cover and freezing. These factors affect different layers differently and sometimes work in opposite directions. The measured settlements are shown in [Figs. 19 a and b](#).

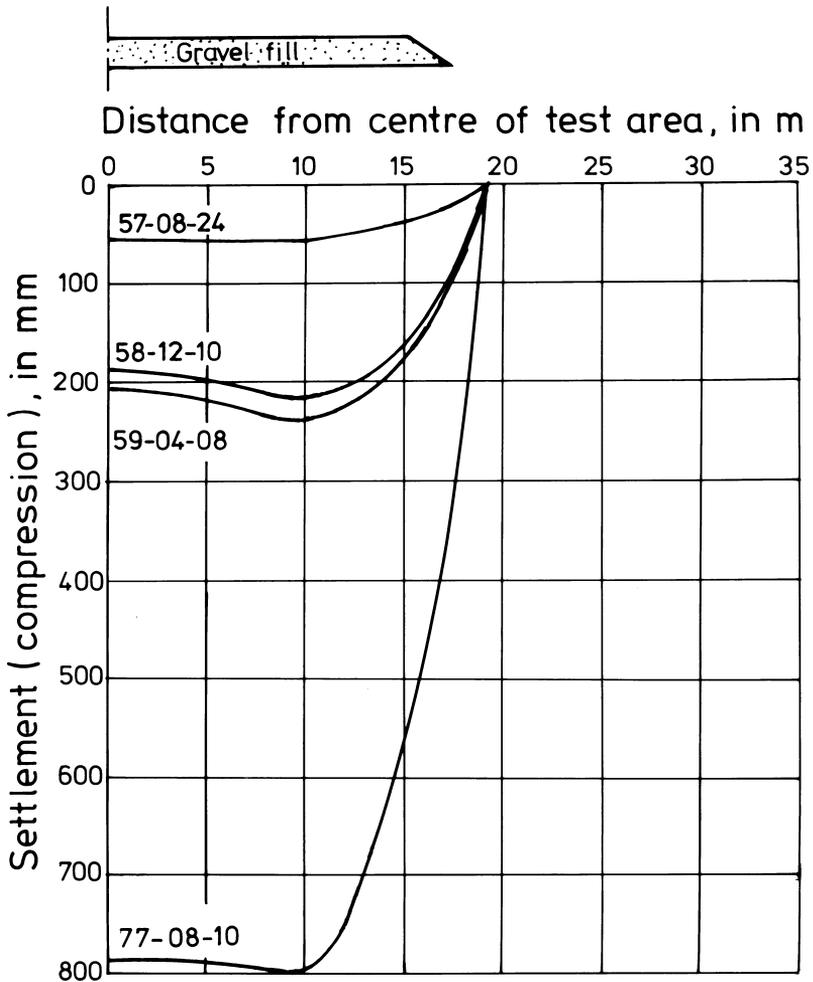
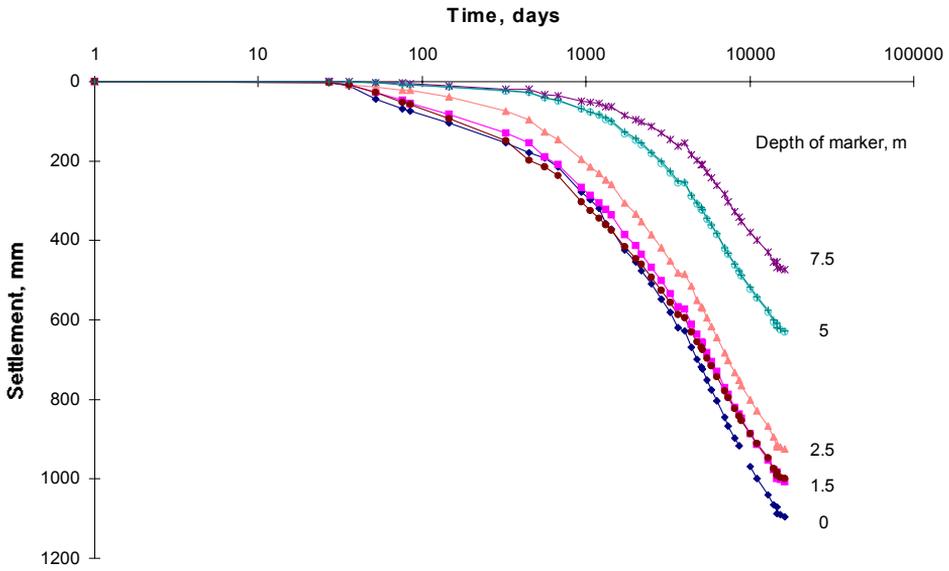
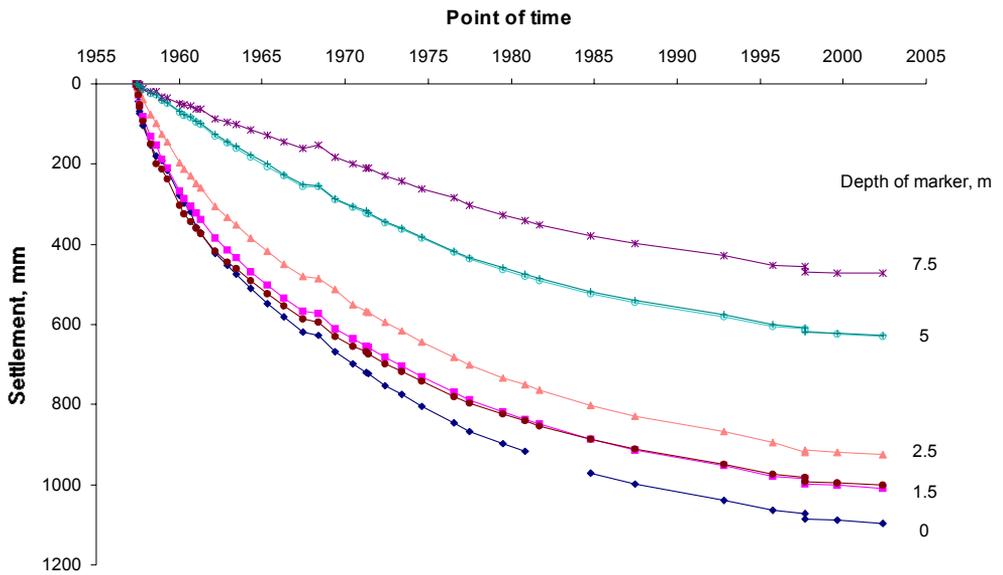


Fig. 18. Distribution of settlements in the undrained circular fill. (Larsson 1986).



a)



b)

**Fig. 19.** Measured settlements under the undrained circular fill at Skå-Edeby.  
 a) time in log scale  
 b) time in linear scale

The settlements continue although at a continuously decreasing rate. The settlement rate in 2002 was 5 – 6 mm/year. The distribution of settlements across the loaded area is still very uniform except for the outermost parts. The difference between the settlements below the central part and below the outer crest of the fill is about 0.25 metres, which means that the settlements below the crest are about 80 % of those below the centre. The distribution is thus similar to that at the fill in Lilla Mellösa, which had similar dimensions and depths to firm bottom.

The settlement distribution with depth under the centre of the embankment has been measured by markers at various depths. It has also been checked by the settlement distribution calculated from the measured change in water content after 45 years in 2002. There is a significant scatter in both original values and those measured in 2002, but the average values from the samples taken at the centre and at the crest of the fill indicate that there has been decrease in water content throughout the profile. The agreement between the calculated and measured distributions indicates that the markers are still functioning, Fig. 20.

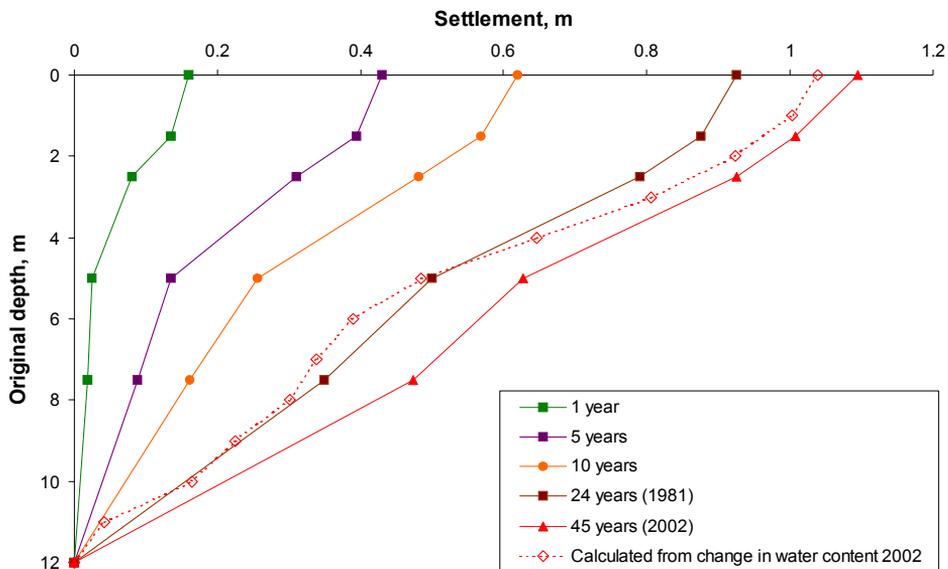


Fig. 20. Measured settlement distribution with depth below the undrained circular test fill at Skå-Edeby.

The observations show that the settlements in the early stages mainly consisted in compression of the upper layers whereas the settlements later on have mainly turned into compression of the bottom layers. The settlements have thereby become fairly evenly distributed in the profile with time. As in Lilla Mellösa, the effective load has decreased due to the submerging of the crust and part of the fill below the groundwater table and parts of the profile are thereby in a state of unloading. The settlements in large parts of the upper soil layers can also be observed to have virtually stopped. The exact distribution of the ongoing compression in the lower layers cannot be discerned since there are no settlement markers below 7.5 metres' depth.

The excess pore pressures after load application were reported by Hansbo (1960). They show that the pore pressure response to the load was only slightly lower than the increase in vertical stress, which confirms that the soil was almost normally consolidated, Fig. 21.

The pore pressures at the centre of the fill as well as in natural ground were carefully measured in 1971, 1982 and in 2002. The excess pore pressures at those times are shown in Fig. 22. It should be observed that about 10 kPa of the decrease in maximum excess pore pressure from around 25 kPa to the present 7 – 8 kPa is due to load reduction because of the settlements.

Samples were taken under the fill in 1985 and 2002 and oedometer tests were performed. The preconsolidation pressures are shown in Fig. 23. The results show that a considerable increase in preconsolidation pressure has occurred throughout the profile. The measured preconsolidation pressures are also significantly higher than the maximum effective vertical stresses that have acted, which indicates that creep processes have created significant quasi preconsolidation effects. The preconsolidation pressures below the crest are also significantly increased but generally lower than those below the central parts, which should be expected since the compression of the soil is less here. The measured preconsolidation pressures below the central parts of the fill in 2002 are about equal to or somewhat less than those measured in 1985. A slight increase would have been expected with regard to the settlements that have occurred between the sampling events although they are relatively small. However, the pre-drilling of the gravel fill can cause disturbance of the clay below. There is also always a risk of getting too close to an old investigation point when making new investigations below old tests fills, where a considerable number of investigations in terms of sampling operations, pore pressure measurements and in situ shear strength tests have been performed throughout the years. Unless such points are clearly marked or can be located

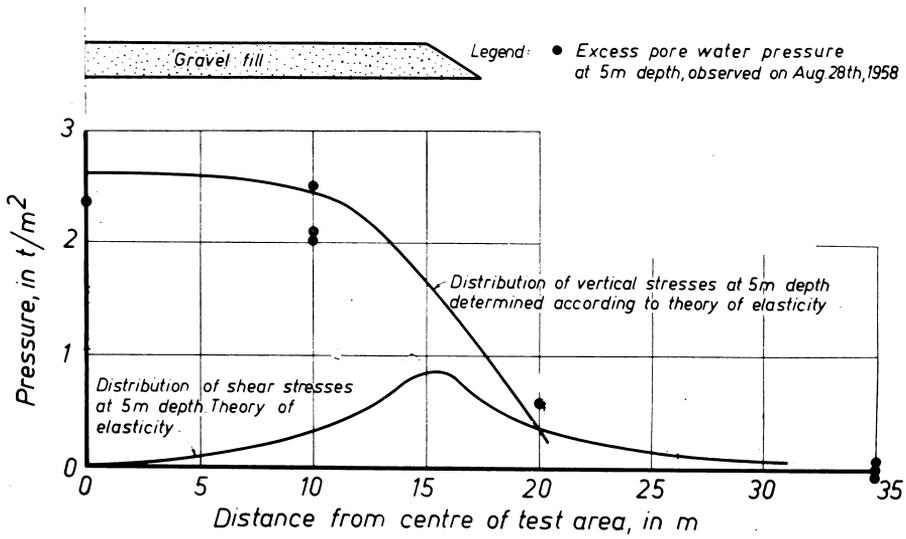


Fig. 21. Observed excess pore water pressure distribution at a depth of 5 metres below the undrained circular fill one year after load application compared with the theoretical stress increase produced at the same depth by the fill. (Hansbo 1960)

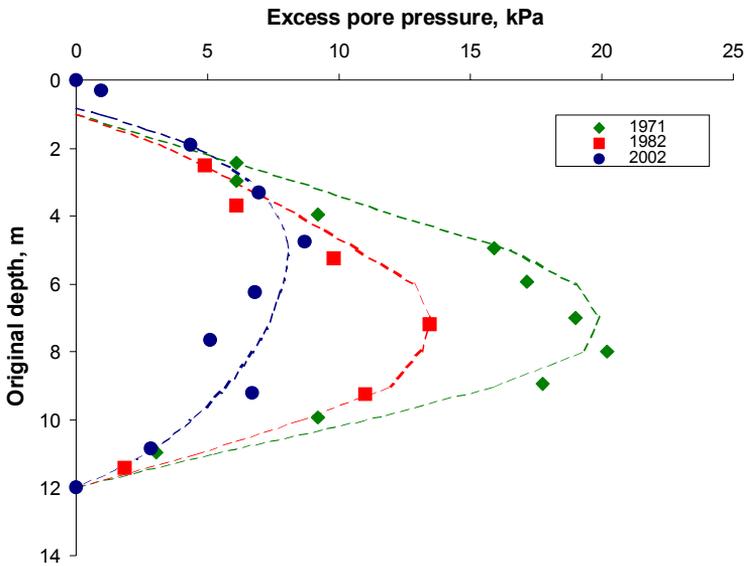


Fig. 22. Measured excess pore pressures at the centre of the circular fill at Skå-Edeby.

