Two Stage-Constructed Embankments on Organic Soils.

- Field and laboratory investigations
- Instrumentation
- Prediction and observation of behaviour

WOJCIECH WOLSKI
ALOJZY SZYMANSKI
JOZEF MIRECKI
ZBIGNIEW LECHOWICZ

ROLF LARSSON
JAN HARTLÉN
KAZIMIERZ GARBULEWSKI
ULF BERGDAHL

LINKÖPING 1988
Two Stage-Constructed Embankments on Organic Soils.

- Field and laboratory investigations
- Instrumentation
- Prediction and observation of behaviour

WOJCIECH WOLSKI
ALOJZY SZYMANSKI
JOZEF MIRECKI
ZBIGNIEW LECHOWICZ

ROLF LARSSON
JAN HARTLÉN
KAZIMIERZ GARBULEWSKI
ULF BERGDAHL

LINKÖPING 1988
PREFACE

This report describes the results from the soils investigations, the construction and the predicted and observed behaviour of two stage-constructed test embankments on organic soil at the Antoniny test site in North-Western Poland.

This project has been carried out jointly by the Department of Geotechnics at the Agricultural University of Warsaw (DG) and the Swedish Geotechnical Institute (SGI). The aim of the collaboration was to combine the resources and experience at DG in the construction of embankments on organic soils and the experience and capabilities at SGI in investigation and instrumentation in very soft soils.

The test embankments constitute part of a large investigation of construction of dykes on organic soils carried out by DG on commission by the Polish Ministry of Agriculture. They also constitute part of a larger research project concerning construction of roads on organic soils carried out by SGI on commission by the Swedish National Road Administration. Results from the Antoniny site have also been incorporated in a research project concerning the engineering properties of organic soils and their determination carried out at SGI. The latter project has been sponsored by the Swedish Council for Building Research and the Swedish National Road Administration.

The construction and observation of the test embankments have mainly been conducted by staff from the DG and the instrumentation of the test sites has mainly been performed by field engineers from SGI. Field and laboratory tests, as well as calculations of stability and deformations, have been performed by both parties.

The project has been supported by internal funds of DG and SGI and by the Polish construction company WZIR Pita. Prefabricated drains were supplied by courtesy of Terrafigo.

The authors especially wish to express their gratitude to Lars Blomqvist, Eugeniusz Koda and Wojciech Sas for their excellent work in the field.

Warsaw and Linköping in september 1987

The Authors
TABLE OF CONTENTS

SUMMARY ........................................................................................................... 7

NOTATIONS AND SYMBOLS .............................................................................. 9

1. INTRODUCTION ............................................................................................. 11

2. DESCRIPTION OF THE SITE AND THE TEST EMBANKMENTS .................. 12
   2.1 Location and description of the test area .............................................. 12
   2.2 Method of construction .......................................................................... 13
   2.3 Monitoring equipment ........................................................................... 17
       2.3.1 Equipment for measurement of settlements .................................. 18
       2.3.2 Equipment for measurement of horizontal movements ............... 21
       2.3.3 Equipment for pore pressure measurements ............................... 22
       2.3.4 Location of the monitoring equipment .......................................... 24

3. SITE INVESTIGATIONS .................................................................................... 27
   3.1 Soundings and samplings ....................................................................... 27
   3.2 Field vane tests ...................................................................................... 31
   3.3 Cone penetration tests and pore pressure soundings ............................ 38
   3.4 Pore pressure observations ................................................................... 42

4. LABORATORY TESTS ...................................................................................... 45
   4.1 Deformation and consolidation characteristics .................................... 50
   4.2 Shear strength characteristics ............................................................... 57
   4.3 Yield envelope ......................................................................................... 64
   4.4 Geodrain tests ......................................................................................... 66

5. GEOTECHNICAL CONDITIONS ..................................................................... 71
   5.1 Stress conditions .................................................................................... 71
   5.2 Shear strength ....................................................................................... 72
   5.3 Compressibility ...................................................................................... 73

6. OBSERVATIONS OF THE TEST EMBANKMENTS ...................................... 74
   6.1 Deformations .......................................................................................... 74
   6.2 Pore pressures ......................................................................................... 88
   6.3 Shear strength increase .......................................................................... 94
   6.4 Influence of vertical drains .................................................................... 103
<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.</td>
<td>PREDICTION OF DEFORMATION AND STABILITY</td>
<td>104</td>
</tr>
<tr>
<td>7.1</td>
<td>Prediction of deformations and course of consolidation</td>
<td>104</td>
</tr>
<tr>
<td>7.1.1</td>
<td>General</td>
<td>104</td>
</tr>
<tr>
<td>7.1.2</td>
<td>Predictions for the embankments at Antoniny</td>
<td>108</td>
</tr>
<tr>
<td>7.2</td>
<td>Stability analyses for embankments on soft soils</td>
<td>134</td>
</tr>
<tr>
<td>7.2.1</td>
<td>General</td>
<td>134</td>
</tr>
<tr>
<td>7.2.2</td>
<td>Predictions of increase in shear strength due to consolidation</td>
<td>138</td>
</tr>
<tr>
<td>7.2.3</td>
<td>Predictions of stability for the embankments at Antoniny</td>
<td>141</td>
</tr>
<tr>
<td>8.</td>
<td>SUMMARY AND CONCLUSIONS</td>
<td>147</td>
</tr>
<tr>
<td>9.</td>
<td>REFERENCES</td>
<td>156</td>
</tr>
</tbody>
</table>
SUMMARY

Two test embankments have been built on top of 8 metres of very soft organic and calcareous soils. The embankments have been built in stages and the increase in shear strength due to consolidation in the different stages has successfully been utilized in the construction of the subsequent stages. Vertical prefabricated drains were installed under one of the embankments.

A comprehensive programme of field and laboratory test was carried out before the construction of the embankments. A large amount of monitoring equipment was also installed in the ground under and outside the embankments. The behaviour of the embankments in terms of settlements, horizontal displacements and pore pressures has been followed and the changes in soil properties have been measured. The behaviour and the changes in properties have been compared to predictions using various methods of prediction. Special investigations have been carried out concerning the increase in shear strength at consolidation and the durability of prefabricated drains in harsh environmental conditions.

The site investigations showed the necessity of careful documentation not only of the stratification of the soil and its mechanical properties, but also the ground water conditions and in this case also the environmental conditions. It was found that in peat special samplers have to be used, even if the peat is very amorphous. Field vane tests proved to be useful provided that the standard procedure for testing is followed and the shear strength values are corrected with respect to the liquid limit. However, the relevance of the field vane test in peat is questionable as the results are sensitive to the size of the vane.

In the laboratory tests was found that most of the testing methods and equipments used for soft mineral clays can be used also for organic and calcareous soils. The procedure for estimating undrained shear strength by normalization towards the preconsolidation pressure alone cannot be used in organic and calcareous soils with very low initial preconsolidation pressures. A new procedure for this estimation has been proposed. The shape of the yield surface was found to be highly anisotropic as it is for most natural soils.

All the monitoring equipments functioned very well. The very large deformations proved to be the limit for the equipments with vertical tubes as some of them were deformed in such a way that the measuring devices could not be inserted at the end of the observation period. The piezometers were of a type with rigid connections to the ground surface. In spite of precautions, there was considerable pushing of the piezometers because of deformations in the overlying soil layers.
The observations of the test embankments showed that large settlements as well as large horizontal displacements occurred. The behaviour of the two embankments was almost identical, except for the first stage, where the horizontal deformations were smaller and the vertical compressions somewhat larger and faster under the embankment with vertical drains. A special investigation showed that the paper filters deteriorated rather quickly in this type of environment and the function of the drains seems to have been limited to the first construction stage which lasted for half a year. The large horizontal deformations reflected the very soft soils and the low factor of safety against shear failure. They were not immediate, but continued for some time after full load application, whereupon they practically stopped. The vertical settlements were large and continued at the end of all three stages.

The observations clearly showed that in observation of embankments not only the behaviour of the embankments and the soil underneath should be observed, but also the variations in the natural ground outside the embankments.

Predictions of deformations have been carried out by a number of methods. It was found that the course of consolidation can be estimated only if the variability of the consolidation parameters, the load and the geometry during the consolidation process is accounted for. A conventional analysis does not give satisfactory predictions. The effect of creep cannot be ignored, especially not in the long-term perspective. Finite element analyses require very sophisticated models to give better results than the combination of initial shear deformations and one-dimensional consolidation.

Calculations of stability were also carried out by a number of methods. No failure occurred, but the initial deformations at loading in the second and third stage indicated that the factor of safety was low. The suggested methods for prediction of undrained shear strengths under embankments coupled with a calculation method with slices and using ADP-analyses then yielded safety factors of about 1.2. This order of the safety factor was further confirmed by effective stress analyses using the observed pore pressures after load application.
NOTATIONS AND SYMBOLS

c'  
\text{Effective cohesion intercept}

c_u  
\text{Undrained cohesion intercept}

CK_0 UTC  
\text{Undrained } K_o \text{-consolidated triaxial compression test}

DSS  
\text{Direct simple shear test}

\frac{Du}{Dt}  
\text{Material derivative}

e  
\text{Void ratio}

E  
\text{Modulus of elasticity (Young's modulus)}

ESL = \frac{\sigma_p'}{\sigma_v'}  
\text{Effective stress level}

f_1  
\text{Volume force}

F  
\text{Safety factor}

FVT  
\text{Field vane test}

k  
\text{Coefficient of permeability}

K_o = \frac{\sigma_h'}{\sigma_v'}  
\text{Coefficient of earth pressure}

m  
\text{Exponent, slope of the relation } \log \left( \frac{\tau_{fu}}{\sigma_v'} \right) 
\text{versus } \log \text{OCR}

m_{nc}  
\text{Exponent, slope of the relation between } \log \left( \frac{\tau_{fu}}{\sigma_v'} \right) 
\text{and } \log (\text{ESL}) \text{ in the normally consolidated state (ESL<1)}

m_{oc}  
\text{Exponent, slope of the relation between } \log \left( \frac{\tau_{fu}}{\sigma_v'} \right) 
\text{and } \log (\text{ESL}) \text{ in the overconsolidated state (ESL>1)}

n  
\text{Porosity}

OCR  
\text{Overconsolidation ratio}

p' = \frac{(\sigma_v' + 2 \sigma_h')}{3}  
\text{Isotropic effective stress in triaxial compression test}

q = \sigma_v' - \sigma_h'  
\text{Deviatoric stress in triaxial compression test}