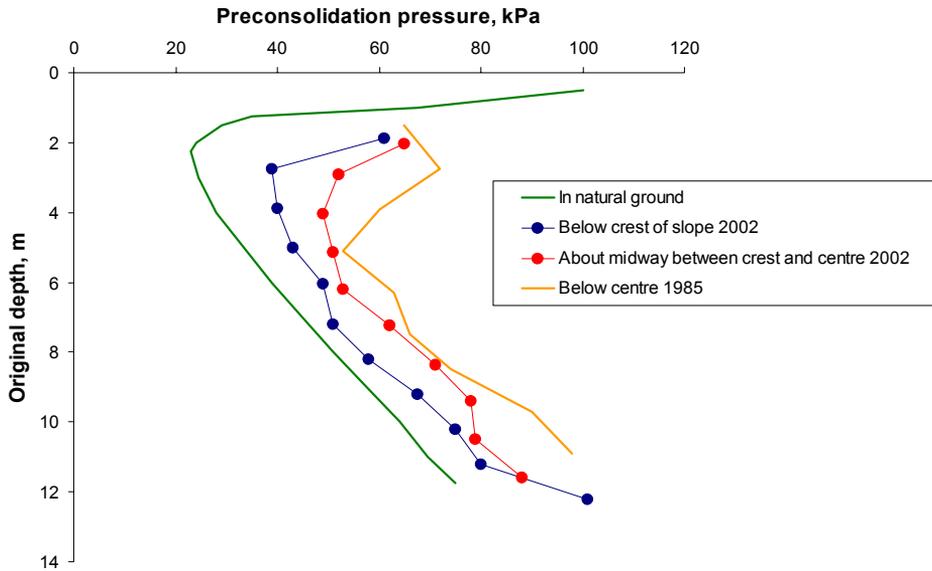


Fig. 23. Measured preconsolidation pressures below the undrained circular fill at Skå-Edeby.

otherwise, which is not the case here, all traces of them vanish with time. New investigations are likely to be located with the same idea of finding an area apparently untouched and uninfluenced by previous installations and operations. However, large parts of the most interesting areas are already occupied by permanent instrumentation and the risk of encountering an old investigation point in remaining seemingly suitable locations is thereby enhanced. The preconsolidation pressures measured in 1985 are therefore considered to be most relevant and the preconsolidation pressures in the lower part of the profile may even be somewhat higher today due to the compression that has occurred since then.

The permeability of the clay was also measured in the CRS tests. The results show that there has been a considerable reduction in permeability due to the compression of the soil, Fig. 24. They also indicate that there has been a significant reduction in the lower part of the profile between 1985 and 2002 which corresponds to the settlement observations. The permeability had decreased below the crest of the fill too but not as much as below the centre. This is also in general agreement with the observed settlements. The measured permeabilities are relatively insensitive to a possible disturbance. On the other hand, they are highly sensitive to variations in water content (or rather void ratio) and a certain scatter is to be expected in this varved soil.

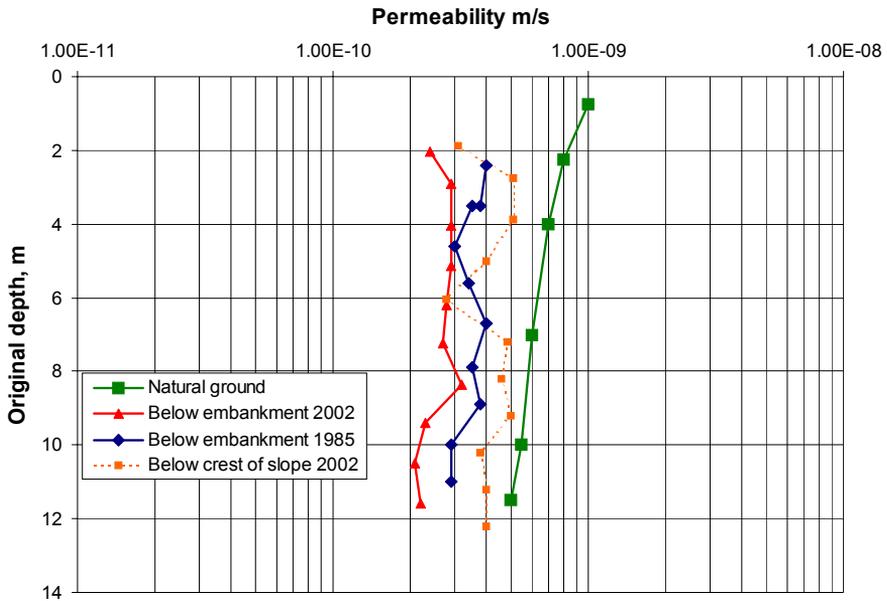
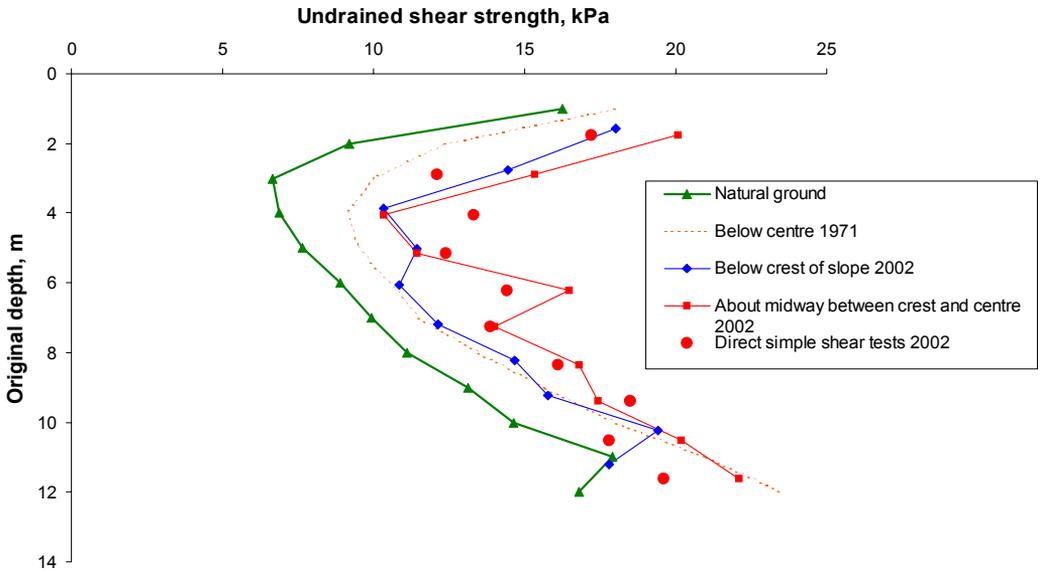


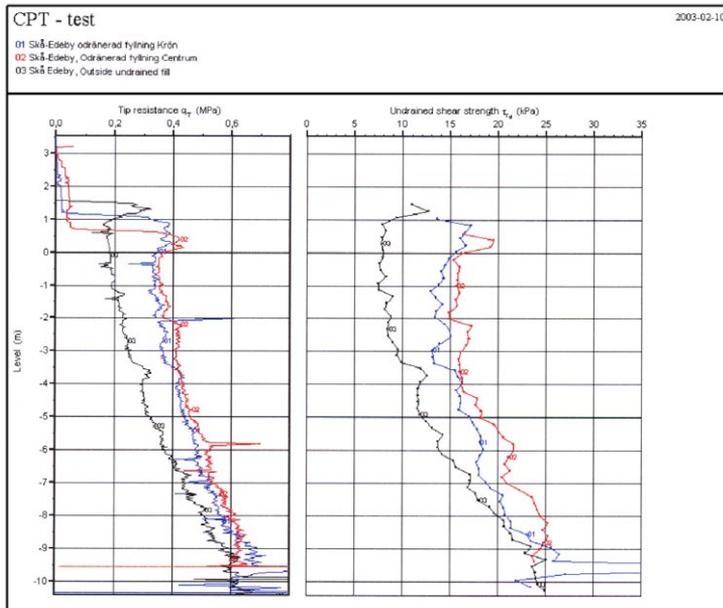
Fig. 24. Measured permeability below the undrained circular fill at Skå-Edeby.

The increase in undrained shear strength under the fill was checked by field vane tests and fall-cone tests by Holtz and Broms in 1971. As in the initial investigations there was a certain scatter in the results. The averaged shear strength profile with special reference to the results from field vane tests corrected according to SGI 1984 is shown in Fig. 25, together with the strength profile from 1957 and the results from the field vane tests in 2002. It can be observed that a considerable increase in shear strength has occurred between the different testing events and that the shear strength below the crest is similar but generally lower than below the centre of the fill. The relevance of the shear strength determined by the field vane tests has also been checked by direct simple shear tests and the results are similar.

Approximately the same relation between the shear strength in natural ground, below the crest and below the centre of the fill is obtained by the CPT tests, Fig. 26. However, the shear strengths evaluated by the CPT tests are generally somewhat higher, which may be related to the varved nature of the soil and the accompanying difficulty of selecting a relevant value of the liquid limit to be used in the evaluation. Also the relation between the shear strength below the crest and the other locations differs somewhat from the rest, which may be related to the aforementioned problem of selecting a relevant overburden pressure at this point.



**Fig. 25.** Results from shear strength tests below the undrained circular test fill at Skå-Edeby at various times and locations.



**Fig. 26** Results from CPT tests at various locations below and outside the undrained circular test fill at Skå-Edeby 2002. (It should be observed that these results are presented versus actual level and have not been corrected to original level as the other results)

### **3.3 THE TEST EMBANKMENT**

The test embankment was constructed in May 1961. At that time, the idea of locating an airfield at Skå-Edeby had been abandoned but the results of the tests were considered very important, especially for road construction. The test embankment was therefore constructed to study the influence of geometry and lateral deformations. It was sponsored by the Swedish Road Administration and the Building Research Council.

#### **3.3.1 Construction and instrumentation of the test embankment**

The embankment was constructed with the surplus gravel from the partial unloading of the most heavily loaded circular fill. It is a rather narrow embankment with a crest width of 4 metres and slopes 1:1.5. The height of the embankment was 1.5 metres and the total length 40 metres. The density of the fill material is 1.8 t/m<sup>3</sup> and the maximum load increase 27 kPa. The safety factor against failure was estimated to be about 1.5. The fill was instrumented with settlement markers and piezometers at different locations and depths under and outside the fill.

Flexible pipes for a newly constructed inclinometer measuring system were also installed at the toes of the embankment slopes, one on each side, and one pipe was installed further out from the embankment. Location of the instrumentation is shown in [Figs. 27 and 28](#). The construction of the fill was made in stages during 22 days and thereafter no changes have been made.

The piezometers functioned fairly well during the first years with frequent readings and maintenance. However, with time most of them ceased to function altogether or started to show odd trends and values. In 1971, 10 years after construction only two of the piezometers below or near the embankment showed apparently reliable values and thereafter they have not been read off anymore.

The settlement markers at depth were of a new type using detachable dial gauges and measuring the movement of the settlement marker relative to a levelled settlement marker on the ground surface or below the embankment. These systems required maintenance in that way that the length of the measuring rods had to be adjusted along with the settlements. They worked well as long as fairly frequent readings were taken but when the reading intervals became longer there were problems with exceeding the travel lengths of the dial gauges and keeping records of the adjusted rod lengths. The recording of the settlement markers at depth therefore stopped in 1981, 20 years after construction. At about the same time, the movements in the inclinometers tubes had become so large that it was no longer

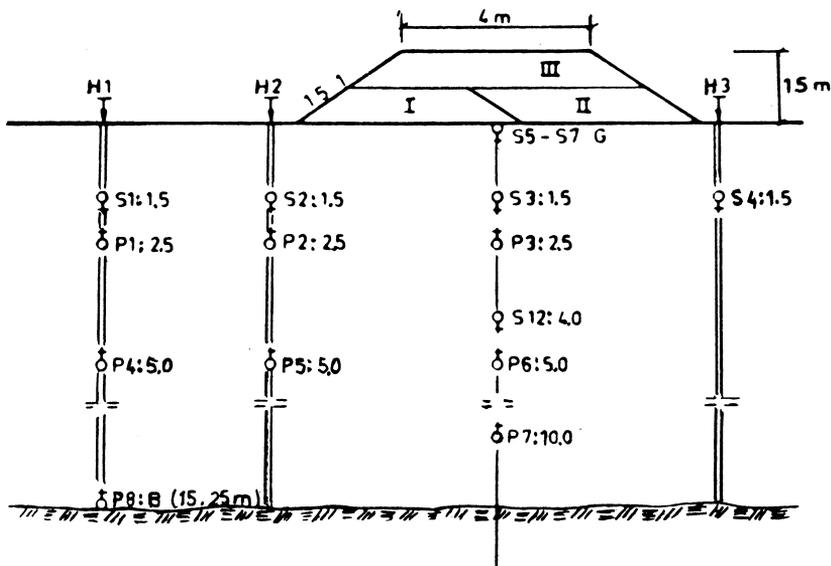


Fig. 27. Mid section of the test embankment at Skå-Edeby showing location of instrumentation. (Holtz and Lindskog 1972)



Fig. 28. The test embankment at Skå-Edeby in 2002. The permanent instrumentation at the midsection can be seen with the strongly inclined tops of the inclinometer tubes sticking up just outside the embankment. The length of the levelling rod is 4 metres.

possible to insert the instrument. Also these measurements were therefore stopped in 1981 and only registration of the surface settlements has continued.

In the present project, sampling and field vane tests have been performed below the centre of the embankment and at the toe of the slope on one side. CPT tests were performed at the same points and also in natural ground well outside the embankment. All these operations as well as the seismic investigations were located in the northern part of the embankment well clear of any influence from the permanent instrumentation.

### ***3.3.2 Soil conditions***

The soil conditions at the test embankment were practically identical to the conditions at the undrained circular test fill, except that the depth to firm bottom was 15 metres instead of 12.

### ***3.3.3 Measured settlements, settlement distributions, excess pore pressures and changes in shear strength, preconsolidation pressure and other properties***

The total settlements during the construction period were measured to be about 0.06 metres. The first stage of construction involved about half of the final load and the settlements were 0.01 – 0.02 metres. The horizontal movements after this first load step were small. The next inclinometer readings were taken two weeks after the completion of the embankment. Thereafter, frequent readings were taken and the horizontal movements were recalculated into a corresponding vertical movement of the embankment.

An extrapolation of the calculated vertical settlements due to lateral displacements indicates that about 0.05 metres of settlement is due to “initial” shear deformations directly after full load application.

The consolidation process has been associated with continuing horizontal deformations. As the horizontal deformations have been measured, the vertical deformations corresponding to them can be estimated and separated.

Some horizontal movements were still in progress 20 years after construction but they were then barely detectable and the relative importance for the total settlements has been steadily decreasing. The “final” consolidation settlement calculated in the ordinary way and disregarding creep effects is about 0.60 metres (0.80 metres if unloading due to settlements is disregarded). Figures of 1.2 and 1.5 metres

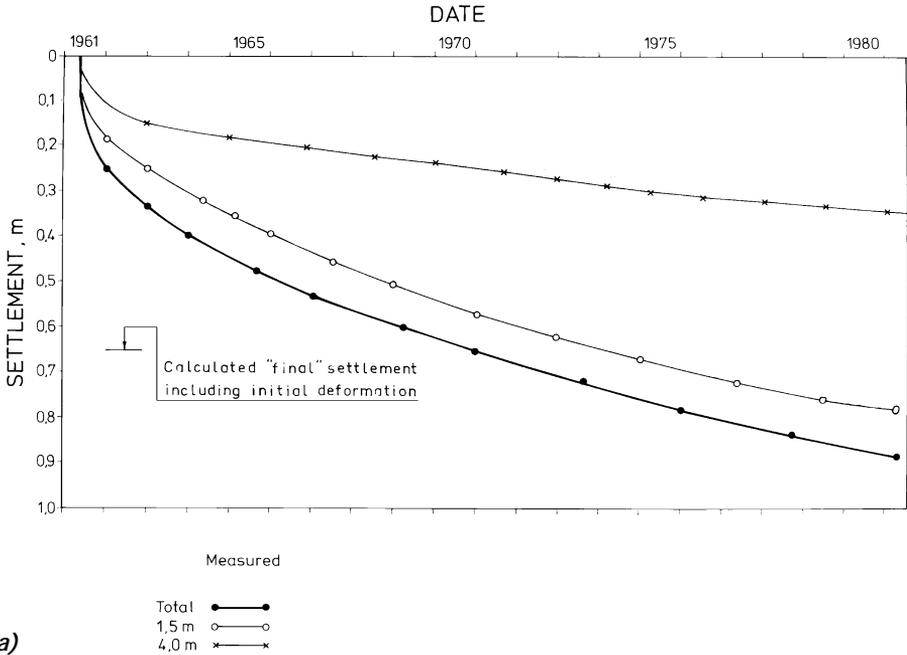
respectively referred to by Holtz and Lindskog (1972) must have been based on a more pessimistic assumption of the preconsolidation than further studies have shown, combined with a neglect of the load distribution with depth that occurs due to the narrow embankment and great depth to firm bottom. Even when correction for limited depth to firm bottom is applied, the load intensity as an average for the profile will still only be about half of the stress applied at the surface.

The calculated “final” settlements disregarding creep effects were reached in 1974 when corrections for the lateral deformations are made. The settlements are continuing but at a continuously decreasing rate. The settlements in 2002 amounted to 1.07 metres and the settlement rate was then 5 – 6 mm/year. The settlements of this narrow embankment are thus today approximately the same as for the wide circular fill with the same load intensity. However, this should be seen in the light of the fact that about 0.2 metres of settlement here depend on horizontal displacements of the soil and that the compressible soil layers are about 25 % thicker. The load from the test embankment has been applied for a somewhat shorter time but this should be more than compensated by the two-dimensional water flow and thereby faster pore pressure dissipation below this narrow construction.

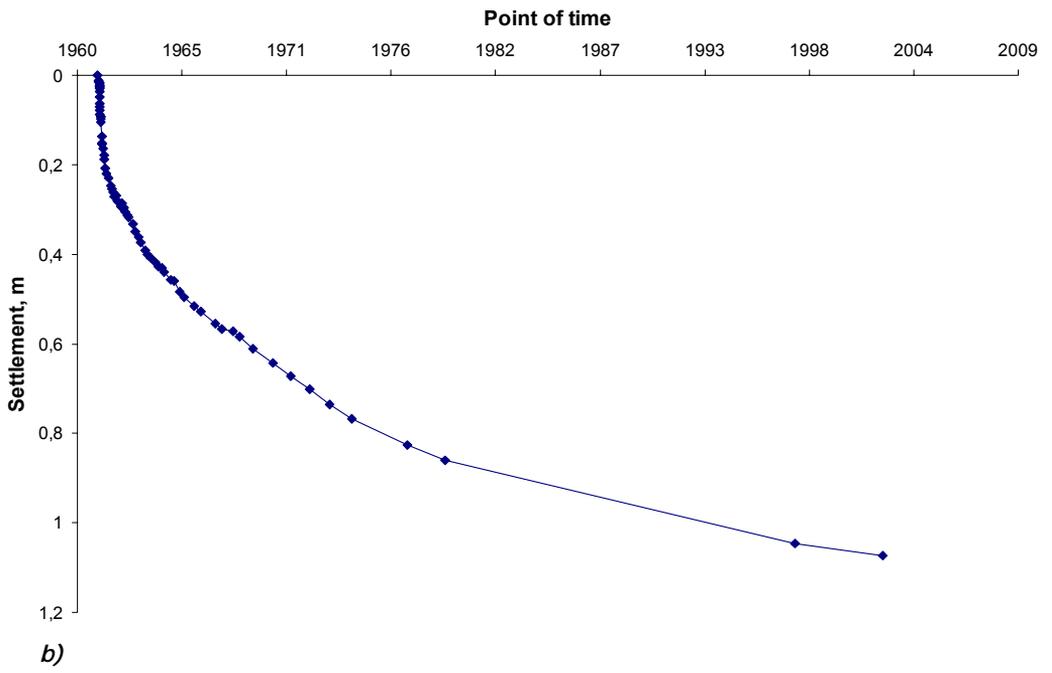
As for the undrained circular fill, a seasonal variation of the settlement rate was observed right from the beginning. Later slowdowns in the settlement rate have been observed for longer periods of a couple of years in 1968 – 1969 and in 1973 – 1974. Apart from these irregularities, which cannot be readily explained except possibly by a steady increase in groundwater level and pore pressures in the ground recorded during 1968 – 69 , the consolidation process has followed a smooth course. No pore pressure observations were made during 1973 – 1974.

The measured settlements are shown in [Fig. 29](#). These settlements include the effects of the horizontal displacements.

Measurements at various depths as well as measurements of changes in water content show that the bulk of the deformations have occurred within the upper part of the profile. The embankment is very narrow and the increase in effective stress decreases fairly rapidly with depth due to the load distribution. The slight overconsolidation is thereby sufficient to prevent any settlements developing below a certain depth except for relatively small creep deformations and horizontal movements. The measured and estimated settlement distributions are shown in [Fig. 30](#). The exact distribution of the settlements calculated from the water contents is uncertain because of a significant scatter, which has been smoothed out in the calculations. For the same reason, the exact depth of the breaking point below



a)



b)

Fig. 29. Measured settlements below the test embankment at Skå-Edeby.  
 a) settlements at different depths up to 1981 (Larsson 1986)  
 b) total settlements up to 2002 with time in linear scale

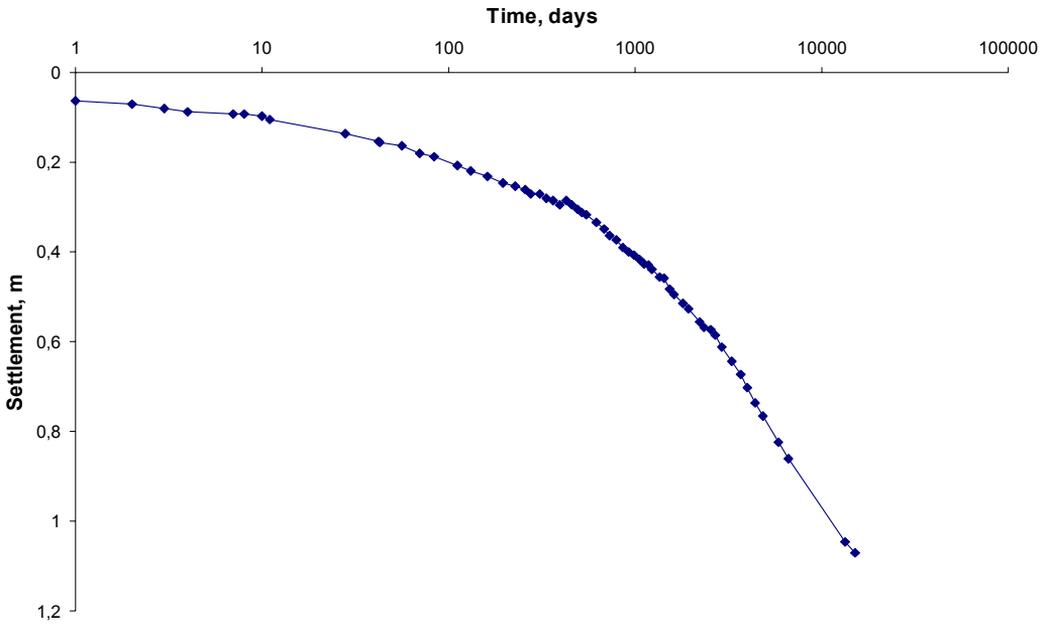


Fig. 29. Measured settlements below the test embankment at Skå-Edeby. c) total settlements up to 2002 with time in logarithmic scale

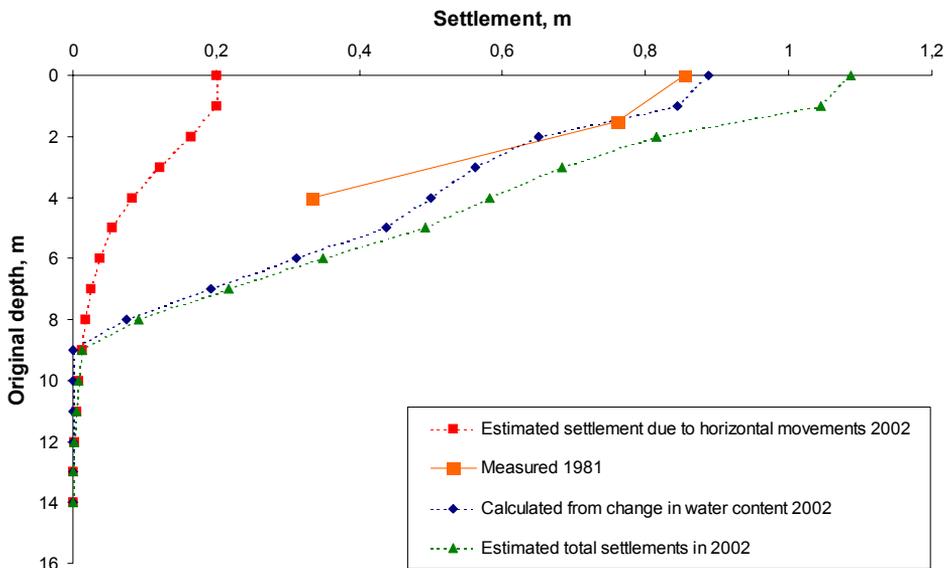


Fig. 30. Settlement distribution with depth below the test embankment at Skå-Edeby.

which the settlements are very small is also somewhat uncertain.

The horizontal movements at the toes of the slopes and also some 4 m outside the embankment are shown in [Fig. 31](#).

The maximum horizontal movements have occurred at a depth of 2 metres and amount to about 0.12 metres in each direction. The relative maximum of the ongoing horizontal movements has been located further down with time. In 1981, the movements at 2 metres depth seemed to have stopped and the maximum movements that occurred then were located at a depth of 4 to 5 metres. The horizontal movements have also shown seasonal variations and, at the slowdown of settlements in 1968 – 1969, the inclinometer tubes seemed to move inwards in the upper 3 – 4 metres. This effect vanished though and, apart from these irregularities and variations associated with measuring accuracy, there has been a smooth decrease in the rate of horizontal deformations. The influence of the horizontal movement on the vertical settlements has steadily decreased and amounts at present to about 0.2 metres or 20 % of the total settlements, [Fig. 32](#). The influence of horizontal movements in this case is unusually large due to the narrow embankment and the low factor of safety against undrained failure.

In 1981, the rate of horizontal movements was so small that the corresponding vertical deformations only amounted to about 5 % of the total rate of vertical deformations. The rate of horizontal movements can be assumed to be much smaller today both in absolute and relative terms and can for practical reasons be assumed to have stopped altogether.

The initial pore pressure response to the applied load was measured by piezometers located at the centre of the embankment at depths of 2.5, 5 and 10 metres. The pore pressure response to the applied surface load of 27 kPa was about 20, 18 and 13 kPa respectively. The calculated stress increase at the same levels was 25, 20 and 13 kPa. This confirms the general picture of an only slightly overconsolidated soil profile with depth and a somewhat higher overconsolidation in the uppermost layers. It should be noted, though, that there were large and sudden variations in the readings of all piezometers during the first years and the pressure in the reference piezometer located outside the loaded area and probably in contact with the draining bottom layers rose about 5 kPa during the loading phase. Part of this variation may have affected the readings further up in the profile. The long-term variations in the readings of the piezometers have all clearly shown a trend similar to the readings of this reference piezometer, [Fig. 33](#).

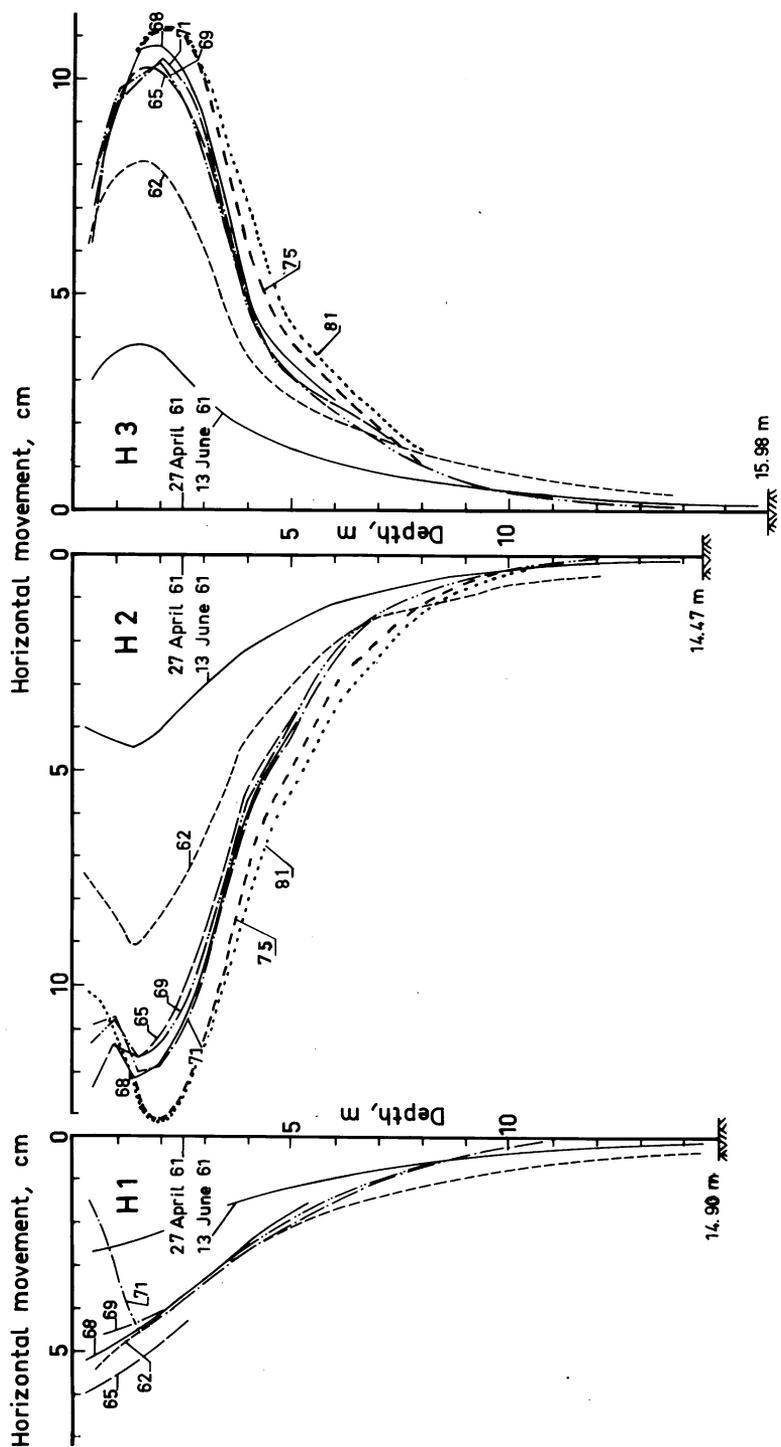
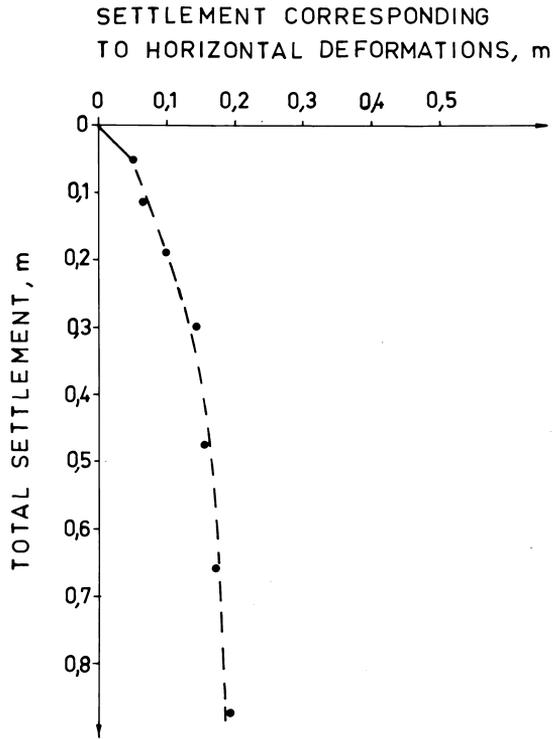