q_j  
\text{Component of the specific discharge vector of pore water}

q_w  
\text{Discharge capacity}

S  
\text{Normalized undrained shear strength at ESL=1}

t  
\text{Time}
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$u_a$</td>
<td>Artesian pore pressure</td>
</tr>
<tr>
<td>$u_h$</td>
<td>Hydrostatic pore pressure</td>
</tr>
<tr>
<td>$u_{i,j}$</td>
<td>Gradient of pore water pressure</td>
</tr>
<tr>
<td>$w_{i,j}$</td>
<td>Component of the displacement gradient</td>
</tr>
<tr>
<td>$w_j$</td>
<td>Displacement vector</td>
</tr>
<tr>
<td>$\beta$</td>
<td>Compressibility of the pore water</td>
</tr>
<tr>
<td>$\delta_{i,j}$</td>
<td>Kronecker's delta</td>
</tr>
<tr>
<td>$\varepsilon_o$</td>
<td>Volume strain</td>
</tr>
<tr>
<td>$\eta=q/p'$</td>
<td>Relation between deviatoric stress and effective isotropic stress</td>
</tr>
<tr>
<td>$\nu$</td>
<td>Poisson's ratio</td>
</tr>
<tr>
<td>$\Xi$</td>
<td>Convective coordinate</td>
</tr>
<tr>
<td>$\sigma'$</td>
<td>Effective stress</td>
</tr>
<tr>
<td>$\sigma'_{h}$</td>
<td>Effective horizontal stress</td>
</tr>
<tr>
<td>$\sigma_{i,j}$</td>
<td>Total stress tensor</td>
</tr>
<tr>
<td>$\sigma'_{i,j}$</td>
<td>Effective stress tensor</td>
</tr>
<tr>
<td>$\sigma'_{p}$</td>
<td>Preconsolidation pressure</td>
</tr>
<tr>
<td>$(\sigma'_{p})_o$</td>
<td>Initial preconsolidation pressure</td>
</tr>
<tr>
<td>$\sigma'_{v}$</td>
<td>Effective vertical stress</td>
</tr>
<tr>
<td>$\tau_{fu}$</td>
<td>Undrained shear strength</td>
</tr>
<tr>
<td>$\tau_i$</td>
<td>Initial undrained shear strength</td>
</tr>
<tr>
<td>$\phi'$</td>
<td>Effective angle of friction</td>
</tr>
</tbody>
</table>
1. INTRODUCTION

The construction of roads and dykes on organic soils is associated with a number of problems as the organic soils are often highly compressible and have low shear strengths.

The methods of construction, as well as the methods and equipments used for sampling and testing that are normally used in other soft soils, may not be adequate in organic soils. Also the calculation methods normally used to predict stability and deformations may not be applicable to this type of soil.

However, in many countries there are large areas with organic soils where different kinds of embankments have to be constructed. In such cases, the prediction of soil behaviour and the selection of a proper design method becomes an important and complex engineering task.

In Poland, embankments and dykes are often located in swampy areas with very soft organic soils (e.g. peat, gyttja, gyttja-bearing calcareous soil).

In 1975, the Department of Geotechnics at Warsaw Agricultural University (DG) was commissioned by the Ministry of Agriculture to investigate the conditions for the construction of dykes in the Notec River Valley.

The initial investigations along the Notec River indicated very difficult geotechnical conditions. Extensive in situ and laboratory investigations have later been performed at two sites in the area. The first site was the Bialosliwie site where the organic soil layers are only about 4 m thick and the second site is the Antoniny site with about 8 metres of organic soils. Several test embankments were built at the Bialosliwie site in the period 1976-1982. The embankments were constructed on varying kinds of organic soils with the aim of studying the consolidation process, the stability and the possibility of utilizing vertical drains in this type of soil. The results of these investigations have been reported in internal reports, doctoral theses and conference papers.

In Sweden, roads are often constructed across peat bogs and other areas with organic soils. The Swedish Geotechnical Institute (SGI) has investigated design methods and developed field and laboratory tests and equipment for this purpose, mainly since 1977. The investigations have mostly been made on commission by the Swedish National Road Administration and the Swedish Council for Building Research has also given substantial grants. The investigations have included test embankments on peat, the development of a new peat sampler, development of laboratory methods and development of calculation methods. Some aspects of the use of vertical drains in organic soils have also been studied.
Discussions on a joint project between DG and SGI concerning construction of embankments on organic soils started in 1981. The aim of the collaboration was to combine the resources and experience at DG in the construction of embankments on organic soils and the experience and capabilities at SGI in investigation and instrumentation in very soft soils. Within this joint project two test embankments have been constructed in stages at the Antoniny site. Under one of these embankments vertical prefabricated drains were installed.

In this report, the results of the soil investigations and the observed behaviour of the embankments are presented. The observed behaviour is compared to the predicted behaviour using various methods for prediction.

The results elucidate the applicability of existing field and laboratory methods for investigation of properties of organic soils with a high degree of decomposition and of organic and calcareous mineral soils. They also illustrate the limitations and problems with monitoring equipments in very soft soils.

The applicability of existing methods for prediction of stability, deformations and increase in shear strength during consolidation is also examined.

2. DESCRIPTION OF THE SITE AND THE TEST EMBANKMENTS

2.1 Location and description of the test area

The test area is located in north-western Poland in the Notec river valley. The river Notec originates in central Poland and first flows north towards the city of Bydgoszcz where it turns westwards. It ends in western Poland connecting to the river Warta, which in turn connects to the river Odra. The river Odra constitutes the western border of Poland and has its outlet in the Baltic at Szczecin. The test site is located about 100 m south of the Notec River and about 3 km from the village Bialosliwie. The river valley here is about 10 km wide and the area is relatively flat, Fig. 1. The ground is covered by grass vegetation and has so far been used mostly as pasture land. Construction of fish ponds is planned in the area. The upper soft soils in the area consist of a layer of calciferous peat on top of a layer of fine-grained calcareous soil. Under the soft soils there is dense sand. The soft soils are quaternary deposits. The sediments originate from the limestone in the area and were deposited after the last glaciation. The fine-grained soft soil forms a tight lid on top of the sand and due to the topography of the valley there is an artesian water pressure in the sand. The area is seasonally flooded.
2.2 Method of construction

The method of construction was chosen with consideration to stability aspects. Due to low initial shear strength in the soil, the embankments were constructed in stages. It was then possible to utilize the increase in shear strength under the embankments due to consolidation.

The safe load for each stage was estimated from stability analyses based on the measured shear strengths of the soil prior to the various stages and the construction schedule was then decided.
Under one of the embankments vertical prefabricated drains were installed in a 1.2 m square grid. Drains with paper filters and a plastic core were used.

The construction of the embankments started in November 1983 and the loading schedules were as follows:

- In STAGE 1, the embankments were built up to a thickness of 1.2 m. Construction time was about a week and the duration for the stage was 4-5 months.

- STAGE 2 started in April 1984 and the thicknesses of the embankments were then increased to 2.5 m for the embankment without drains (embankment No. 1) and 2.7 m for the embankment with vertical drains (embankment No. 2). Construction time was 1-2 weeks and the duration for the stage was 13 months.

- The THIRD STAGE started at the end of May 1985. The thicknesses of the embankments were then increased to 3.9 m for embankment No. 1 and to 4.0 m for embankment No. 2. Construction time was 2-3 weeks and this stage was applied for 2 years.

Field vane tests were performed before construction started, at the end of stages 1 and 2 and during stages 2 and 3, Figs. 2 and 3.

The embankments were constructed of layers of sand, about 0.2 m thick. The sand had an average bulk density of 1.75 t/m$^3$ and a natural water content of about 10%, Table 1. The embankments were designed with base dimensions 35 x 40 m and slopes with inclinations of 1:3 on the sides and 1:2 at the ends, Figs. 4 and 5.

The soil below and outside the embankments was instrumented with monitoring equipment before construction started.
Fig. 2. Construction schedule of Embankment No. 1.

Fig. 3. Construction schedule of Embankment No. 2.
Table 1. Physical properties of sand in Embankments No. 1 and No. 2.

<table>
<thead>
<tr>
<th>Embankment</th>
<th>Stages</th>
<th>Water content $w_N$</th>
<th>Density $t/m^3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>I</td>
<td>10.6</td>
<td>1.68</td>
</tr>
<tr>
<td></td>
<td>II</td>
<td>8.0</td>
<td>1.76</td>
</tr>
<tr>
<td></td>
<td>III</td>
<td>8.5</td>
<td>1.85</td>
</tr>
<tr>
<td>2</td>
<td>I</td>
<td>13.3</td>
<td>1.64</td>
</tr>
<tr>
<td></td>
<td>II</td>
<td>5.9</td>
<td>1.74</td>
</tr>
<tr>
<td></td>
<td>III</td>
<td>10.6</td>
<td>1.82</td>
</tr>
</tbody>
</table>

Cross-section A-A

Cross-section B-B

Fig. 4. Dimensions of Test Embankment No. 1.
2.3 Monitoring equipment

The monitoring equipments were selected from the equipments available at DG and SGI. Expected deformations and earlier experiences from test embankments were taken into account when the equipments and their locations were selected.

A description of the various equipments used at the Antoniny site and their locations is given below.

Readings of all the monitoring equipments were taken with short time intervals just after the start of a new load stage and at longer time intervals thereafter. The readings have mainly been taken by personnel from DG. Once a year, field engineers from SGI have visited the test site for additional investigations and measurements.
2.3.1 Equipment for measurement of settlements

Settlement plates and screw plates were installed at the ground surface and at the interface between peat-gyttja and the calcareous soil to determine the settlement distribution in the main layers. These plates and screw plates were extended to the surface of the embankment by rods inside protecting pipes, Fig. 6.

In order to obtain continuous settlement distributions across the embankments, flexible tubes were placed in shallow ditches at the ground surface before the construction started. Also, the heave outside the toes of the embankments was measured in these tubes. The level of the tubes was measured by a hose settlement gauge type SGI II, Fig. 7.

The measuring unit consists of two plastic tubes with different diameters. The tube with the smaller diameter contains air and an electric cable and is inserted into the larger tube. The annular space between the two tubes is filled with a liquid (generally water). The lower ends of the tubes are connected to the measuring head, which contains a pressure transducer. This transducer measures the liquid pressure in relation to the atmospheric pressure. The upper ends of the plastic tubes are connected to an open standpipe. Thus the difference in level between the measuring head, which is inserted in the flexible tube under the embank-
ment, and the liquid level in the standpipe can be measured. The liquid level in the standpipe is in turn levelled in relation to a fixed reference point located outside the test area. The equipment has been found to have an accuracy of ±3 mm.