Fig. 31. Horizontal movements at the test embankment at Skå-Edeby. Inclinator tubes H2 and H3 are located at the toes of the embankment slopes; one at each side at the middle section. Tube H1 is located in the same section but about 4 metres outside the embankment. (Larsson 1986)



*Fig. 32. Settlements corresponding to horizontal deformations versus total settlements at centre of the test embankment at Skå-Edeby. (Larsson 1986)*

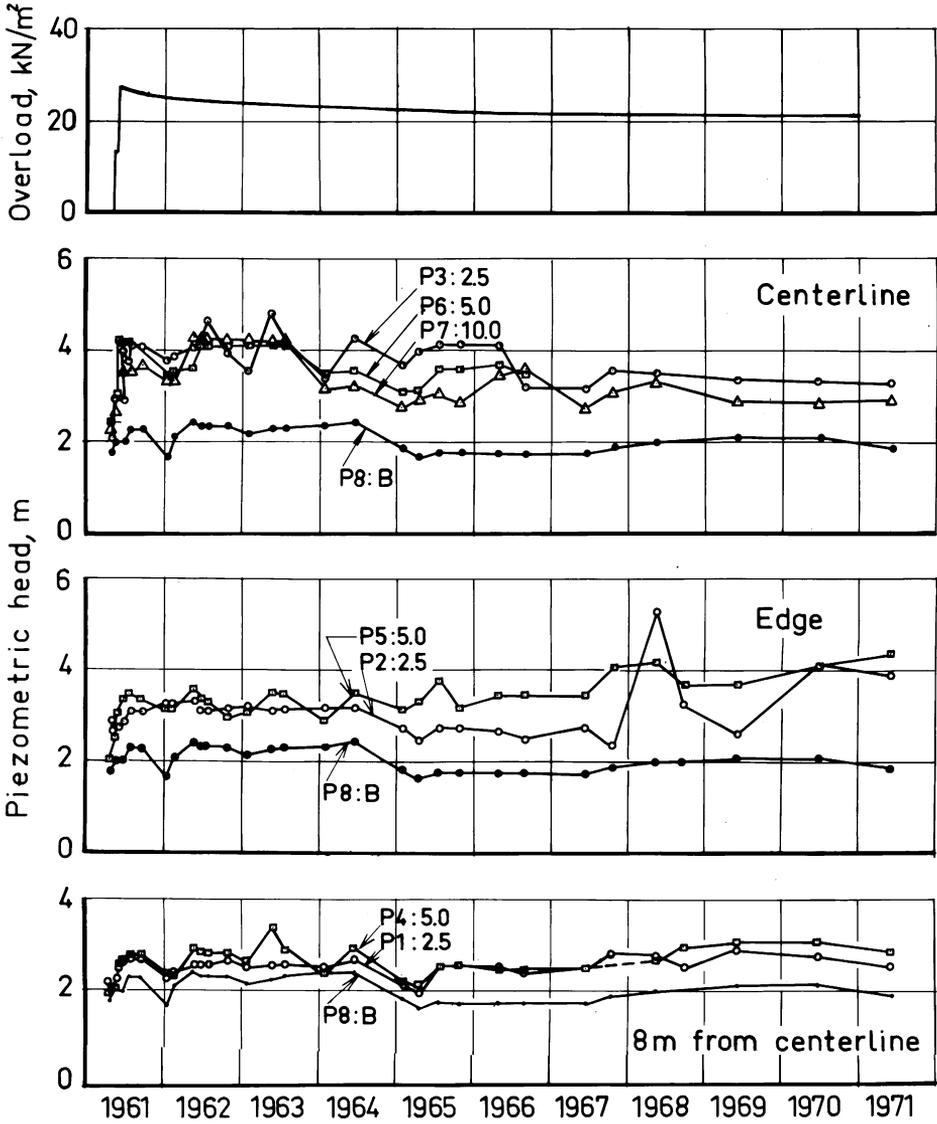


Fig. 33. Applied surface load and pore pressure readings at the test embankment. (From Holtz and Lindskog 1972)

Only two piezometers below the embankment were still in apparent function in 1971. The excess pore pressures measured then are shown in Fig. 34 together with the maximum excess pore pressures measured after load application in 1961 and those measured in the present investigation in 2002. As for the circular test fill, much of the pore pressure dissipation can be expected to be related to the decrease in effective load because of the settlements and the accompanying submersion of the crust and part of the fill below the groundwater table.

The preconsolidation pressures have been measured on samples taken in 2002 at the centre of the section across the embankment and at the toe of the slope at one side. The results show that significant increases of the preconsolidation pressure have only occurred within the 6–7 upper metres below the centre of the embankment, Fig. 35. Some increase has also occurred in the upper 4 metres at the toe of the slope, but this is more limited.

The change in permeability is a further indication of the compression of the soil. The permeability has been evaluated from CRS tests on samples from the centre and the toe of the slope of the embankment, Fig. 36. Also these values show that the major part of the compressions has occurred within the upper 6 metres or so of the soil profile and that the compression is somewhat larger at the centre than at the toe of the slope.

The results from the field vane tests scatter but show that there has been an increase in strength roughly in the 8 upper metres of soil below the embankment, Fig. 37. Also the soil below the toe has increased in strength but the scatter is too large to estimate whether there is any significant difference in strength to that below the centre. There is an anomaly with very low values around 4 metres original depth. At this depth, the liquid limit of the soil both at the centre and at the toe of the fill was considerably higher than in the general profile, and thereby in most of the determinations at this depth in the test field. It thus appears that the soil below the test embankment in the investigated section is somewhat different from the reference soil at this depth.

Approximately the same relation between the shear strength in natural ground, below the crest and below the centre of the fill is obtained by the CPT tests, Fig. 38. However, as for the circular fill the shear strengths evaluated by the CPT tests are generally somewhat higher, which may be related to the varved nature of the soil and the accompanying difficulty of selecting a relevant value of the liquid limit to be used in the evaluation. Also the relation between the shear strength at the toe and the other locations differs somewhat from the rest, which may be related to the problem of selecting a relevant overburden pressure at this point.

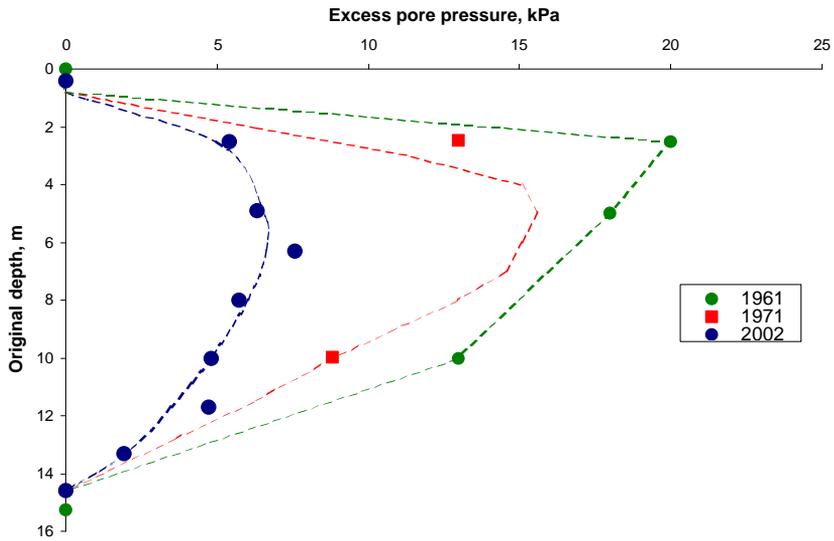


Fig. 34. Measured excess pore pressure profiles below the test embankment at Skå-Edeby.

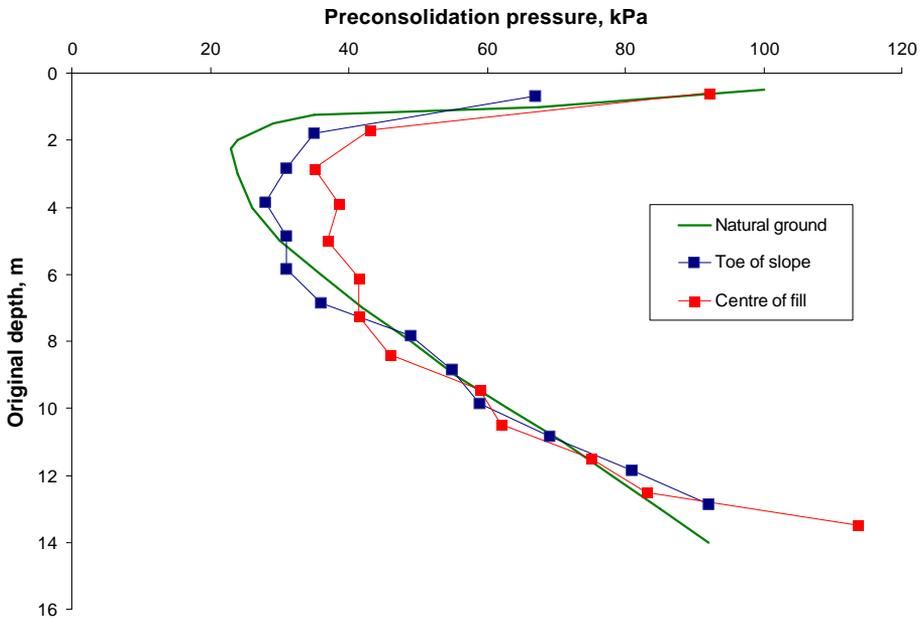


Fig. 35. Evaluated preconsolidation pressures at the test embankment at Skå-Edeby in 2002.

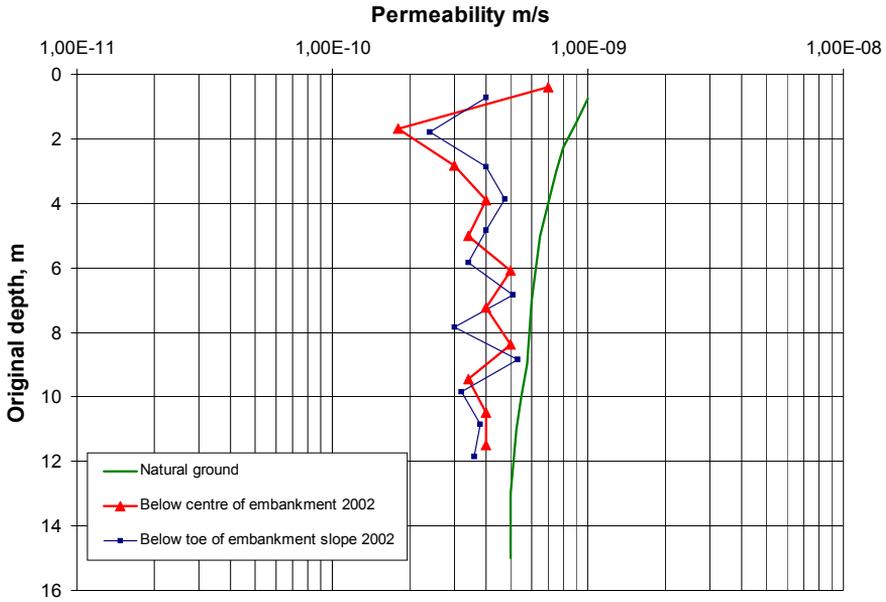


Fig. 36. Evaluated permeability at the test embankment at Skå-Edeby.

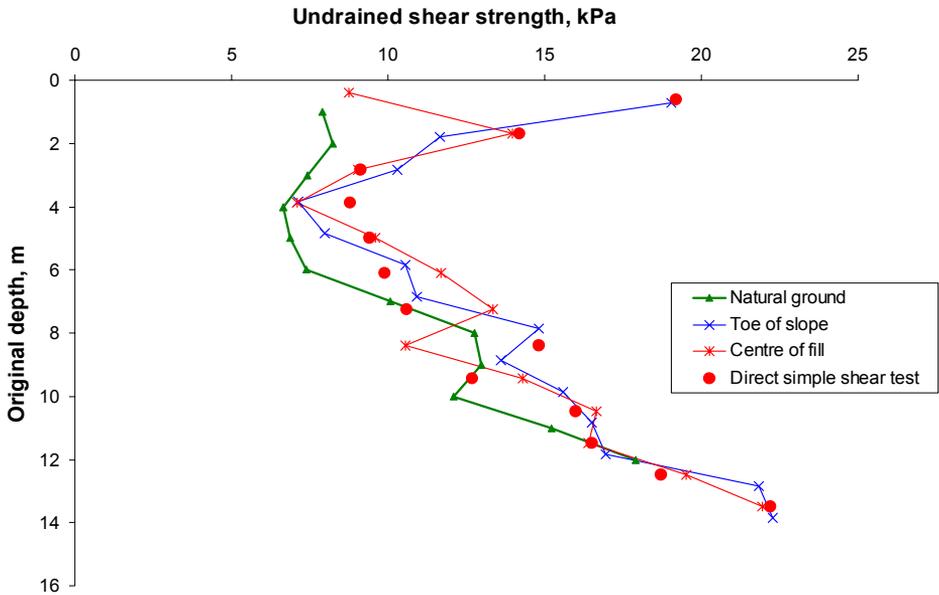
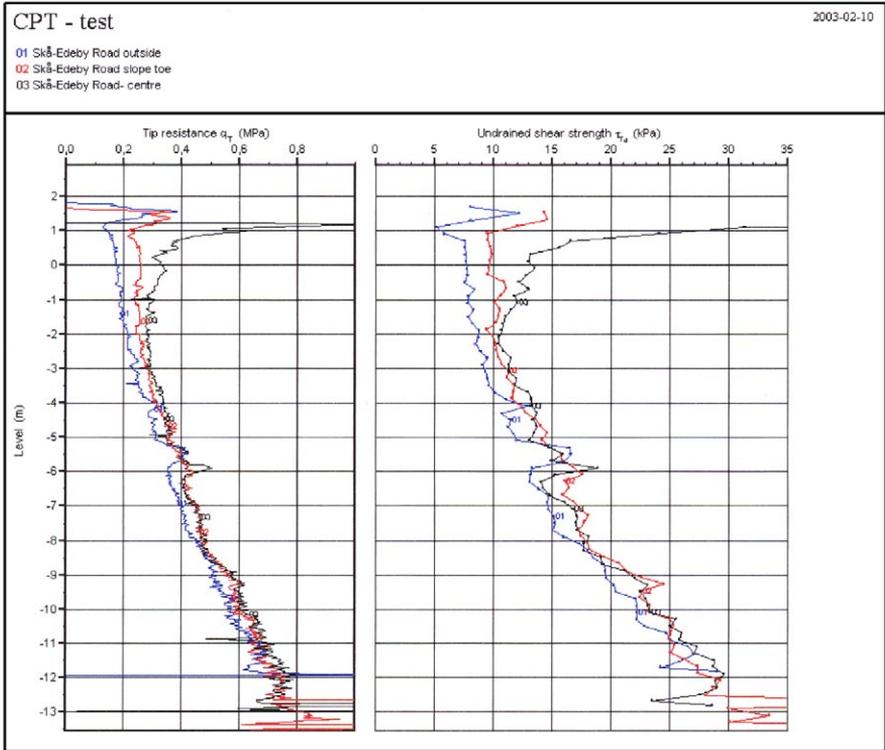


Fig. 37. Shear strength measured by field vane tests at the test embankment at Skå-Edeby.



**Fig. 38.** Results from CPT tests at various locations below and outside the test embankment at Skå-Edeby 2002. (It should be observed that these results are presented versus actual level and have not been corrected to original level as the other results)

# Chapter 4.

## Measurements with cross-hole tomography

### 4.1 BACKGROUND AND HYPOTHESIS

Measurement of the shear wave velocity in the field is often used to estimate the in-situ initial shear modulus  $G_0$ . This parameter is essential for prediction of vibrations and movements in connection with earthquakes, traffic vibrations and other sources of vibrations such as construction work. The most common methods of measuring the shear wave velocity in the ground are cross-hole measurements and down-hole measurements and nowadays also the seismic CPT test, which is a rational way of performing down-hole measurements. Also spectral analyses of surface waves have recently been tried for this purpose (e.g. Dahlin et al. 2001).

It has been shown that the shear modulus in cohesive soils can be expressed as a function of the undrained shear strength (e.g. Andréasson 1979, Larsson and Mulabdic 1991). In principle, it should therefore be possible to estimate also the undrained shear strength from the measured shear wave velocity. However, both the shear modulus and the undrained shear strengths are functions of the square of the shear wave velocity and accurate estimations of the shear strength thereby require very accurate measurements of the shear wave velocity.

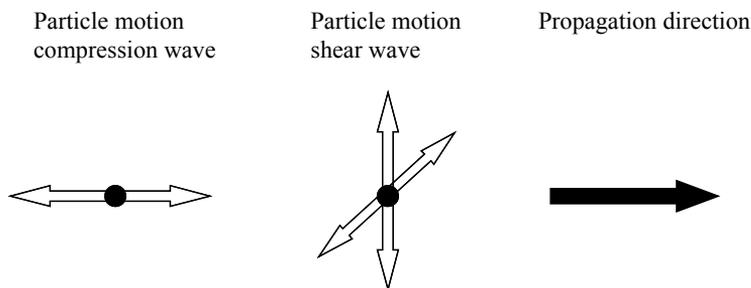
Furthermore, the relation between the undrained shear strength and the shear modulus, and thereby also the shear wave velocity, is also a function of the overconsolidation ratio (Andersen et al. 1988, Atkinson 2000). This also limits the possibility to estimate the undrained shear strength from the shear wave velocity. However, a prerequisite for considerable consolidation settlements and shear strength increases below embankments is that they have been built on normally consolidated or only slightly overconsolidated soft ground. The soil below the embankment will then remain in a normally consolidated or only slightly overconsolidated state also after the load application throughout the consolidation process and afterwards, unless a significant unloading is made. There should thus still be a possibility to estimate the undrained shear strength below such embankments from the shear wave velocity provided that this can be measured accurately enough.

This way of estimating the undrained shear strength has therefore been tried below the undrained test fills and embankment at Lilla Mellösa and Skå-Edeby. The primary aim of the investigations was to find out if the method is suitable for estimating the shear strength below railway embankments, where any tests involving intrusion on the embankment itself and interruption of the traffic should be avoided.

## 4.2 THE METHOD

### *Seismic waves*

Seismic investigations are based on the propagation of elastic waves in the ground (soil and bedrock). An elastic wave is generated e.g. during an earthquake, an explosion or when the ground is hit with a hammer. The propagation velocities of elastic waves are governed by the elastic properties of the ground, such as compressions modulus, shear modulus, density and Young's modulus (e.g. Kulhánek, 1993). Elastic waves in the ground are usually separated into compression waves, shear waves and surface waves, and looking at the motion of rock particles shows the distinction between the different wave types. Shear waves have a particle motion that is perpendicular to the direction of propagation while compression waves have a particle motion that is parallel to the direction of propagation, Fig. 39.



*Fig. 39. Particle motions of compression waves and shear waves with respect to the direction of propagation.*

The shear wave velocity,  $V_s$ , can be expressed in the following way:

$$V_s = \sqrt{\frac{G}{\rho}}$$

where  $G$  = shear modulus and  $\rho$  = density

If the shear wave velocity and the density are known it is possible to calculate the shear modulus according to:

$$G = V_s^2 \rho$$

Shear waves cannot propagate in fluid materials since the shear modulus of fluids equals zero.

The shear modulus that is measured in most seismic tests is the initial shear modulus at very small strains,  $G_0$ . The relation between the initial shear modulus and the undrained shear strength,  $c_u$ , in Swedish normally consolidated or only slightly overconsolidated clays has been found to be (Larsson and Mulabdic 1991)

$$G_0 \approx \frac{504 c_u}{w_L}$$

which yields

$$c_u \approx \frac{V_s^2 \rho w_L}{504}$$

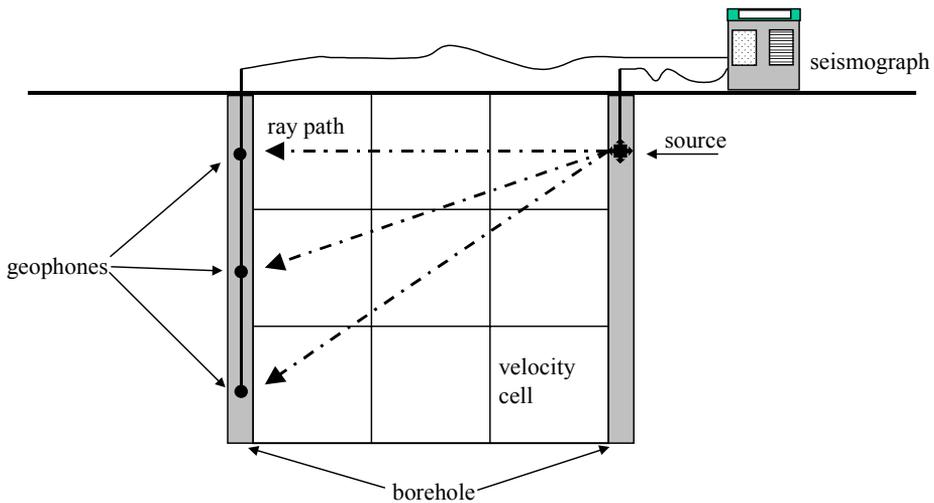
where  $w_L$  is the liquid limit.

This relation mainly refers to the velocity of horizontally polarised shear waves travelling in the vertical direction, but also waves with other polarisation and travel directions have been included in the basis for the correlation. The differences are assumed to be small in homogeneous normally consolidated and only slightly overconsolidated soils, (Butcher and Powell 1997). The undrained shear strength in the correlation refers to values obtained in corrected field vane tests and direct simple shear tests.

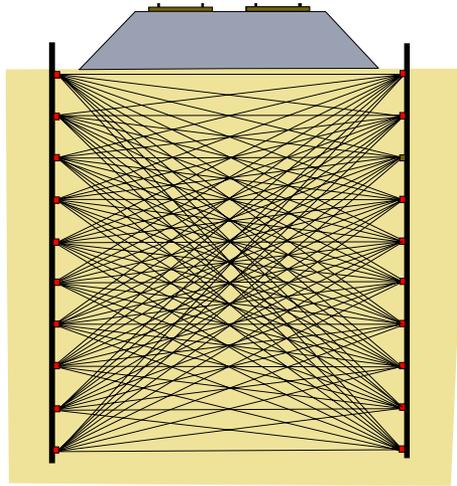
### ***Seismic tomography***

Tomography is a well-known technique in many branches of science to create images of projections (tomograms) of hidden objects by the use of X-rays, ultrasound or electromagnetic waves (tomo = slice, graph = picture). During the past few decades the use of tomography has become more common in the earth sciences, mainly in prospecting after oil and gas.

There are different kinds of tomographic measurement techniques and what was used in this project is termed *seismic crosswell direct wave traveltime tomography*. However, it is commonly called cross-hole tomography and will generally be called seismic tomography or just tomography in this report. The basic principle of the technique is to estimate a velocity model of the ground by measuring the time it takes for elastic waves to propagate from a source to a receiver. To perform seismic crosswell tomography measurements it is necessary to have two (or more) boreholes, Fig. 40. An array of geophones is inserted in one hole and in the other an elastic wave is generated. A seismograph measures the time it takes for the wave to propagate from the source point to the geophones. The source is then moved to another position in the hole and the procedure is repeated. The measurements will produce a number of arrival times of waves that have crossed the investigated area. The geophone distance and the wave frequency mainly govern the data resolution, : the shorter distance and the higher frequency, the better the resolution. The geometry of the investigated area, meaning the spatial relation between the depth of, and distance to, the boreholes is also an important parameter since shallow boreholes and a large distance will lead to poor ray coverage, Fig. 41.



**Fig. 40.** Schematic picture of instrument setup and ray paths during a seismic tomography measurement.



*Fig. 41. Schematic picture of the coverage of the wave paths. It should be observed that the coverage is relatively poor within vertical strips close to the boreholes and in zones midway between the boreholes at the upper and lower boundaries of the investigated section.*

The following paragraph presents a simplified explanation of the calculation procedure producing velocity models, or tomograms. For a detailed explanation see e.g. Nolet (1987). The measured first arrival times and the co-ordinates of the geophones and the source points are stored in a simple ascii data file. The area between the boreholes is divided into a grid of velocity cells, see Fig. 40. The size of the cells can be varied in any particular way, but it is seldom relevant to use a smaller size than the geophone distance. Each cell is assigned an initial start value. The model program then calculates the time it takes for different rays to travel through the area between the boreholes. The calculated times are compared to the measured travel times, and the errors in the calculations are the differences between these two parameters. Different rays intersect each cell and the best-fit velocity is estimated by the least squares method. The procedure is repeated for a predetermined number of iterations or until a chosen limit is reached, the so-called RMS-residual. A residual is defined as the difference between the measured travel time and the corresponding value calculated by the model. The RMS (Root Mean Square) residual is the square root of the mean of the squared residuals and indicates how well the model corresponds to the measured data. The calculated velocity model does not provide a unique solution to the inversion problem, but with information about the geological conditions at the site it is possible to determine if the established model is physically reasonable.

The fit of the model is thus indicated by the RMS value. The size of an acceptable RMS-value depends mainly on the measuring accuracy of the travel times for the shear waves. In this particular case, an RMS-value of up to 25 – 30 ms is considered to indicate a good model fit.

### ***Software for tomography inversion***

The software used in this project is called 3DTOM: Three-Dimensional Geophysical Tomography (Jackson and Tweeton 1996). 3DTOM is a DOS program developed for three-dimensional modelling in mines, but the program also performs two-dimensional calculations. It can be downloaded free of charge at the web site of USGS on <http://www.usgs.org/>. It makes no difference if one works with seismic compression, shear or with electromagnetic waves. For inversion of travel time data 3DTOM uses the SIRT method (Simultaneous Iterative Reconstruction Technique; Peterson et al. 1985). It is possible to model straight rays, crooked rays or combinations of these (hybrid modelling). The data analyses are performed with velocity histograms and by the use of stereographic projection plots that show the ray coverage. The start model can be varied between a homogenous, a horizontally layered or a checkerboard model. The calculated models are presented as contour plots in colour or in grey scale and there are functions that allow a comparison between different models (such as difference or quotient). Some simple filtering can also be performed.

### ***Field equipment and procedure***

The equipment for collecting the data consisted of a TERRALOC MARK 3 (ABEM) seismograph and three 5-component 28 Hz sensor geophones (BG-K5) with pneumatic clamping devices. The vibration source was an ordinary screw plate attached to a hollow drilling pipe. A free-running inner rod system was inserted into the pipes and was used to generate vertically polarised shear waves. The rods were then lifted and allowed to fall onto the top of the screw plate. The triggering of the seismograph was carried out by a standard geophone (PE-3) attached to the drill pipe at the upper end. The geophones were set up to detect vertically polarised shear waves and were mounted at a distance of 1.0 metre from each other.

The geophones were lowered in a vertical bore hole with a plastic casing and attached by inflating the pneumatic clamping devices. The casing consisted of a plastic bellows hose that is vertically elastic, which ensures good transmission of the signal from the clay to the geophones. In the first measurement position the uppermost geophone was put as close as possible to the ground surface, the second

and third geophones were then positioned at about 1 and 2 metres depth respectively. The screw plate was screwed 0.1 – 0.2 metres down into the ground. The trigger geophone was attached to the pipe and a measurement was performed. The set of geophones were then lowered 3 metres and a new measurement was performed. When the geophone array reached the bottom of the borehole, the screw plate was advanced down to one metre depth and the procedure was repeated with the geophones instead being lifted in 3-metre stages. This was repeated with the screw plate being advanced in 1-metre steps until it reached firm ground, which generally occurred at the same depth as the bottom level in the geophone hole.

Since the trigger geophone was mounted at the top of the drill pipe a time delay was introduced because the wave had to travel up along the pipe before it reached the geophone. The delay increases linearly with depth of the source point. A calculation shows that the maximum size of the delay is 2.4 ms. Each data set was corrected for the delay before the tomographic modelling started.