Fig. 7. Hose settlement gauge type SGI II.
In order to obtain more detailed information on the distribution of settlements with depth, a number of magnetic screw settlement gauges were installed. First, plastic tubes with screw tips were installed into the soil. Around these guiding tubes a number of screw plates were installed at desired depths. These screw plates contain small magnetic rings and are designed to slide freely along the plastic tube, Fig. 8.

The distance between the fixed reference plate at the bottom and the sliding plates is measured by a settlement indicator. This indicator consists of a measuring tape with a magnetic switch at the bottom. When the measuring tape is lowered into the plastic tube, an electric circuit is closed each time the switch passes the magnetic rings in the screw plates. The accuracy of the indicator is about ±1 mm.

Fig. 8. Magnetic screw settlement gauge.
2.3.2 Equipment for measurement of horizontal movements

The horizontal movements of the soil have been measured with an SGI type inclinometer inside flexible plastic tubes driven to the dense bottom layer of sand. The equipment is shown in Fig. 9.

The measuring unit has a cylindrical shape. Two pairs of guiding bosses are pressed against the inner wall of the plastic tube by a spring. A pendulum inside the cylinder is suspended by a leaf spring. The bending moment in the spring is measured electrically by strain gauges. The measuring system is compensated for bending moments in directions other than the measured direction.

The measuring unit must be oriented in the measuring direction with great precision if small horizontal movements are to be measured. This is achieved by attaching the measuring unit to a string of rods with torsionally rigid connections. The measuring direction is found by taking the bearing to a horizontal scale fitted on the tube. The horizontal scale is oriented towards a distant reference point with the aid of a telescope fitted to the scale.
The plastic tubes have an inner diameter of 42 mm. They are installed as vertically as possible into the ground. The stiffness of the plastic tubes is so low that the tubes will follow the horizontal movements of the soil. The inclination of the tubes is normally measured at every metre of depth. The measurements are usually taken in pairs, one reading in the measuring direction and one in the direction perpendicular to it.

A change in inclination in a certain direction is a measure of a corresponding angular strain in the soil. The position of the tube in relation to its tip can be calculated by integrating the measured inclinations from the tip and upwards. The higher the point of consideration is located, the greater the number of terms in the calculations becomes and thus also the error. The change in position of the tube is a measure of the horizontal movement of the soil.

Problems may occur when the tubes tend to buckle due to large settlements in the soil. A special telescopic tip on the tubes can be used to overcome this problem. This type of tip was used at the Antoniny site.

From experience, it has been found that inclinations can generally be determined within an error of less than 0.25% and the direction of measurement can be fixed within half a degree. An estimated reading and contact error of 0.2 mm/m should be expected for the individual measurements of the inclination. The greater the inclination of the tube is, the greater the error band for the readings becomes.

In calculation of the horizontal movements, it is usually assumed that no movement takes place below the level of the deepest reading. If possible, the tubes should be installed in such a way that this assumption is fulfilled. At the Antoniny site, this was not possible due to the dense sand. The change in inclination of the bottom part has therefore been taken into account in the integration.

2.3.3 Equipment for pore pressure measurements

The pore water pressures in the bottom sand layer outside the test embankments and the free water levels inside the embankments were measured in open standpipes. The standpipes inside the fill material were provided with square plates 0.3 x 0.3 m to ensure that the tips followed the settling base of the embankment.

The pore pressures in the compressible layers were measured at different levels and locations using the Swedish BAT system.
In the BAT system, the filter tips and the extension pipes are separated from the measuring sensor. The BAT piezometer Mk II consists of a plastic tip with a ceramic filter. The filter tip is saturated with de-aerated water by boiling the whole tip in water. The upper part of the tip is shaped as a nozzle and is sealed with a special rubber disc. The tips are threaded onto one-inch galvanized steel pipes and are then driven to the desired location in the ground. The tips and pipes are permanently installed. At the Antoniny site, protecting pipes were installed outside the piezometer pipes to about 1 metre from the tip to prevent them being pushed further into the soil due to the settlements of the overlying soil and embankment.

Readings of the pore pressures are taken by lowering a sensor containing a pressure transducer inside the pipes until it comes into contact with the filter tip. At the lower end of the sensor, there is a hypodermic needle which penetrates the rubber disc and provides contact between the transducer and the water in the filter tip. A stabilized reading of the pore pressure is usually obtained within 3 - 20 minutes, Fig. 10.

---

**Fig. 10. BAT piezometer system.**
After the reading is taken, the sensor is withdrawn and the rubber disc automatically seals off the filter tip. This procedure can be repeated hundreds of times without damaging the rubber disc.

The function of the system can be checked by taking a new reading after the sensor has been lifted 20 - 30 mm and the calibration can be checked if the water level inside the pipe is known. The sensor is then moved to the next filter tip and the procedure is repeated.

2.3.4 Location of the monitoring equipment

The locations of the monitoring equipments for the two embankments are shown in Figs. 11 and 12.

- Settlement plates and screw plates were placed just below the original ground surface and at the interface between peat-gyttja and the calcareous soil. They were installed at the centre of the embankments, at the middle of the slopes, at the toes of the slopes and outside the embankments.

- Magnetic screw settlement gauges were placed under the centres of the embankments and under the middle of the slopes.

- Plastic tubes for measurements with the hose settlement gauge were installed in shallow ditches across the centre parts of the embankments.

- Inclinometer tubes were placed on one side of each embankment. They were placed at the middle of the slopes, at the toes of the slopes and outside the slopes. At embankment No. 1 two inclinometer tubes were installed 3 and 7 metres outside the toes of the slopes and at embankment No. 2 only one inclinometer tube was installed 5 metres outside the toe of the slope.

- Piezometers were installed at three levels at the centre of the embankments and at two levels under the slopes of the embankments. At embankment No. 1 piezometers were also installed at three levels 7 metres outside the toe of the slope at one side of the embankment.
LEGEND:

- S - settlement gauge
- M - magnetic settlement gauge
- H - hose settlement gauge
- L - inclinometer
- P - BAT piezometer

Fig. 11. Location of monitoring equipment at Test Embankment No. 1, without vertical drains.
Fig. 12. Location of monitoring equipment at Test Embankment No. 2, with vertical drains.
3. SITE INVESTIGATIONS

A large number of weight soundings, samplings, vane shear tests and pore pressure observations and also some cone penetration tests and pore pressure soundings were made in the test area in 1983 prior to the construction of the embankments, Fig. 13. A number of special testing programmes were also carried out to investigate the influence of different equipments and testing procedures on the results of vane shear tests. The spread in test results was also investigated.

Further investigations aimed at finding out changes in soil properties under the embankments have been made at the end of the various loading stages.

3.1 Soundings and samplings

Soundings to investigate the thicknesses of the soft soil layers were made with the Borro equipment for weight sounding. A large number of samples in the soft soils were taken with a Borro ø 60 mm piston sampler. Sampling in a number of adjacent holes was arranged in such a way that the samples from the two holes overlapped and "continuous cores" were obtained.

Sampling in organic soils often involves problems with sample disturbance. In fibrous and highly permeable peaty soils, the cutting resistance is high and the risk for compression of the soil during sampling is also high. In highly "elastic" organic soils such as gyttja there is a risk both of compression during sampling and elongation during the removal of the sampler. Additional samples were therefore taken with the ø 50 mm Swedish standard piston sampler (SGI 1961) and a newly constructed peat sampler which has a sample diameter of 100 mm. The standard piston sampler was used for taking samples from the entire soil profile, while the peat sampler was only used in the upper 3 metres with peaty soil.

The new peat sampler consists of a razor-sharp wave-toothed cutting edge mounted on a plastic tube, Fig. 14. The plastic tube has an inner diameter of 100 mm and a length of 1 m. On top of the tubes there is a driving head. Depending on the type of soil, the sampler can be driven by either a light pressure combined with an oscillating twisting movement or by light, rapid blows. The sampler has open ends and the samples are taken at the bottom of predrilled holes.
Fig. 13. Location of samplings and field tests.
The results of the soundings and samplings in 1983 indicated that at the location for embankment No. 1 there was 3.3 - 3.5 m of peaty soil and below that calcareous soil down to a depth of 7.7 - 8.1 m. At the location for embankment No. 2 there was 3.3 m of peaty soil and calcareous soil down to 8.0 - 8.1 m below the ground surface. The ground surface and thus also the soil layers were fairly horizontal.

New samples were taken with piston samplers under the embankments at the end of the various loading stages.

In the initial testing programme, DG also performed a number of investigations with the Polish SLVT penetrometer (Borowczyk 1982). The SLVT is a Polish standard dynamic penetrometer provided with a vane at the tip, Fig. 15. The penetrometer is driven into the soil by blows from a 10 kg hammer with a 0.5 m free fall. The number of blows for each 0.1 m of penetration is recorded. A vane test is performed at every 0.5 m of penetration, whereby the maximum torque required to turn the penetrometer is recorded.

Results from two such tests are shown in Fig. 15. The test results indicate a somewhat stiffer surface layer, followed by very soft soil down to about 3 m. A somewhat stiffer layer is detected about 4 m below the ground surface, whereupon the soil again becomes soft down to a depth of 8 m. The resistance in the sand layer below 8 m depth indicates that the sand is rather dense.

The measured torques in the SLVT-tests were somewhat lower than the corresponding values obtained in the subsequent field vane tests. This result is in accordance with previous experience regarding the effect of the design of vane testing equipments.
Fig. 15. Design of the Polish SLVT penetrometer with results from penetration tests.
3.2 Field vane tests

A large programme of field vane testing was carried out at the Antoniny site. Both DG and SGI were involved in different programmes before and during the period of construction and observation. Both Polish and Swedish equipments were used. They were mostly used for separate programmes, but in some cases they were also compared.

Previous investigations have shown that the design of the vane testing equipment as well as the testing procedure have important effects on the test results (e.g. Torstensson 1973). The most important parameter in this aspect in the testing procedure, which is also most likely to vary between different tests, is the speed of rotation which can also be expressed as time to failure. In fibrous peat, it has furthermore been found that the size of the vane has a pronounced effect on the test results, (Golegbiewska 1976, Landva 1980). There is normally a relatively large scatter in the test results in peat.