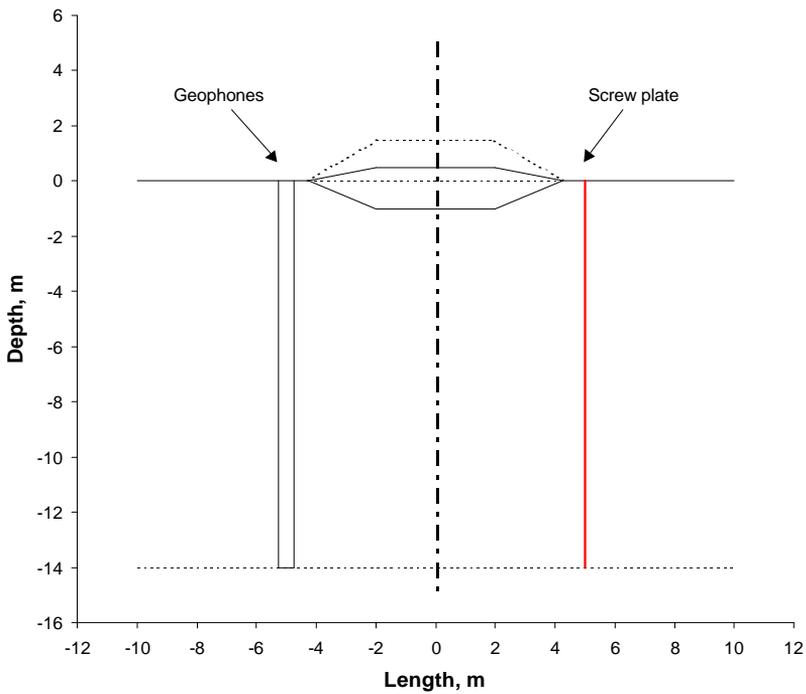
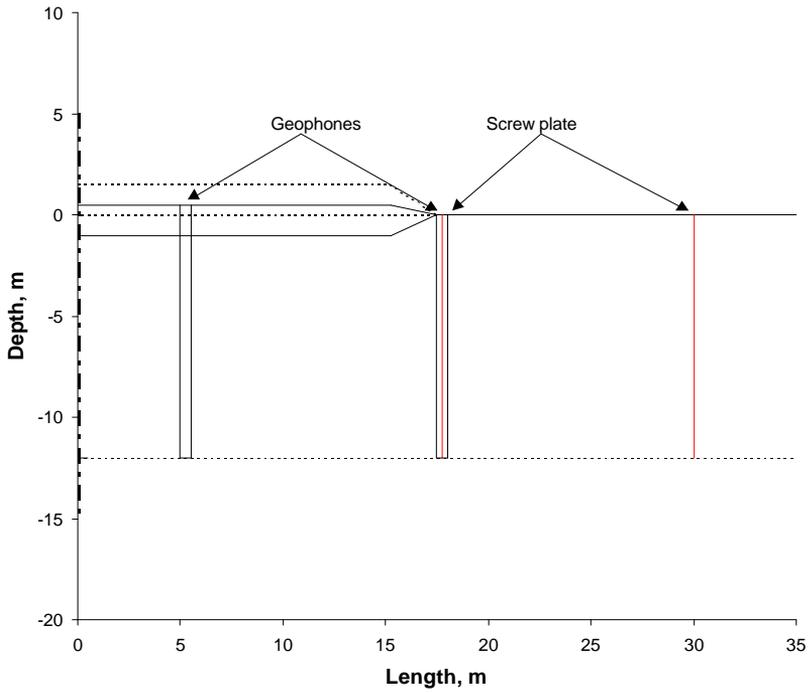
### **4.3 RESULTS**

Three tomography measurements were performed at the test site in Skå-Edeby and two at the site in Lilla Mellösa. In Skå-Edeby one measurement was performed beneath the circular fill, with one borehole positioned in the central part but well outside the permanent instrumentation and the other at the perimeter of the fill. The second measurement was performed in the natural soil outside the fill and the third measurement was performed across the road-like embankment, [Fig. 42 a and b](#). In Lilla Mellösa one measurement was performed beneath the square fill, from the outer border to the central part, and one in the natural soil outside the embankment, [Fig. 42c](#).

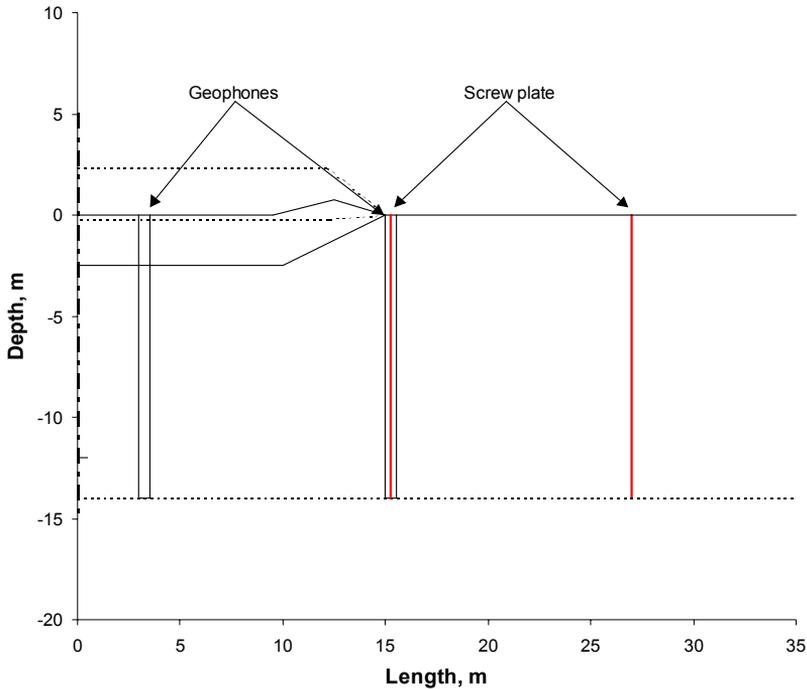
The distance between the boreholes varied between 9 and 15 metres and the depth to solid ground varied in approximately the same way, which resulted in fairly square geometries and a ray-coverage of about  $0^{\circ} - 45^{\circ}$ .

#### ***Data analysis***

Picking the first arriving shear wave was performed continuously during the measurements with an inbuilt function in the seismograph. The data quality was generally high and most often it was easy to identify both direct compression and shear waves. Problems with noisy data occasionally occurred when the geophones were positioned near the ground surface, most likely caused by bad coupling between the geophones and the dry crust. The velocity of the direct compression



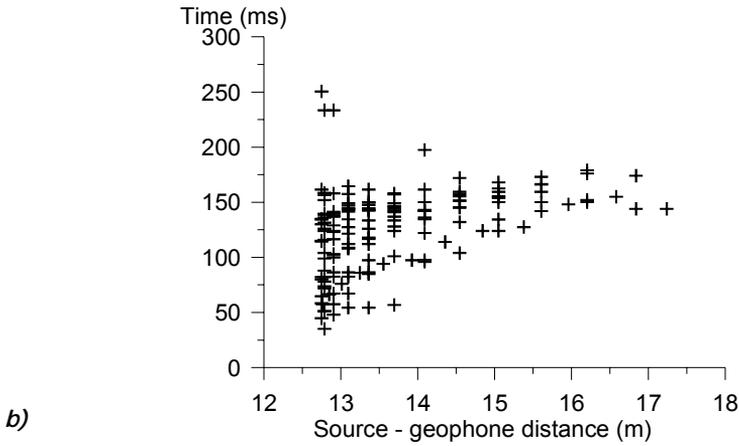
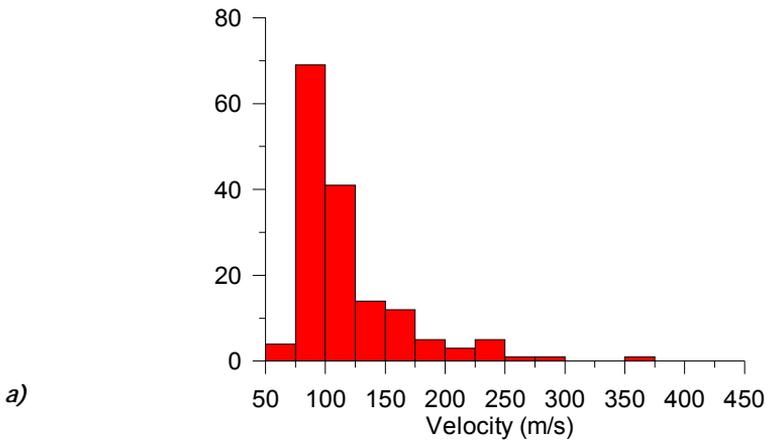
**Fig. 42.** Schematic layout of the tomography measurements.  
 (Red lines show the locations of the boreholes for the screw plate)  
 a) At the circular test fill at Skå-Edeby  
 b) At the test embankment at Skå-Edeby



**Fig. 42.** *Schematic layout of the tomography measurements.  
(Red lines show the locations of the boreholes for the screw plate)  
c) At the square test fill at Lilla Mellösa*

wave is estimated at 1400-1500 m/s (straight ray approximation). The average velocity of the shear waves in the clay is about 80-100 m/s and the frequencies range from 40 Hz to 85 Hz, which corresponds to wavelengths of 0.9 – 2.5 metres.

Velocity histograms (straight ray approximation) and time-distance graphs help to indicate the scatter in the data and to identify obvious outliers. An example is shown in Fig. 43 with data from the circular fill at Skå-Edeby. The histogram in Fig. 43a shows an approximately log-normal distribution, slightly skewed towards higher velocities, and a few anomalous values of higher velocities. The time-distance plot in Fig. 43b indicates a continuous increase in time with increasing distance between source and geophone. The scatter in the data is fairly high at short distances and decreases with increasing distance. Three outliers fall close together in the upper left part of the diagram. The data from the other measurements look very much the same as in Fig. 43. The anomalous velocities may be caused by badly picked arrival times, but may also originate from true velocity differences.



**Figure 43.** Tomography data from the Skå-Edeby circular test fill.  
 a) Histogram showing the shear wave velocity distribution.  
 b) First arrival times plotted against the distance between the source and the geophones.

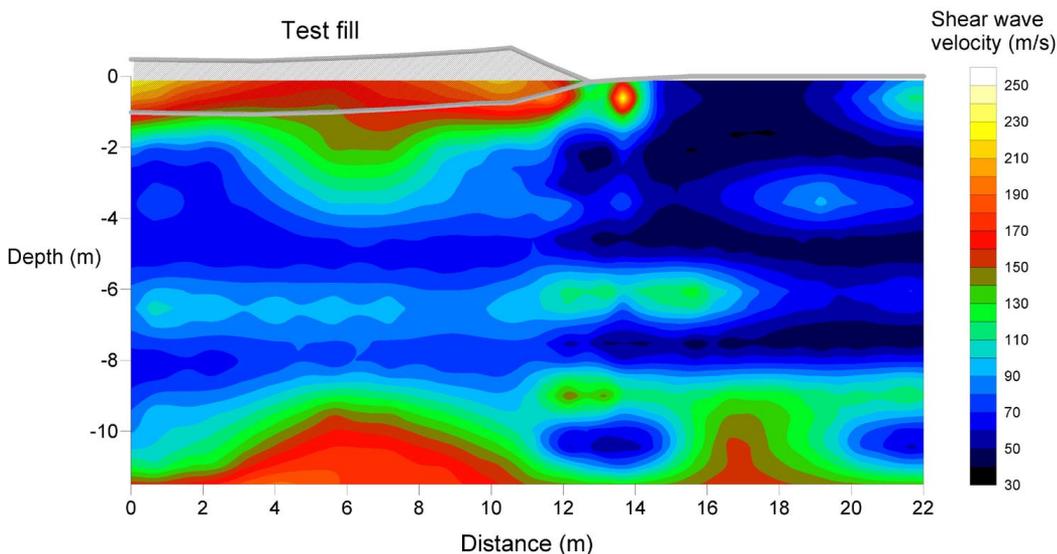
## *Tomography inversion*

The first step in the data processing was to use straight ray inversion, which is the simplest and fastest technique. This gives a good overview of velocity contrasts, data resolution capacity and it indicates unreasonable velocity anomalies. Each dataset was then modelled using crooked ray paths, which is the most time-consuming technique but also the most accurate. The working principle was to use raw data and not to clean the data sets from “erroneous” time readings. Differences are very small between models based on raw and on “cleaned” data. Since the soils at both sites have distinct layering, horizontally layered starting models were used. If a homogenous starting model is used instead, the velocity anomalies will be more vertically elongated, but the general velocity distribution is not significantly affected.

Each data set was modelled separately, which resulted in five different tomograms. The measurements of undisturbed soil were carried out in direct connection to, and along the same line as, the measurements beneath the circular test fill at Skå-Edeby and the square test fill at Lilla Mellösa. The models from undisturbed and disturbed soil were therefore put together and are presented as one single contour plot of each area. The model of the undisturbed soil at Skå-Edeby has also been added on each side to the test embankment model to provide comparison possibilities.

### *Skå-Edeby*

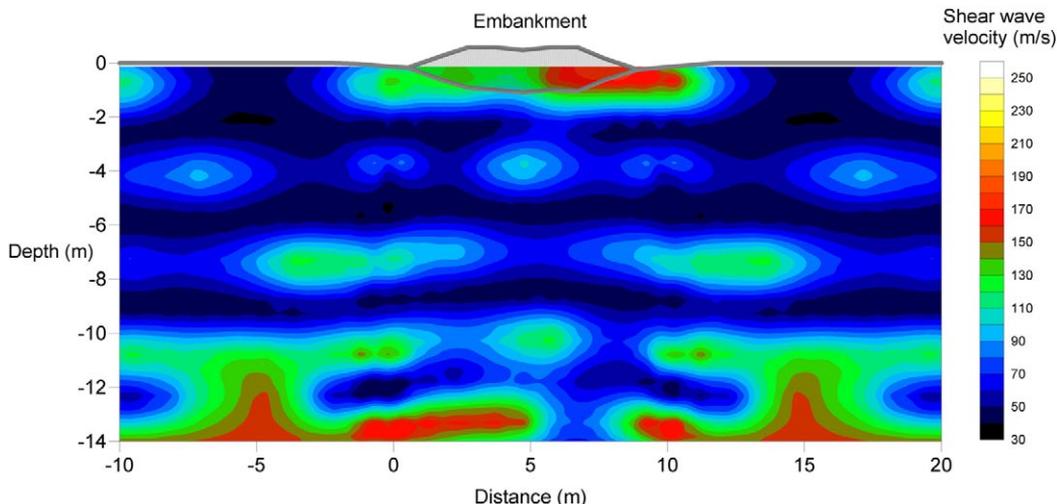
In [Fig. 44](#), the tomograms from beneath the circular fill and of the natural soil outside are presented. The hatched area indicates the present position of the fill. Note the subsidence of about 1 metre. There is a large velocity contrast between the gravel fill and the clay soil. Velocities of 40 – 60 m/s dominate in the uppermost 2 metres of the natural soil whereas the subsided part of the fill shows velocities ranging from 100 m/s to 250 m/s. Below 2 metres and down to about 5 metres' depth there are still considerably higher velocities under the test fill than in the undisturbed soil, with about 20 – 30 m/s higher velocities below the fill. At 6 – 7 metres' depth there is a horizontal high-velocity sub-layer that cuts across the entire soil section. Below this layer, from 7 to 9 metres' depth, the shear wave velocity is still higher beneath the test fill than in the natural soil outside. When the bottom of the clay layer is reached (at about 11 metres' depth according to the CPT tests) the velocity rapidly increases from 60 – 90 m/s to 120 – 180 m/s, which indicates that a stiffer material underlies the clay. The dome-shape of this layer is most likely an artefact created during the model inversion, which is caused by a combination of a fast velocity increase and a lack of data related to the bad ray coverage close to the boundary. The same effect can be seen below the fill material and at the lower



**Fig. 44.** *Contour plot of the tomograms from the circular embankment and natural soil at the Skå-Edeby site.*

boundary of the section in natural soil. The circular anomalies appearing along a vertical line at the 13 m distance are also caused by the lack of coverage and true data, but are here due to the borehole being situated in this position. The RMS value of the test fill model is 21ms and for the undisturbed soil it is 25ms.

The cross-section below the road-like test embankment at Skå-Edeby was measured with a distance between the boreholes of 9.45 metres and a depth of 13.5 metres, which gives a favourable geometry. In Fig. 45 the model of undisturbed soil is attached on each side of the embankment model. The high-velocity layer in the top metre ( $V_s = 120 - 180$  m/s) under the embankment is caused by the subsided embankment. Between 1 and 1.5 metres' depth there is a rather sharp transition and the velocity decreases down to 50 – 60 m/s. At about 4 metres' depth there is a thin high velocity layer ( $V_s = 90 - 100$  m/s), which is followed by a low velocity layer. These layered variations, which are indicated in the natural soil as well, are repeated twice before the bottom of the soft soil is reached at 12 – 13 metres depth. The RMS value of 39 ms is fairly high and it indicates some problems in fitting the model data to the measured travel times.



**Fig. 45.** *Contour plot of the tomogram from the road-like test embankment and natural soil at the Skå-Edeby site.*

### ***Lilla Mellösa***

The results of the measurements at Lilla Mellösa are shown in Fig. 46. They greatly resemble those from the circular embankment at Skå-Edeby. The RMS values are low, for the embankment model it is 16 ms and for the natural soil it is 10 ms, which indicates that the models fit statistically well to the measured data. At Lilla Mellösa the test fill has subsided about 2 metres down into the soft soil. This is clearly seen as a high-velocity layer ( $V_s = 150 - 230$  m/s) in the uppermost 2 metres beneath the ground surface. In the natural soil outside the embankment the velocities are significantly lower, 60 – 80 m/s in the upper layers. In the upper part of the soil profile beneath the surface the shear wave velocity is significantly higher under the embankment than in the natural soil. The average velocity contrast is about 40 m/s. In the lower part of the profile the soil below the embankment and the natural soil show similar velocity variations, but the layers are displaced downwards below the embankment all the way down to the firm bottom at about 13 metres' depth. The boundary between the soft soil and the bottom is not as well-defined at the Lilla Mellösa site as it is at the Skå-Edeby site. At Lilla Mellösa the clay layers are underlain by sand whereas those at Skå Edeby rest on rock or till. The circular anomalies appearing along a vertical line at 10 – 12 metres' distance are caused by the lack of coverage and true data due to the borehole being situated in that position.

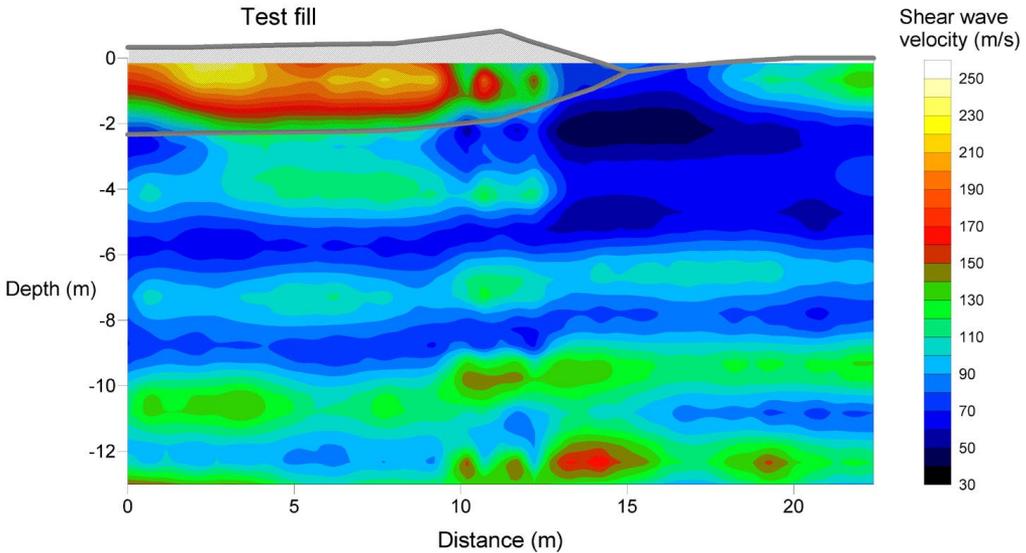


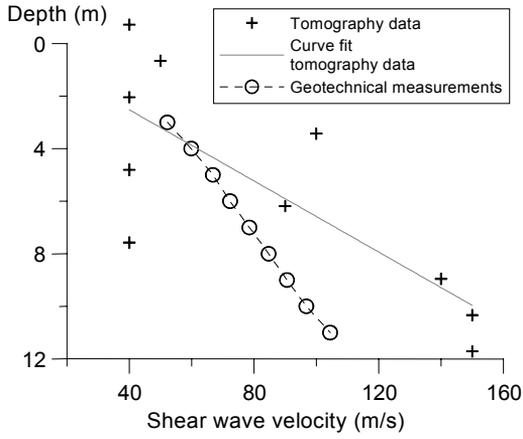
Fig. 46. Contour plot of the tomograms from the square test fill and natural soil at Lilla Mellösa.

#### 4.4 DISCUSSION OF THE RESULTS

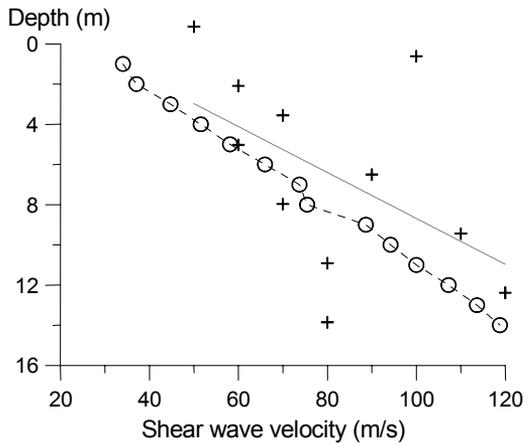
All measurements are connected with some uncertainty and in the case of cross-hole tomography this is enhanced by the lack of coverage in parts of the sections. A specific figure for the uncertainty in the velocity determination is difficult to present, but the standard deviations of the mean velocities in the natural soils are one way of obtaining estimates. At Skå-Edeby the standard deviation in unloaded soil is 19 m/s and at Lilla Mellösa it is 15 m/s. Below the test fills it is of approximately the same sizes. The data from the road-like embankment at Skå-Edeby differ from the other datasets and the large RMS value of 39 ms makes the interpretation of these data more uncertain.

A reliability control of the tomography models was performed by comparing the estimated velocities in natural soil to velocities from these sites based obtained from seismic cone measurements by Larsson and Mulabdic' (1991). The comparison indicates that similar values are obtained by the two techniques, although the average model velocities are slightly higher, Fig. 47. The scatter in the model data is high, but both examples indicate linearly increasing velocities with depth and the modelled and calculated velocities are roughly of the same size. Much of the scatter and the difference may be explained by the anomalies in the parts of the sections with poor coverage in the cross-hole tomography.

*Lilla Mellösa*



*Skå-Edeby*



**Fig. 47.** Comparison between shear wave velocity in natural soil from tomography data and estimated from seismic CPT tests.

## Chapter 5.

# Correlation between the evaluated results and measured increases in shear strength

From a first glance at the tomograms in [Figs 44–46](#) it is obvious that a considerable increase in shear wave velocity and undrained strength has occurred below the circular and square test fills in Skå-Edeby and Lilla Mellösa. It also tells that any such increase below the road-like test embankment at Skå-Edeby is considerably smaller and limited to the upper layers below the embankment.

When making a more detailed evaluation, certain aspects have to be taken into account. These are

- All values close to the boreholes are more or less faulty because of poor wave path coverage and thereby misleading.
- All values at the upper and lower boundaries of the section in a part midway between the boreholes are more or less faulty if the soft soil is over and underlain by considerably stiffer material. Even when this is not the case, the values are uncertain because of poor coverage in these parts.
- The relation between undrained shear strength and shear wave velocity is sensitive to the liquid limit of the soil. This has to be taken into account in profiles where this property varies, i.e. in almost every case.

Before evaluation, the data in the vertical strips with poor coverage close to the boreholes should be excluded. The width of these strips can be estimated by a sketch of the wave paths for the actual distances between the boreholes and depths between the measuring points, see [Fig. 41](#). A similar estimate of uncertain zones at the upper and lower boundaries should also be performed together with a judgement of how much of any dome-shaped bulbs below the fills and at the firm bottoms are caused by the waves taking crooked paths through these stiffer layers.

Since no information on distribution of liquid limit below the loaded area at the present conditions is normally available, this has to be estimated from the data in the natural soil outside or investigations performed before the application of the load together with an estimate of the distribution of settlements with depth. The estimation of the total settlements below the fills and the embankment is fairly straightforward since the border between the fill material and the underlying clay is rather distinct. The estimation of the distribution with depth is more crude and has to be made with consideration for how the levels of different layers and stiffness borders below and outside the embankments are located in relation to each other. From the visual inspection of the tomograms below the large fills at Skå-Edeby and particularly at Lilla Mellösa it is quite obvious that downward movements have occurred throughout the clay profiles below the fills. A more detailed distribution is difficult to interpret but an assumption of an even distribution of the compression with depth appears to be reasonable and will have to suffice. The estimation of the settlement distribution with depth below the road-like embankment is more difficult. However, the tomogram clearly indicates that the embankment has settled about 1 metre and that there should not be any significant settlements below 6-7 metres' depth. A rough estimation is that the settlements are evenly distributed down to this depth.

The next step is to draw enlarged and more detailed tomograms with clear and fairly closely spaced contour lines for the shear wave velocities. For the tomograms of the natural soil, horizontal lines are drawn at selected evenly spaced depths. The zones with estimated erroneous data are excluded and the average velocity at these depths within the remaining zone is estimated. Some consideration may have to be taken also for the geometry of the loading when selecting relevant data in the upper layers, but this is not important at geometries and borehole locations such as those used in this investigation. The zones below the slopes and just outside these have here already been excluded because of poor coverage at the boreholes at the toes.

The same procedure is used for the tomograms below the loaded areas, but here the horizontal lines are adjusted with consideration for the estimated settlements at each level in such way that lines corresponding to the original depths are produced. The estimated average velocities at the "original" depths can then be compared with a certain confidence that it is the same type of material that is involved and that the comparison thereby is relevant.

There are then two ways of estimating the undrained shear strength below the loaded areas. The first is to use the empirical relation between shear wave velocity,

density and liquid limit and the undrained shear strength. The estimated shear wave velocity below the fill is then used together with density and liquid limit at the original depth. At large settlements there is also a certain increase in density but this is of limited importance.

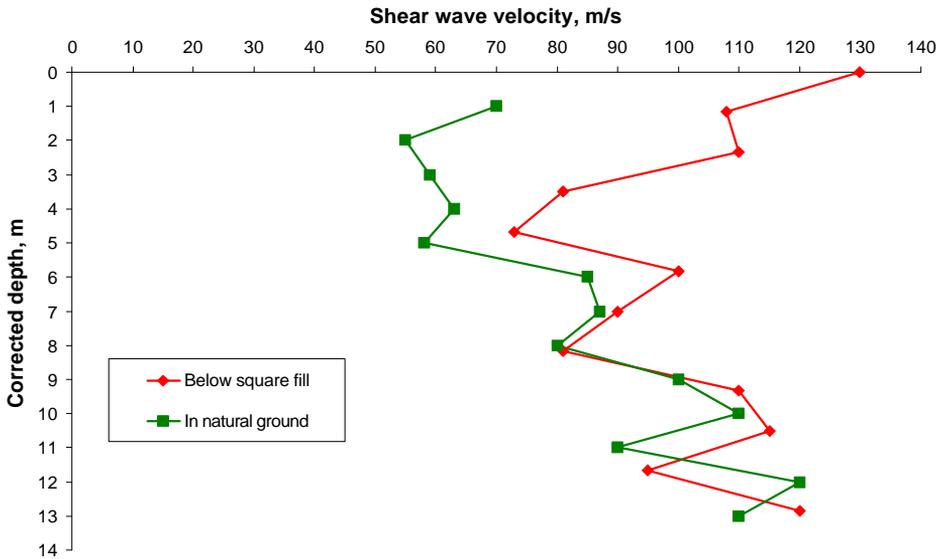
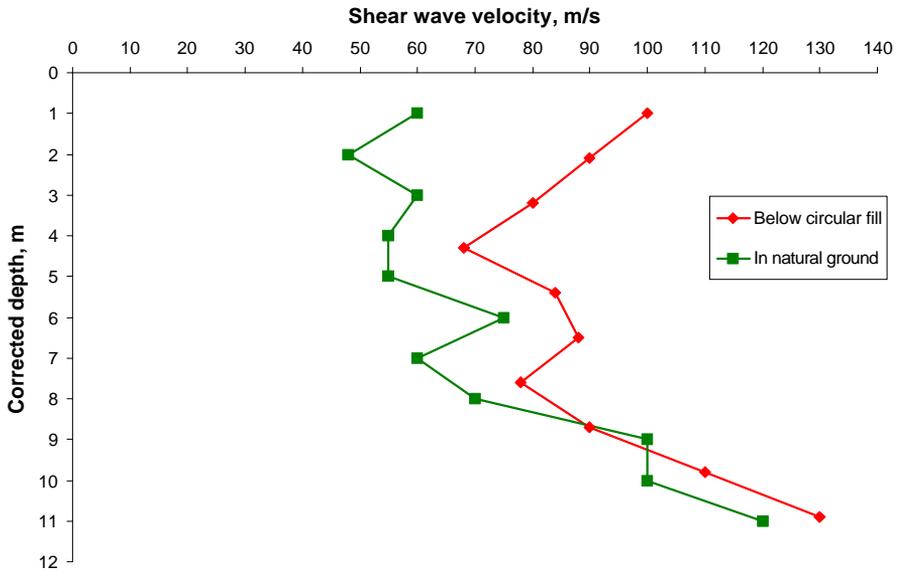
However, this calculation presumes that the particular soil follows the empirical relations and there is always a certain spread in such relations. A more direct way is to use the undrained shear strength measured in the natural ground outside the loaded area. The shear wave velocities at each depth are then compared to velocities corresponding to the same original depths below the loaded area. The undrained shear strength below the loaded area is then calculated as the undrained shear strength in the natural soil multiplied by the square of the quotient between the shear wave velocities

$$c_{u2} = c_{u1} \left( \frac{V_{s2}}{V_{s1}} \right)^2$$

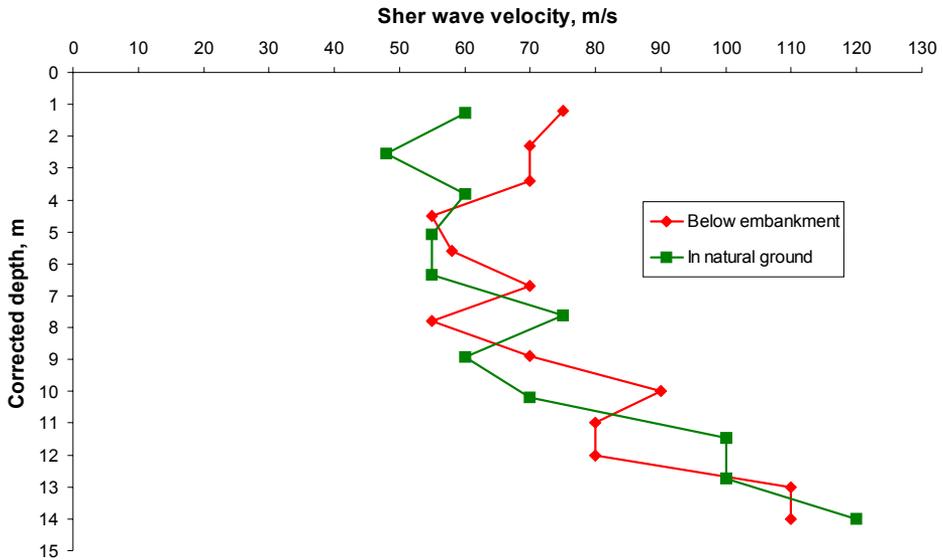
where  $c_{u2}$  = undrained shear strength below loaded area  
 $c_{u1}$  = undrained shear strength in natural ground  
 $V_{s2}$  = shear wave velocity below loaded area  
 $V_{s1}$  = shear wave velocity in natural ground

Both methods have been applied to the measured data.