A vane testing programme with different rates of rotation was therefore carried out by SGI in order to find out if the rate effects in this type of soil are similar to those normally encountered in Scandinavian clays and gyttjas. A large number of tests according to the standard procedure were carried out in order to determine the magnitude of the scatter in the results and comparative tests were also performed with a large vane in order to investigate the influence of the size of the vane.

These field vane tests were performed with the SGI type of field vane equipment where the rods are protected by a casing and also the vane is protected during most of the penetration (Cadling and Odenstad 1950). The equipment was fitted with an electric motor and a gearbox to obtain the desired rates of rotation. The torque was recorded on a field vane instrument of the Geotech type, Fig. 16.

At the same time, DG performed a large number of field vane tests in connection with the initial field investigation using the Polish field vane equipment PS0-1 (Fig. 17). These investigations were so numerous that a corresponding evaluation of average values and scatter as in the SGI tests could be made and the results could be compared. There is no recording instrument on the PS0-1 equipment, but the stress-strain curves were obtained by manual recording of torque and rotation.

A large programme of field vane testing during the different phases of the observation period was later carried out by DG to investigate the successive increase in undrained shear strength during consolidation. (See Chapter 6.3). In this programme the PS0-1 equipment was used.
Fig. 16. Recording field vane instrument of the Geotech type fitted with an electric motor and a gearbox for rotation of the vane (upper photo).
Pushing the vane instrument into the soil with the Geotech type boring rig (photo below).
Fig. 17. Field vane equipment type PSO-1.
At the end of each stage, the shear strength under embankment No. 1 was measured using both SGI and PSO-1 equipments.

In the initial investigations carried out by SGI, seven profiles were tested with the standard vane (65 x 130 mm) and the standard rate of rotation, which gives failure in approximately 3 minutes. In addition, two profiles were tested with the standard vane, but with rates of rotation 10 times faster and 10 times slower than the standard rate respectively. Finally, one profile was tested with standard rate of rotation but with a larger vane with the dimensions 80 x 160 mm.

The results of these tests are shown in Figs. 18 - 21. The results are given as shear strength values. These values have to be corrected with respect to the plasticity of the soil to obtain a useful undrained shear strength.

The real time to failure was measured in all tests and the results from the tests with standard rate of rotation have been corrected for the measured differences in time to failure according to Torstensson (1973). A constant rate of rotation at the instrument does not entail a completely constant rate of rotation of the vane, as the deflection of the recording spring and the torque in the rods affect the resulting rotation of the vane. The results have been corrected to correspond to a time to failure of 3 minutes. For tests with standard rates of rotation the corrections are small, however.

The correction factors for time to failure are shown in Table 2. The measured values should be divided by these factors.

In the standard field vane tests, strength values between 9.5 kPa and 18 kPa were measured, Fig. 18. The lowest values were measured in the peat at 2 m depth and the highest values in the stiffer layer at 4 m depth. The average values varied between 10.5 and 16.8 kPa. The maximum deviation of a single value from the average was about 30%. If all values are considered, the standard deviation from the average is between 6 and 13% for the different levels. When a few obviously odd values are excluded, the standard deviations are reduced to about half of these values.

The results and the scatter obtained in the standard tests using SGI equipment or PSO-1 equipment were almost identical.
Fig. 18. Results of field vane tests according to Swedish standard: 
Vane size 65 x 130 mm and time to failure about 3 min.

Fig. 19. Results of vane tests with different rates of strain.
Shear strength values, kPa

Fig. 20. Results of field vane tests with different rates of strain. Values corrected to standard rate according to Torstensson (1973).

Fig. 21. Results of field vane tests with different sizes of vane.
Table 2. Correction factor for the different times to failure.

<table>
<thead>
<tr>
<th>Time to failure, sec</th>
<th>Correction factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>1.141</td>
</tr>
<tr>
<td>30</td>
<td>1.100</td>
</tr>
<tr>
<td>45</td>
<td>1.076</td>
</tr>
<tr>
<td>60</td>
<td>1.060</td>
</tr>
<tr>
<td>120</td>
<td>1.022</td>
</tr>
<tr>
<td>180</td>
<td>1.000</td>
</tr>
<tr>
<td>240</td>
<td>0.985</td>
</tr>
<tr>
<td>300</td>
<td>0.973</td>
</tr>
<tr>
<td>360</td>
<td>0.964</td>
</tr>
<tr>
<td>420</td>
<td>0.956</td>
</tr>
<tr>
<td>480</td>
<td>0.949</td>
</tr>
<tr>
<td>540</td>
<td>0.943</td>
</tr>
<tr>
<td>600</td>
<td>0.930</td>
</tr>
</tbody>
</table>

The results with different rates of strain (Fig. 19) show that a large decrease in rate of strain generally resulted in lower measured strength values. An increase in rate of strain gave higher strength values in the calcareous soil, but not in the peaty soil. Strain rate effects in peat are complex, however, as drainage plays a large role also in these relatively quick tests, due to the high permeability of the soil. In Fig. 20 the tests have been corrected for strain rate according to Torstensson (1973). It is found that this correction seems to provide almost identical results from tests with different rates of strain in the calcareous soil. In peat, however, this kind of correction is not useful. The sensitivity of the test results to changes in strain rate in the peat is small, though. Within the tested variation of strain rates, the results are almost always within the normal scatter at the standard rate of strain.

The results of the vane tests with a larger size of vane showed no influence of vane size on the results in the calcareous soil. In the peaty soil, the measured strength values generally became somewhat lower in the tests with the larger vane. This is in accordance with the experience previously mentioned. The results from the field vane tests in the peaty soil thus have to be treated cautiously.

The results from the field vane tests in natural ground in 1983 were almost identical, regardless of whether SGI-equipment or PSO-1 equipment was used. In the field vane tests under embankment No. 1 at the end of stage 1 and stage 2 the strength values from the PSO-1 equipment generally became somewhat higher, Fig. 22. This may be a coincidence. When due consideration to the sensitivity to exact depth is taken, the results are within the limits for the natural variation.
3.3 Cone penetration tests and pore pressure soundings

Previous laboratory tests at DG had indicated a good correlation between cone resistance and shear strength values in peat (Mirecki 1983). The cone penetration test in combination with pore pressure measurements during penetration is also an excellent tool for soil profiling. Furthermore, the pore pressure dissipation when the penetration is stopped is sometimes used to estimate the coefficient of consolidation at horizontal water flow.

The very soft soils would have required very sensitive equipment to estimate the soil properties. No such equipment was available at the time for the initial soil investigations, but it was still considered interesting to use the existing equipments to obtain a general picture of the soil profile.

The equipments used were a standard Borro type of probe for measurement of a maximum point resistance of 10 MPa and a pore pressure probe similar to one of the BAT designs, Fig. 23. The point resistance and the generated pore pressures were thus measured in separate penetration tests. In both cases the probes were driven into the soil by a Geotech rig at the standard rate of 0.02 m/sec.
Fig. 23. Design of Swedish probe for pore pressure soundings in cohesive soils (figure).

Instrumentation of the pore pressure soundings (photo).
The electrical signals from the probes were recorded on a TOA Electronics Ltd recorder model EPR-IFA.

A typical result of a cone penetration test is shown in Fig. 24. The curve shows that there is a type of crust down to about 0.8 m depth and that the tip of the probe reaches the sand layer at 7.8 m depth. A slightly stiffer layer is indicated between 3 and 4 m depth, where also the field vane tests gave higher strength values.

![Figure 24. Result of a cone penetration test.](image)

The point resistances in the soft soil were only of the order of 1 per cent of the capacity of the probe and the zero-drift as measured in zero off set from before to after the tests was of the order of 30 per cent of the measured point resistances. Therefore, no estimates of shear strengths can be made from the measured point resistances.

In Fig. 25 results from a pore pressure sounding are shown. The pore pressures generated during penetration are shown together with the equilibrium pore pressure, $u_0$, measured in the dissipation tests.
The excess pore pressures during penetration became negative in the peat and positive but low in the calcareous soil. When a pore pressure probe is pushed into normally consolidated clay an excess pore pressure is generated. At the actual location of the filter, the excess pore pressure is usually of the order of about 5 times the undrained shear strength (Torstensson 1975). The more overconsolidated the clay is, the lower the generated excess pore pressures become (e.g. Jamiolkowski et al 1985). The results from the Antoniny site thus indicate that the overconsolidation ratio is rather high.

The rate of dissipation of the excess pore pressure in the soil was used to estimate the coefficient of horizontal consolidation, $c_h'$, in the calcareous soil. In the peat where the excess pore pressures were negative no such estimation was possible.

The average of the values of $c_h'$ in the calcareous soil estimated in this way was about an average $4 \cdot 10^{-5} \text{ m}^2/\text{s}$.

According to Torstensson (1977) the $c_h'$ values evaluated from dissipation tests should be divided by about 2 in normally consolidated soils. Campanella et al (1982) have found that the evaluated $c_h'$ values should be divided by at least 3 and assume that it is rather the coefficient of consolidation in the overconsolidated range that is measured in the dissipation tests.
A comparison with the laboratory tests shows that the $c_h$ values from the dissipation tests are about two times the $c_v$ values from oedometer tests in the overconsolidated range. The $c_v$ values, however, drop by about 10 times when the preconsolidation pressure is exceeded and the $c_h$ values from the dissipation tests can be considered as totally irrelevant for prediction of settlement rates in the actual case.

### 3.4 Pore pressure observations

The pore pressures in the soil were measured for a short period before the construction of the embankments. The water pressure in the sand layer and the free ground water level in the peat have been observed during the whole construction period in open standpipes and holes respectively.

The average free ground water level in the peat has been 0.2 m below the ground surface. The fluctuation during the construction period has been from 0 to 0.5 m below the ground surface. Seasonally the area has been flooded.

At one time before construction started, the free ground water level sank to more than 1 m below the ground level, but that has not been repeated. The water pressures in the sand layer below the soft soils have been found to be artesian and 12 - 15 kPa higher than the hydrostatic pressure from the free ground water level in the peat. The water head in the sand layer has thus been 1 to 1.5 m above the ground surface. Fig. 26.

The pore pressure measurements in the compressible layers before construction started showed that the artesian excess pore pressure gradually evened out and that the pore pressures from 2.0 m below the ground surface and upwards were hydrostatic from the free ground water level.

The high ground water level, combined with the artesian excess pore pressures and soils with relatively low bulk densities, entailed that the initial effective stresses in the profile were very low and only amounted to a few kPa throughout the compressible profile, Fig. 27.
a) Location of open piezometers

b) Open piezometers readings.

Fig. 26. Ground water conditions during the observation period.
Fig. 27. Initial pore pressure and vertical stress conditions.

\[ u_0 = u_h + u_a \]
4. LABORATORY TESTS

Samples from the Bialosliwie site have been tested at the laboratories at SGI and DG.

A first series of tests was performed at the SGI laboratory in January 1983. These tests were performed jointly by staff from DG and SGI. The samples were taken with the Borros ø60 mm sampler at two levels, 1.47 – 1.81 m and 5.67 – 6.01 m.

The samples were examined concerning density, water content, Atterberg limits, organic content, carbonate content, undrained shear strength and sensitivity by Swedish standard methods. The structure of the soil was studied in a scanning electron microscope and the mineral contents were examined by X-ray diffraction.

Oedometer tests were performed in the SGI constant rate of strain oedometers as well as in incrementally loaded oedometers. Constant rate of strain tests were performed with different rates to study rate effects on the results. Incremental tests were performed with standard loading procedure as well as larger load steps to simulate the field loads. Creep effects were studied in the incremental tests.

The undrained shear strength was determined in the standard investigations with a large number of Swedish fall cone tests. Series of active and passive triaxial tests and direct simple shear tests were also run in the SGI equipment.

Further laboratory tests comprising routine tests and oedometer tests were performed at SGI in August 1983. Prior to this, new samples had been taken at every metre of depth with the Swedish standard piston sampler and samples from the upper peat layer had been taken with the new Swedish peat sampler.

Complementary tests concerning the initial soil properties were performed at DG using Polish equipment.

The results from all these investigations have been reported in detail in annual reports from DG (1983, 1984 and 1985) and a preliminary report from SGI (1984).

The determinations of organic content and carbonate content have later been repeated as new and better procedures for these determinations have been established (Larsson et al. 1985).

The routine tests showed a soil profile according to Table 3.
Table 3. Initial soil properties at the Antoniny site.

<table>
<thead>
<tr>
<th>Depth m</th>
<th>Soil Description</th>
<th>Density t/m³</th>
<th>Water content %</th>
<th>Liquid limit %</th>
<th>Plastic limit %</th>
<th>Plasticity index %</th>
<th>$\tau_{fu}$(^1) cone kPa</th>
<th>Sensitivity</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.47-1.64</td>
<td>Black very calciferous amorphous peat</td>
<td>1.11</td>
<td>341</td>
<td>313</td>
<td>189</td>
<td>124</td>
<td>5.8</td>
<td>7</td>
</tr>
<tr>
<td>3.0</td>
<td>Dark brown very calciferous dy-bearing gytta</td>
<td>1.19</td>
<td>227</td>
<td>221</td>
<td></td>
<td></td>
<td>8.5</td>
<td>12</td>
</tr>
<tr>
<td>4.0</td>
<td>Yellow white calcareous soil (marl)</td>
<td>1.31</td>
<td>134</td>
<td>129</td>
<td></td>
<td></td>
<td>9.8</td>
<td>12</td>
</tr>
<tr>
<td>5.0</td>
<td>Grey calcareous soil</td>
<td>1.38</td>
<td>109</td>
<td>96</td>
<td></td>
<td></td>
<td>7.0</td>
<td>13</td>
</tr>
<tr>
<td>5.67-5.84</td>
<td>Grey calcareous soil</td>
<td>1.42</td>
<td>106</td>
<td>103</td>
<td>54</td>
<td>49</td>
<td>8.7</td>
<td>9</td>
</tr>
<tr>
<td>6.0</td>
<td>Grey calcareous soil</td>
<td>1.41</td>
<td>108</td>
<td>97</td>
<td></td>
<td></td>
<td>8.3</td>
<td>13</td>
</tr>
<tr>
<td>7.0</td>
<td>Green-grey calcareous gytta</td>
<td>1.33</td>
<td>148</td>
<td>138</td>
<td></td>
<td></td>
<td>7.1</td>
<td>11</td>
</tr>
</tbody>
</table>

Samples from 1.47-1.64 m and 5.67-5.84 m are taken with Borro ø60 mm sampler.

Samples from 3, 4, 5, 6 and 7 m are taken with Swedish standard piston sampler.

1) Corrected according to SGI recommendations of 1984.

2) Samples taken with Swedish peat sampler.
The upper two metres consist of peat which is very calciferous. The organic content in the peat is highly variable but of the order of 50%. Further down, the carbonate content increases and the organic material decreases and occurs as gyttja.

At 4 m depth, there is a yellow-white layer of almost pure carbonate soil (marl). Further down, the soil is calcareous with a content of calcium carbonates of 80 to 90 per cent. The organic content here is about 5%. At about 7 m depth, the organic content increases somewhat and the soil is classified as a calcareous gyttja. The organic contents and contents of calcium carbonates are shown in Fig. 28.

![Fig. 28. Organic contents and contents of calcium carbonates. Antoniny, Bialoswie.](image)

The samples, which were freeze-dried and later examined in the scanning electron microscope, revealed that the peat consists of a mixture of decomposed plant remains and mineral particles in silt to clay sizes. The mixture is rich in various kinds of diatoms. The X-ray diffraction showed a large amount of calcium, considerable amounts of silicon and iron and some sulphur, Fig. 29.

The corresponding examination of the calcareous soil from about 6 metres depth showed a homogeneous material consisting of particles or aggregates of particles in silt to clay sizes. The X-ray diffraction showed a very large amount of calcium and this peak is so dominant that nothing else can be observed in the diagram, Fig. 30. The investigations at DG, however, have shown that there is a significant amount of sulphur also in this layer.
Fig. 29. Micrograph and X-ray diffraction diagram for peat from Antoniny, Bialosliwie.
Fig. 30. Micrograph and X-ray diffraction diagram for calcareous soil from Antoniny, Bialosliwie.
4.1 Deformation and consolidation characteristics

Oedometer tests were performed as CRS tests and incrementally loaded tests on the samples taken with a Borro Ø 60 mm sampler, as CRS tests on the samples taken with the standard piston sampler and as step loaded compressiometer tests on samples taken with the Swedish peat sampler. The oedometer tests were performed on samples with Ø 50 mm diameter and a sample height of 20 mm, while the compressiometer tests were performed on samples with a diameter of 100 mm and a height of 45 mm.

The preconsolidation pressure $\sigma'$, the coefficient of consolidation $c_v$ and the coefficient of secondary consolidation $\alpha_S$ have been evaluated from the steploaded oedometer and compressiometer tests. The preconsolidation pressure has been evaluated according to Casagrande (1936), Fig. 31.

In this method a horizontal line and a tangent to the oedometer curve at the point with the smallest radius of curvature are drawn. The angle between the horizontal line and the tangent is bisected. The straight portion of the oedometer curve is extended and the preconsolidation pressure is evaluated as the pressure at the intersection of this line and the bisectrix.

The coefficient of consolidation has also been evaluated according to Casagrande. This method involves plotting the deformation versus the logarithm of time, Fig. 32.
Fig. 32. Casagrande construction of $c_V$.

$U=0$ is constructed by assuming a parabolic shape of the first part of the curve. $U=100\%$ is constructed as the intersection between the tangent to the curve at its point of inflexion and the extension of the straight end part of the curve. $\varepsilon_{50}$ at $U=50\%$ is then calculated, $t_{50}$ is constructed and $c_V$ is calculated from

$$\varepsilon_{50} = T_{50} H^2 \frac{H^2}{t_{50}}$$

For oedometers with drainage from both ends $H_{50} = H_0 (1-\varepsilon_{50})/2$

where $H_0$ is initial sample height and the time factor $T_{50}=0.197$.

The coefficient of secondary consolidation can then be evaluated from the slope of the curve after the excess pore pressure has disappeared and thus the hydrodynamic delay of the deformations has ceased. The coefficient of secondary consolidation can be expressed as

$$\alpha_s = \frac{d\varepsilon}{d\log t}$$
The oedometer tests with constant rate of strain are evaluated according to Swedish practice as follows (Larsson and Sällfors 1985):

- From the tests, continuous curves are obtained for the relations effective vertical stress versus deformation and permeability versus deformation. From the first relation, a continuous curve for variation of the compression modulus with effective stress can be evaluated, Fig. 33.

- The preconsolidation pressure is evaluated according to Sällfors, (1975). The two straight parts of the stress - strain curve are extended and intersected. An isosceles triangle is inscribed between the lines and the stress - strain curve. The intersection point between the base of the triangle and the upper line represents the preconsolidation pressure \( \sigma'_p \). This construction is sensitive to scales and is therefore always made in a plot where the scales are such that the length representing 10 kPa on the stress axis corresponds to the length representing 1% on the strain axis.

- After determination of the preconsolidation pressure, the stress strain curve for higher stresses is moved horizontally a distance \( c \) to pass through the point where \( \sigma'_p \) was evaluated (Larsson 1981). With the low testing rates used according to Swedish practice, the value of \( c \) is usually small. As shown by Larsson and Sällfors (1985) the adjusted stress strain curve so obtained corresponds very well to the curve obtained from standard incremental tests.

- The modulus-stress plot is now modified. The initial constant modulus \( M_0 \) is extended to \( \sigma' \). At \( \sigma' \), the modulus is assumed to drop instantaneously to the second constant modulus \( M_1 \). The part of the curve where the modulus increases linearly with effective stress is moved \( c \) kPa to the left. The stress at the intersection with the constant modulus \( \sigma'_L \) is evaluated and the modulus number \( M' \) is evaluated as \( \Delta M/\Delta \sigma' \) for the part of the curve where the compression modulus increases linearly with effective stress.

Thus the curve is divided into three parts:

1. The part in the stress interval \( \sigma' - \sigma'_p \) where \( M=M_0 \)
2. The part in the stress interval \( \sigma'_p - \sigma'_L \) where \( M=M_L \)
3. The part in the stress region where \( \sigma'>\sigma'_L \) and where \( M=M_L +M'(\sigma' - \sigma'_L) \).
Fig. 33. Results from CSR-test and evaluation of compression and permeability properties.
• The initial modulus from the first loading of a natural "undisturbed" sample in the oedometer is never used. It is always too low compared to in situ initial modulus due to sample disturbance, swelling, and imperfect fit in the oedometer. In most cases $M_0$ has been estimated from empirical relations such as $M_0 = 250 \sigma_u$ or $M_0 = 50 \sigma_0$. To obtain a useful value of $M_0$ in the laboratory, the sample has to be unloaded when $\sigma_0$ is just exceeded to the "in situ" effective vertical stress $\sigma_0'$. It should then be allowed to swell before it is reloaded. $M_0$ is then evaluated from the reloading curve.