The evaluated shear wave velocities in natural ground and below the loaded areas are shown in Fig. 48 a-c. The results scatter but the patterns suggest that an increase in shear wave velocity has probably occurred throughout the profiles below the large fills. This increase is large at the top but decreases with depth to become almost zero at the bottom. At the fill in Skå-Edeby the increase in the bottom layers can be considered to be verified whereas it is only hinted at below the fill in Lilla Mellösa. Below the narrow road embankment at Skå-Edeby, there is a significant increase in velocity down to 4 metres' depth. The effect probably reaches down to 6 – 7 metres' depth, but the anomaly found in the shear strength determinations by the field vane tests at 4 metres' depth is found also in the shear wave velocities. Below 6 – 7 metres' depth no increase in shear wave velocity is even hinted at.



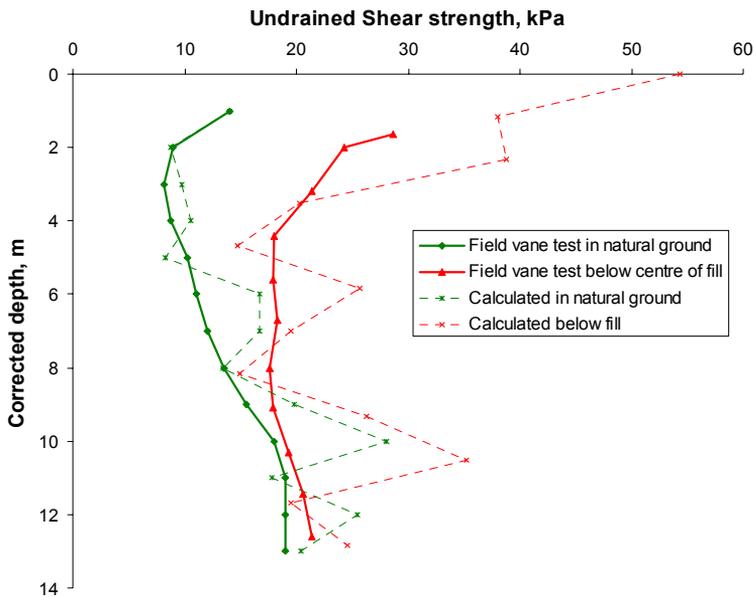
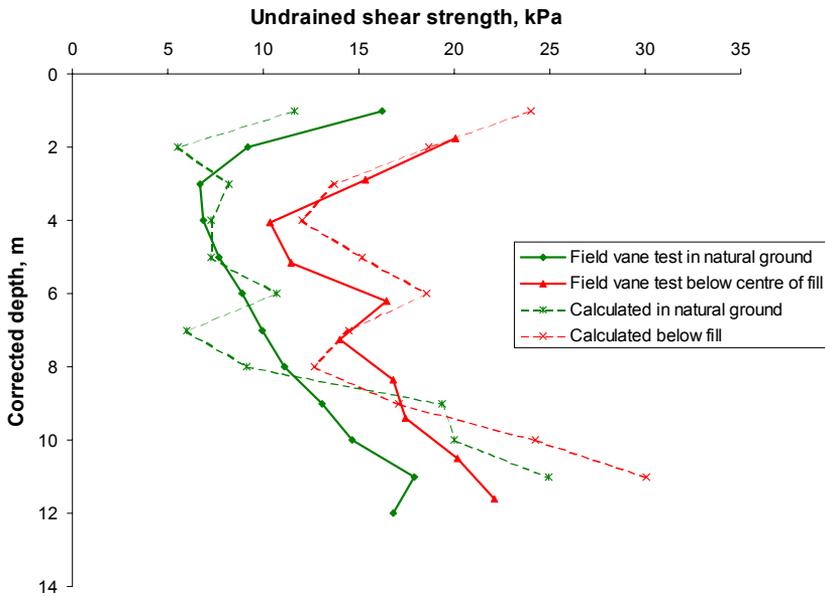
**Fig. 48.** Evaluated shear wave velocities with the levels below the loaded areas corrected for the estimated settlements.  
 a) At the circular fill at Skå-Edeby  
 b) At the square fill at Lilla Mellösa



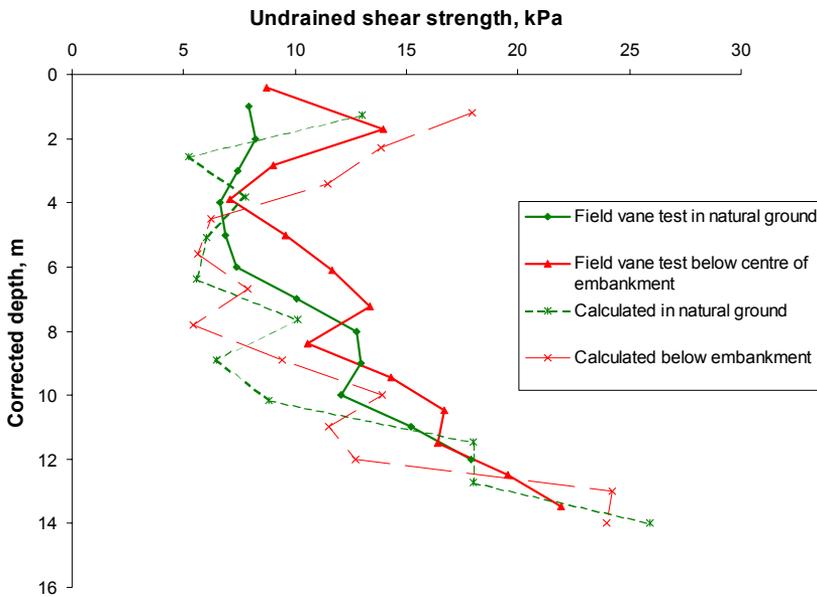
**Fig. 48.** Evaluated shear wave velocities with the levels below the loaded areas corrected for the estimated settlements.  
c) At the test embankment at Skå-Edeby.

The corresponding undrained shear strengths calculated by the empirical relation  $c_u = V_s^2 \cdot \rho \cdot w_L / 504$  are shown in Fig. 49 together with the undrained shear strengths evaluated from the field vane tests. The results show that although the undrained shear strength calculated thus are of roughly the right size, the estimation is often too coarse to be used to accurately evaluate the absolute shear strength values that are sought.

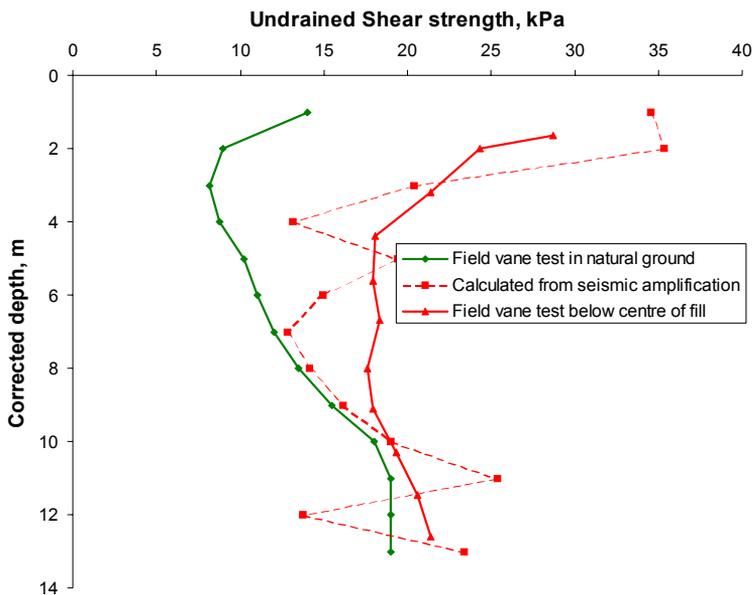
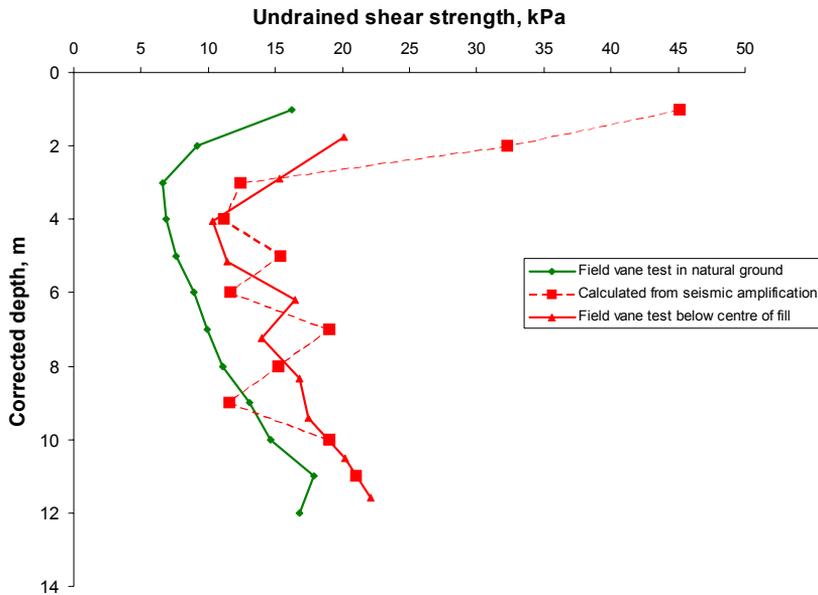
The undrained shear strength calculated from the measured shear strengths in the field and the amplification  $(V_{s2} / V_{s1})^2$  estimated from the measured shear wave velocities are shown in Fig. 50. This method provides a better estimate of the real shear strength. The amplification is the same but the base values are more relevant provided that a good estimate of the undrained shear strength in the natural ground has been obtained by the field vane tests.



**Fig. 49.** *Evaluated undrained shear strength using the empirical correlation. The levels below the loaded areas corrected for the estimated settlements.*  
 a) *At the circular fill at Skå-Edeby*  
 b) *At the square fill at Lilla Mellösa*

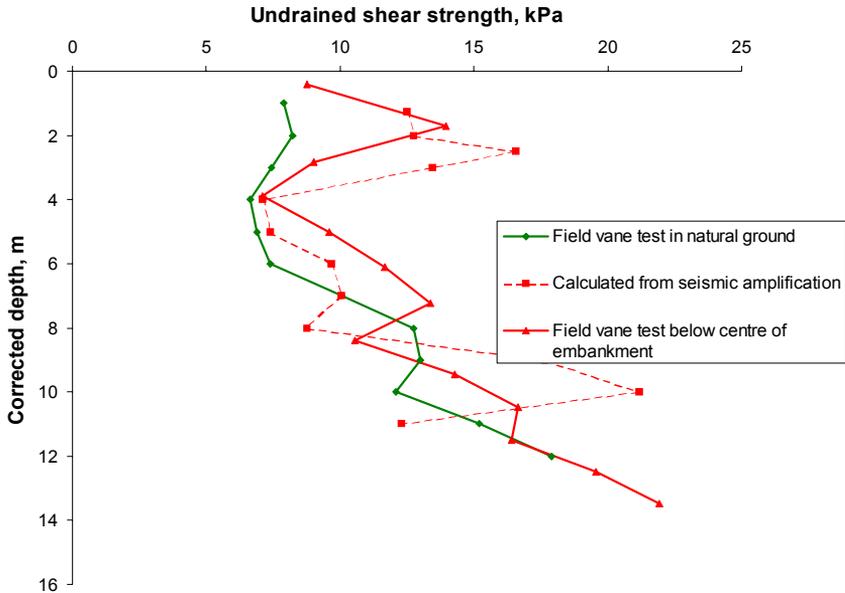


**Fig. 49.** *Evaluated undrained shear strength using the empirical correlation. The levels below the loaded areas corrected for the estimated settlements. c) At the test embankment at Skå-Edeby*



**Fig. 50.** *Evaluated undrained shear strength using the undrained shear strength in natural ground and the amplification estimated from the measured shear wave velocities. The levels below the loaded areas corrected for the estimated settlements.*

- a) At the circular fill at Skå-Edeby*
- b) At the square fill at Lilla Mellösa*



**Fig. 50.** *Evaluated undrained shear strength using the undrained shear strength in natural ground and the amplification estimated from the measured shear wave velocities. The levels below the loaded areas corrected for the estimated settlements.  
c) At the test embankment at Skå-Edeby*

The results of the seismic cross-hole tomography have thereby been shown to provide a good general picture of the shear strength below the embankments and also fairly good estimates of the actual sizes of the shear strengths. The results scatter somewhat, but the general pattern of the shear strength variation is obtained and an estimate of the operative strength on the basis of the tomographs would come out fairly close to the actually measured values.

The comparison has been made versus shear strength in the direct shear mode. This is the shear strength that is normally considered to be most relevant for embankment stability, (e.g. Zdrakovic et al. 2002). It may be advocated that the active shear strength is more relevant for most of the part of a potential slip surface in a stability calculation that runs below the embankment. However, the shear strengths in active shear and direct shear are proportional to each other. The amplification factor would therefore be the same and the active shear strength can be estimated in the same way as the direct shear strength provided that the active shear strength in the natural ground is known.

## Chapter 6.

# Demand for development

The method of cross-hole tomography has been shown to be a useful tool for estimation of shear strength below embankments in principle. However, in order to make the method a more rational tool, there are some things that should be improved.

In this project, only readily available equipment was used. The original intention had been to use slender plastic tubes in the boreholes for the geophones. Such tubes are fairly cheap and easy to install. However, the measurements should then preferably be made with horizontally orientated shear waves. No source for such waves that could be inserted to different depths in the ground was found and vertically orientated waves were therefore used. This meant that vertically flexible bellows hoses had to be used for the geophones. Such hoses are both expensive and laborious to install and also require a certain time to become firmly fixed in the surrounding soil. A prerequisite for the method to become rational is therefore that a suitable source for horizontally polarised shear waves is found that can also be lowered into slender plastic pipes.

In the evaluations, certain problems have been encountered in connection with large differences in stiffness between the fill material and the soft ground below as well as between the soft soil and underlying till or bedrock. In order to better account for sharp velocity contrasts within the investigated media, more sophisticated commercial computer software for tomography inversion should be evaluated.

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**RAPPORT/REPORT****No****År/Year**

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**RAPPORT/REPORT****No****År/Year**

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**RAPPORT/REPORT****No****År/Year**

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