• The permeability is evaluated by simplifying the log permeability-strain curve to a straight line. The initial permeability $k_i$ is evaluated at the intersection of the straight line and the horizontal line $\varepsilon = 0$ and the decrease in permeability with compression is expressed by the parameter $\beta_k = -\Delta \log k/\Delta \varepsilon$.

Test on dry crusts, silts and remoulded clays can be evaluated using the same parameters, although the patterns often differ (Larsson 1981).

Recent investigations at SGI have shown that the compression parameters used for clay are useful also for peats. Lefebvre et al 1984 have suggested that natural strains should be used for peats and this may be a more accurate description of the compression characteristics. However, for the limited range of stresses that have been of interest in Swedish projects the difference is small.

No information on the rate of secondary consolidation is obtained from a CRS-test. To obtain this soil property, either empirical relations have to suffice or supplementary incremental tests have to be performed.

There was no obvious difference in evaluated preconsolidation pressure in the different types of test or the different samples. The evaluated stress-strain curves from incremental tests and tests with constant rate of strain were also compatible. The stress-strain curves from different samples of the peat indicated, however, that the samples taken with the standard piston sampler in this material were slightly more disturbed (compressed) than the other samples.

The results from the oedometer tests are listed in Table 4.
Table 4. Results from oedometer tests (average values).

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Test</th>
<th>Sampler</th>
<th>Number of tests</th>
<th>$\sigma_p$ (kPa)</th>
<th>$M_0$ (kPa)</th>
<th>$\sigma'_L$ (kPa)</th>
<th>$M'$ (kPa)</th>
<th>$a$ (kPa)</th>
<th>$k$ (m/s)</th>
<th>$\beta_k$</th>
<th>$c_v$ (m$^2$/s)</th>
<th>$\alpha_s$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.47-1.64</td>
<td>CRS</td>
<td>$\phi 60$</td>
<td>5</td>
<td>14.8</td>
<td>122</td>
<td>31.8</td>
<td>6.8</td>
<td>13.9</td>
<td>$1.7 \times 10^{-8}$</td>
<td>4.2</td>
<td>$10^{-6}-10^{-8}$</td>
<td>2.6-2.0</td>
</tr>
<tr>
<td>1.47-1.64</td>
<td>Step</td>
<td>$\phi 60$</td>
<td>3</td>
<td>$\approx 15$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.47-1.64</td>
<td>Step</td>
<td>$\phi 100$</td>
<td>3</td>
<td>$\approx 14$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td>CRS</td>
<td>$\phi 50$</td>
<td>5</td>
<td>15</td>
<td>120</td>
<td>28</td>
<td>8.1</td>
<td>13.0</td>
<td>$1.7 \times 10^{-8}$</td>
<td>4.2</td>
<td>$10^{-5}-10^{-7}$</td>
<td>2.9-2.3</td>
</tr>
<tr>
<td>3.0</td>
<td>CRS</td>
<td>$\phi 50$</td>
<td>3</td>
<td>17</td>
<td>240</td>
<td>65</td>
<td>7.3</td>
<td>32</td>
<td>$4 \times 10^{-9}$</td>
<td>3.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.0</td>
<td>CRS</td>
<td>$\phi 50$</td>
<td>2</td>
<td>23</td>
<td>382</td>
<td>50</td>
<td>8.9</td>
<td>7</td>
<td>$4 \times 10^{-10}$</td>
<td>2.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.0</td>
<td>CRS</td>
<td>$\phi 50$</td>
<td>3</td>
<td>21</td>
<td>260</td>
<td>39</td>
<td>10.5</td>
<td>14</td>
<td>$5 \times 10^{-10}$</td>
<td>2.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.67-5.84</td>
<td>CRS</td>
<td>$\phi 60$</td>
<td>5</td>
<td>19.4</td>
<td>288</td>
<td>41.2</td>
<td>11.7</td>
<td>16.6</td>
<td>$8.3 \times 10^{-10}$</td>
<td>2.2</td>
<td>$10^{-7}-10^{-9}$</td>
<td>2.0-1.4</td>
</tr>
<tr>
<td>5.67-5.84</td>
<td>Step</td>
<td>$\phi 60$</td>
<td>3</td>
<td>$\approx 20$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.0</td>
<td>CRS</td>
<td>$\phi 50$</td>
<td>5</td>
<td>19.8</td>
<td>320</td>
<td>44</td>
<td>11.9</td>
<td>17</td>
<td>$8.8 \times 10^{-10}$</td>
<td>2.2</td>
<td>$10^{-7}-10^{-9}$</td>
<td>2.0-1.4</td>
</tr>
<tr>
<td>7.0</td>
<td>CRS</td>
<td>$\phi 50$</td>
<td>2</td>
<td>23</td>
<td>145</td>
<td>40</td>
<td>9.3</td>
<td>24</td>
<td>$1.6 \times 10^{-9}$</td>
<td>3.4</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
The preconsolidation pressures showed an unusual profile, Fig. 34. Down to 4 metres depth, the soil seemed to be normally consolidated for a ground water level about 1.5 m below the ground surface, while the soil in the bottom was normally consolidated for a ground water level only 0.5 m below the surface.

![Preconsolidation profile at Antoniny site, Bialosliwie.](image)

The results from the oedometer tests gave unusually large deformations up to the preconsolidation pressures and consequently very low recompression moduli. This is normally interpreted as a sign of disturbance, but the results were consistent for all samplers and tests.

It was later found that the effective stresses in the ground were very low due to artesian water pressure and the soil was overconsolidated in spite of the low preconsolidation pressures. This usually means that the soil has swelled due to unloading and that the recompression modulus becomes relatively low. Recompression moduli calculated from overconsolidation ratio and empirical swelling characteristics were of the same order as the measured values.

The artesian water pressures also largely explain the unusual preconsolidation in the profile.

The compression characteristics measured in the oedometer tests can thus be expected to be fairly representative for the soil in situ.
The coefficients of consolidation evaluated from the different tests vary greatly in the peat.

As the highest values are measured in the tests with the largest specimens, incomplete saturation can be assumed to be the main reason for this inconsistency. The permeabilities and coefficients of consolidation for the peat measured in the oedometer tests are thus not quite relevant for the field conditions.

The coefficients of consolidation evaluated from the step-loaded tests on calcareous soil are consistently somewhat lower than the corresponding values from CRS tests, which indicates that a certain amount of creep occurred in the tests.

The maximum coefficients of secondary consolidation ranged from 2.6 -2.9% per log cycle of time in the peat and 2.0 - 2.1%/log t in the calcareous soil.

Tests have been performed at DG in order to measure the coefficient of earth pressure during consolidation $K_0$. Drained triaxial tests have then been performed to estimate "moduli of elasticity" to be used in deformation analyses. The samples in the tests have first been $K_0$-consolidated and then sheared with constant horizontal stress. The "modulus of elasticity" (Young's modulus) and its variation with deformation were measured during the shearing phase on samples with different consolidation stresses.

Other tests have been performed to measure the anisotropic consolidation properties in a triaxial apparatus equipped with an ultrasonic measuring device to measure lateral strains. (Wolski et al 1985)

4.2 Shear strength characteristics

The undrained shear strength as measured in the routine fall cone tests conformed to the undrained shear strength from field vane tests, Fig. 35.

The strength of the soil was also investigated by drained and undrained direct simple shear tests and drained and undrained triaxial tests. The latter were performed as both active compression and passive extension tests. The equipments used are specially designed for soft soils and have been described by Larsson (1981).
Undrained shear strength, kPa

- average of field vane tests
- fall-cone tests
(all tests corrected according to SGI 1984)

Fig. 35. Initial undrained shear strength at Antony site as measured by field vane tests and fall cone tests.

Drained direct simple shear tests were run on samples from 1.5 and 5.8 metres with vertical stresses from 15 kPa, which is the lower limit for the apparatus, and upwards. Peak failure was not obtained in any of the tests, but the drained shear strength was evaluated according to Swedish practice as the shear stress at 15% deformation. The drained shear strengths thus obtained were $\tau_{rd} = 0.35 \sigma'$ for the peat and $\tau_{rd} = 0.34 \sigma'$ for the calcareous soil.

When the tests are corrected for dilatancy effects the internal angle of friction at constant volume becomes just below 30° for both peat and calcareous soil.

Undrained direct simple shear tests were also performed but due to the limitations of the apparatus the samples had to be preconsolidated to an elevated effective vertical stress higher than the initial stresses as well as the initial preconsolidation pressure. A new preconsolidation pressure of 50 kPa was chosen. The undrained shear strength at this stress level was $\tau_{fu} = 0.33 \sigma'_p$ for peat and $\tau_{fu} = 0.30 \sigma'_p$ for the calcareous soil.
Drained triaxial tests were performed as active tests. The tests were stopped at 15% axial deformation. At that deformation shear failure was neither obtained nor approached. The tests followed the usual pattern for very soft soils with a rather stiff response for stresses up to the preconsolidation pressure. At the preconsolidation pressure, large volumetric compressions started and the stress increased only slowly with increasing vertical deformation. The angle of internal friction corrected for volume change was found to be about 32° for the peat and about 30° for the calcareous soil.

Undrained triaxial tests were performed as active and passive tests on samples reconsolidated to very low "in situ" stresses. Active tests were also run on samples which had consolidated for stresses above the in situ stresses to investigate the increase in shear strength with preconsolidation pressure and the effective strength parameters.

For the peat the undrained tests at in situ stresses gave an average active undrained shear strength of 6.8 kPa and a passive undrained shear strength of 5.0 kPa. The corresponding values for the calcareous soil at 5.7 m depth were 8.8 kPa and 5.0 kPa. The failure deformations were for both soils about 5 - 7% axial strain. Typical stress-strain curves are shown in Fig. 36.

The stress paths in the undrained triaxial tests gave effective stress parameters of $c' = 2$ kPa and $\phi' = 30^\circ$ for the peat as well as for the calcareous soil, Fig. 37.
Fig. 36. Stress-strain curves in undrained triaxial tests on soil from the Bialosliwie site.

Fig. 37. Effective stress paths in undrained triaxial tests on soil from the Bialosliwie site.
A comprehensive series of tests, including triaxial tests as well as direct simple shear tests, has later been carried out at DG to investigate the effect of consolidation stress on the undrained shear strength. It was then found that the undrained shear strength depended on the effective stress level before shear in the soil. The effective stress level ESL is related to the initial preconsolidation pressure in the soil \((\sigma'_p)_0\) and is expressed as

\[
ESL = \frac{(\sigma'_p)_0}{\sigma'_v}
\]

where \(\sigma'_v\) is the vertical effective stress before shear. The normalized undrained shear strength was found to be a bilinear function of log ESL so that

\[
\tau_{fu} = \sigma'_v \cdot S \cdot (ESL) \quad ESL \leq 1
\]

\[
\tau_{fu} = \sigma'_v \cdot S \cdot (ESL) \quad ESL > 1
\]

where

- \(S\) = ratio of normalized undrained shear strength in the initial normally consolidated state \((\tau_{fu}/\sigma'_v)\).
  This ratio varies with initial preconsolidation pressure.

- \(m_{nc}\) = slope of the relation between log \((\tau_{fu}/\sigma'_v)\) and log ESL in the normally consolidated state \((ESL \leq 1)\).

- \(m_{oc}\) = slope of the relation between log \((\tau_{fu}/\sigma'_v)\) and log ESL in the overconsolidated state \((ESL > 1)\).

The results are in good agreement with the preliminary investigations at SGI. They show, however, that the usual assumption that the undrained shear strength is a direct function of the preconsolidation pressure is an oversimplification for this type of soil.

Taking all undrained triaxial and direct simple shear tests into account the initial undrained shear strength is evaluated as in Table 5.
Fig. 38. Normalized undrained shear strength versus normalized effective stress level from laboratory tests.

Table 5. Undrained shear strength from triaxial tests and direct simple shear tests.

<table>
<thead>
<tr>
<th>Soil</th>
<th>Undrained shear strength, kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( t_{active} )</td>
</tr>
<tr>
<td>Peat</td>
<td>6.8</td>
</tr>
<tr>
<td>Calc. soil</td>
<td>8.8</td>
</tr>
</tbody>
</table>

The results from the more advanced laboratory tests are compared to the results from vane shear tests and fall cone tests in Fig. 39.
The average shear strength ($\tau_{\text{AVERAGE}}$) agrees closely with the corrected field vane and fall cone tests in the peat layer, but is about 20 percent lower in the calcareous soil. This difference probably partly depends on sample disturbance but factors such as membrane corrections and measuring accuracy may also play roles when the material is so soft and has such a low strength as in this case. Furthermore, the correction factors for vane shear tests and fall cone tests originate from experience with more ordinary soils and their validity for an almost pure calcareous soil may be questioned. The field vane tests and the fall cone tests gave unusually high shear strength values in relation to the preconsolidation pressures. According to the SGI recommendations, there is then a considerable risk that the shear strength values will have to be reduced more than is done with the general correction factors.

Further oedometer tests have been performed at SGI and DG to establish the preconsolidation pressures at the end of stages 2 and 3.
4.3 Yield envelope

To enable a prediction of deformations with an elasto-plastic soil model, the different stress ranges where elastic strains occur and where the strains become plastic have to be separated. This is often done by defining a yield envelope in a $q - p'$ (deviatoric stress - isotropic effective stress) stress space which separates the two types of behaviour (Schofield and Wroth 1968).

The most common theoretical soil models assume yield surfaces whose locations are dependent solely on the void ratio of the soil and whose shape is independent of the stress history of the soil. Laboratory tests on natural soft clays, however, have shown that the shape of the yield surface is strongly affected by the stress history of the soil. Thus Tavenas and Leroueil (1977) suggest that the yield envelope for soft soils can be described by an ellipse centered around the $K_0$-line during consolidation and Larsson and Sällfors (1981) suggest that the yield envelope can be defined by the preconsolidation pressures in (usually) vertical and horizontal directions and the Mohr/Coulombian failure lines.

In order to determine the yield envelope for the calcareous gyttja at the Antoniny site several triaxial tests were performed at DG (Lechowicz and Szymanski 1987).

Two series of tests were run. In the first series of tests, the samples were consolidated along a stress-path with $K_0 = 0.45$ and in the second series with $K_0 = 0.6$. In both series, the samples were consolidated for a vertical stress of 205 kPa, whereupon they were unloaded. After unloading, the samples were reloaded in small drained steps along stress paths with constant $q/p'$ ratios.

The yield points were estimated from the stress-volume change curves as the points where large volume changes start, similar to the evaluation of a preconsolidation pressure from an oedometer test.

The yield points and various yield envelopes are shown in Fig. 40. As can be seen in the figure, the yield points along stress paths with high horizontal stresses strongly deviate from the Cam-clay model. On the other hand, they relatively closely agree both to the shape suggested by Tavenas and Leroueil (1977) and the more closely defined shape suggested by Larsson and Sällfors (1985). The yield envelope for the calcareous gyttja at the Antoniny site can thus be described in the same way as the yield envelopes for natural clays and its shape is strongly dependent on the consolidation stresses.
Fig. 40. Yield points and yield envelope for calcareous soil.
4.4 Geodrain tests

The low shear strength of organic soils often necessitates some kind of soil improvement or special construction procedure. One of the methods most often used in engineering practice is preloading and construction by stages. This method, however, requires that the soil is allowed to consolidate for the imposed loads before any improvement can be taken into account. The consolidation process often takes a long time, especially in highly compressible soils with low permeabilities, such as gyttja and highly decomposed peat. As a relatively short construction time is often important, the consolidation process is often accelerated by use of vertical drains. Vertical drains, both sand drains and prefabricated drains of various types, have been used with good results in clays. Good results have also been obtained in Sweden by using sand drains in organic soils.

In order to check the effectiveness and durability of prefabricated vertical drains in organic soils, a special investigation was made at DG in addition to the installation of prefabricated drains with paper filters under one of the embankments. This investigation was made in order to find an answer to the following questions:

- What is the rate of deterioration of drains with paper and polyester filters in organic environments

and

- How does this affect the discharge capacity of the drains?

To study these effects, thin-walled perforated steel tubes were pushed into the soil outside the test embankment with vertical drains, Fig. 41. The steel tubes had a diameter of 320 mm and a length of 6 m. A prefabricated drain was installed in each tube. Two types of drains were used, one with a polyester filter and one with a paper filter. The tubes with the drains were then left in the ground for some time and then the whole tubes containing soil and drains were pulled out, Fig. 42.

The first tubes were pulled out 250 days after the installation of the drains and the following tubes were pulled out after 500 days and 1,000 days in the ground. The tubes were cut in pieces and the outer steel tubes were then removed so that samples of soil with a central drain and dimensions 150 mm diameter and 300 mm length could be trimmed.

"Undisturbed" samples prepared in this way were mounted in a special triaxial cell developed at DG to evaluate the discharge capacity of the drain, Fig. 43.
Fig. 41. Location of perforated tubes with prefabricated drains.

Fig. 42. Tube with a prefabricated drain being pulled out of the ground.
Fig. 43. Laboratory equipment for testing of discharge capacity of prefabricated drains and photo of a tested drain. This drain originally was equipped with a paper filter which has deteriorated after installation in the ground.
The triaxial cell has specially designed base and top plates with slots for insertion of the ends of the drains. The slots are connected to a water flow system by large diameter tubes enabling transmission of water through the drains without interfering flow resistances in the measuring system.

The samples were first consolidated to the in situ stresses and the discharge capacity of the drains was measured by unit hydraulic gradient. The consolidation stresses were then increased in steps up to 250 kPa and the discharge capacity was measured after each step.

The values for initial discharge capacity were obtained by testing laboratory prepared samples. These samples were formed of remoulded peat or organic calcareous soil with drains in the centre. The samples were then consolidated for 10 days.

The initial discharge capacities for the two types of drain were almost equal. They were about 2,400 m³/year at very low confining pressures. In both cases, they dropped by about 40 per cent at very high confining pressures, Fig. 44.

![Fig. 44. Influence of stress and time on discharge capacity.](image_url)
The discharge capacity for the drains that had been in the natural ground for some time decreased considerably. In the case of the polyester filters, the discharge capacity at low confining stresses decreased by about 25 per cent during the first 250 days in the ground but the decrease thereafter was very moderate. In the organic calcareous soil, the discharge capacity decreased with increasing confining stress in a way similar to the initial discharge capacity. In the highly decomposed peat, the decrease in discharge capacity with increasing confining pressure became higher, which indicates that some clogging of the drains may occur due to organic matter being squeezed through the pores of the filter.

The decrease in discharge capacity was much greater for the drains with paper filters. Already after 250 days in the ground, the discharge capacity at low confining stresses had decreased by 50 per cent and this decrease continued with time. After 1,000 days, less than 10 per cent of the original discharge capacity remained. Furthermore, the discharge capacity rapidly decreased with increasing confining stress and became very small at higher pressures.

The reason for the large environmental effects on the paper filters could readily be observed at a visual inspection of the filters. After 250 days in the ground, virtually nothing remained of the paper filters. The remaining discharge capacity was due to the fact that the soil had not been squeezed into the channels in the plastic core, which were mainly standing open. With time and increasing pressure, however, they became more and more clogged with soil. The polyester filters, on the other hand, seemed practically unaffected by the environmental conditions.

The decrease in discharge capacity with time and increasing pressure is normally not a serious problem, as the discharge capacity required for the drain to function is relatively small. It can thus be concluded that drains with polyester filters should work as intended also in organic environments.

However, the function of drains with paper filters in organic environments must be questioned. In the case described here the paper was totally destroyed in less than 250 days. Even if some discharge capacity remained in the tests, it should be considered that the drains were protected against mechanical actions and stress increases similar to those occurring in a consolidation process under a loaded area.
5. GEOTECHNICAL CONDITIONS

This chapter summarizes the geotechnical conditions before the test embankments were constructed. The presentation is consciously simplified to give a general idea of the soil conditions.

The upper two metres of the soil consists of amorphous peat which is very calciferous. The ground surface is covered with grass vegetation and there is an abundance of cracks and root channels in the upper parts of the peat. Further down, the carbonate content increases and the organic material decreases and occurs as gyttja. At 4m depth there is a yellow-white layer of almost pure carbonate soil (marl). Further down, the soil is calcareous with a content of calcium carbonates of 80 to 90 per cent. The organic content there is about 5 per cent. At 7 m depth, the organic content increases somewhat and the soil is classified as a calcareous gyttja. Below 7.8 m depth there is dense sand.

5.1 Stress conditions

The bulk density of the organic and calcareous soil is low. This fact, combined with a high ground water level and artesian water pressure in the underlying sand, has resulted in very low effective stresses in the natural ground. No extreme variations in the ground water situation have occurred during the construction period, even if the area has been seasonally flooded and the pore pressure in the sand layer has varied somewhat. As an average condition, it can be assumed that the free ground water level is at a depth of 0.2 m below the ground surface. Down to 2 m depth the pore water pressure can be assumed to be hydrostatic from the free ground water level. In the underlying sand at 7.8 m depth the water pressure is artesian and has been measured as corresponding to a water head 1.0 to 1.5 metres above the ground surface. This means that there is an upcoming ground water flow. The gradient seems to be fairly constant between 2.0 and 7.8 m depth.

The initial effective stress situation is relatively unusual. The effective vertical stresses in the compressible layers are only a few kPa and are almost constant with depth. The soil is overconsolidated, but the overconsolidation ratio varies depending on the fluctuations in ground water level and artesian water pressure in the underlying sand. Due to the low stresses, the degree of overconsolidation is very sensitive to the ground water situation.

The site is usually flooded in the spring and at other periods the ground water level may be lower than the normal level. A ground water level at a depth of 0.5 m has been registered during the construction period. Owing to the short drainage paths and the high overconsolidation, the effects of changes in the ground water level will relatively
quickly affect the stress situation in the peat. The effect of changes in free ground water level and artesian water pressure in the calcareous soil will take longer to penetrate the entire layer. The normal stress situation is shown in Fig. 45.

Fig. 45. Initial soil properties at Antdoniny site.

5.2 Shear strength

The undrained shear strength of the soil has been measured in a large number of vane shear tests in the field and by fall cone tests during the routine investigations in the laboratory. Series of active compression and passive extension triaxial tests and direct simple shear tests were run on samples from two levels. The undrained shear strength profile is shown in Fig. 39. The evaluated undrained shear strengths are very low and only amount to 5 - 6 kPa in the peat and 7 - 8 kPa in most of the calcareous soil. There is no unique relation between the undrained shear strength and the preconsolidation pressure in these soils. A better relation for the variation of shear strength with stress can be obtained by an exponential function. The parameters in this function change at the preconsolidation pressure.

The undrained shear strength is strain rate dependent and it is therefore important to follow standardized testing procedures. Shear strength
values obtained in field vane tests have to be corrected. Such correction factors related to the liquid limit of the soil have been proposed by SGI (Larsson et al 1984). These correction factors are mainly based on experience from other types of soil but a comparison between the average undrained shear strength from qualified laboratory tests and the corrected strength from field vane and fall cone tests shows that the factors are of the right order also for the soils at the Antoniny site.

The effective strength parameters for both the peat and the calcareous soil have been found to be \( c' = 2 \text{ kPa} \) and \( \phi' = 30^\circ \) within the stress levels of interest.

5.3 Compressibility

The compression characteristics have been determined in a large number of incremental oedometer and compressiometer tests, as well as oedometer tests with constant rate of strain. Samples taken with different types of samplers have been tested. There is no obvious difference in evaluated preconsolidation pressures in the different tests or the different samples. The evaluated stress-strain curves from incremental tests and tests with constant rate of strain were also compatible. The stress-strain curves obtained from different samples in the peat indicate, however, that the samples taken with the Swedish standard piston sampler in this material were slightly more disturbed than the other samples.

The preconsolidation pressures corresponded to a soil normally consolidated for a ground water level 1.5 m below the ground surface down to 4 m depth and a ground water level 0.5 m below the ground surface below 5 m depth. The prevailing ground water conditions are a free ground water level only 0.2 m below the ground surface and artesian water pressures in the sand at 7.8 m depth. This means that in spite of the low preconsolidation pressures the soil is overconsolidated.

The soil is highly compressible with moduli just after passing the preconsolidation pressure of the order of 120 kPa in the peat and 300 kPa in the calcareous soil. Also the recompression moduli at stresses below the preconsolidation pressure are fairly low.

The initial permeabilities are low, ranging from \( 2 \cdot 10^{-8} \text{ m/s} \) in the peat down to \( 4 \cdot 10^{-10} \text{ m/s} \) in the calcareous soil. They decrease significantly with compression. The low moduli combined with the low permeabilities make the consolidation process time-consuming.
6. OBSERVATIONS OF THE TEST EMBANKMENTS

6.1 Deformations

The vertical movements in the soil were observed by four types of measuring devices: hose settlement gauges, surface plates, screw plates and magnetic settlement gauges. The magnitude and the distribution of total vertical settlement was measured at the contact surface between the original ground and the embankment by hose settlement gauges. The total settlement distributions at various times for the two test embankments are shown in Fig. 46.

The compressions of the peat layer and the calcareous soil have been separated by levelling the surface plates and the screw plates at the interface between the two types of soil, Figs. 47 and 48.

The distribution of settlements with depth has been measured by the magnetic settlement gauges and is shown in Fig. 49. Due to the large deformations problems with buckling of the tubes for the measuring device occurred with time and at the end of the observation period the device could not be lowered to the bottom in all the tubes.

The horizontal displacements were measured by inclinometers in and outside the slopes of the embankments. The results are shown in Figs. 50 and 51.

The deformations in the soil developed largely as follows:

- In the FIRST STAGE, when 1.2 metres of sand had been in place for about 150 days, the vertical settlements for both fills were about 0.4 metres. The maximum horizontal displacements for the fill without drains were about 0.15 m while they were only about 0.05 m for the fill with vertical drains. The rate of consolidation seems to have been higher under the fill with drains in this stage.

- In the SECOND STAGE, the fill without drains was increased by 1.3 m of sand and the fill with drains by 1.5 m of sand. The settlements a year later had increased to 1.1 m for the fill without drains and to 1.2 m for the fill with vertical drains. The maximum horizontal deformations had increased to about 0.35 m for both fills.

- In the THIRD STAGE, the fill without drains was increased by another 1.4 m of sand and the fill with vertical drains by 1.3 m of sand. Two years after the final load application, the settlements had increased to 1.8 m for the fill without drains and 1.9 m for the fill with drains. The maximum horizontal displacements had increased to about 0.55 m in both cases.
Fig. 46. Distribution of settlements under the test embankments.
Fig. 47. Subsoil settlement under Embankment No. 1. (Settlement gauges).
Fig. 48. Subsoil settlement under Embankment No. 2. (Settlement gauges).
Fig. 49. Subsoil settlement under test embankments (magnetic gauge readings).
Fig. 50. Horizontal displacements under Test Embankment No. 1.
Fig. 51. Horizontal displacements under Test Embankment No. 2.
A comparison between the two embankments shows no significant differences, except for the first load stage where the horizontal movements became much smaller and the rate of consolidation settlements became higher for the embankment with vertical drains. In the second stage on the other hand, the horizontal displacements became larger for the embankment with vertical drains; 0.3 m compared to 0.2 m for the embankment without drains. However, this can be explained by the somewhat larger load increment for the embankment with vertical drains, combined with the very low factor of safety against undrained shear failure. The rates and magnitudes of settlements in stages 2 and 3 and the development of horizontal deformations in stage 3 were almost identical for the two embankments. There is thus some indication of an effect of the vertical drains during the first load stage but none thereafter.

The horizontal deformations developed during the load application and shortly thereafter, but have practically stopped with time. The vertical deformations on the other hand have continued and in no loading stage has any kind of final settlement been obtained, Fig. 52.

The horizontal deformations have caused a corresponding vertical deformation during and just after the uploading phases but their effect on the long-term vertical consolidation process has been limited, Fig. 53.

The difference in maximum horizontal deformations that occurred during stage 1 was evened out in the second load stage under the middle of the slopes. The difference under the toes of the slopes has remained but not increased during stages 2 and 3. The slightly different behaviour at the two locations can be attributed to the geometrical conditions, Fig. 54.

The distribution of vertical and horizontal movements based on the settlement gauges and the inclinometer readings is presented in Figs. 55–58.
Fig. 52. Development of deformations under the test embankments.
Fig. 53. Horizontal displacements versus total settlements under test embankments, Antony site.
Fig. 54. Horizontal displacements under the embankment slopes.
Fig. 55. Distribution of vertical displacements under the test embankments.

BEFORE STAGE 2

Embankment No.1

STAGE 2

STAGE 1

0.1 m

PEAT

CALCAREOUS SOIL / GYTTJA

Embankment No.2

0.1 m

PEAT

CALC. SOIL / GYTTJA

END OF STAGE 2

Embankment No.1

0.0 m

0.25 m

0.50 m

0.75 m

1.0 m

0.1 m

PEAT

CALC. SOIL / GYTTJA

Embankment No.2

0.0 m

0.25 m

0.50 m

0.75 m

1.0 m

0.1 m

PEAT

CALC. SOIL / GYTTJA
BEFORE STAGE 2

Embarkment No. 1

STAGE 2

STAGE 1

PEAT

0.10 m

0.15 m

0.05 m

CALC. SOIL / GYTTJA

SAND

END OF STAGE 2

Embarkment No. 2

PEAT

0.05 m

CALC. SOIL / GYTTJA

SAND

Fig. 56. Distribution of horizontal displacements under the test embankments.
Fig. 57. Distribution of vertical displacements under the test embankments (during stage 3, October 8th, 1986).

Fig. 58. Distribution of horizontal displacement under the test embankments (during stage 3, October 8th, 1986).
6.2 Pore pressures

The pore pressures in the ground below and outside the embankments have been measured with BAT piezometers. The piezometer pipes were protected by outer pipes in order to follow the soil movements at the level where they were installed. However, the piezometer readings showed obvious signs of pushing and later inspections of the positions of the filter tips have shown that almost all piezometers under the embankments have been dislocated. The lowest piezometers under the centre of the embankments have even been pushed out of the calcareous soil and are standing on top of the sand layer. The initial and final positions of the piezometers are shown in Fig. 59.

The results of the total pore pressure measurements under and outside the embankment without drains are shown in Fig. 60. The corresponding measurements under the embankment with vertical drains are shown in Fig. 61.

The interpretation of the pore pressure readings is not straightforward as there are several factors that affect the readings. First, there is the pore pressure response to the applied load. Then there is the increase in pore pressure as the piezometer follows the settlements to a lower level. This increase is complicated by the artesian water pressure. An assumption has to be made about the distribution of the artesian water pressures after the excess pore pressures have dissipated. Furthermore, there is the effect of pushing both in change in level and in pore pressure generation due to the pushing. The effect of the latter change in level is almost certainly non-hydrostatic because of the artesian water pressures. Finally, there is the effect of varying external ground water conditions. Under the embankment with vertical drains there is a further complication as the drains, if they were functioning would affect the distribution of the artesian excess pore pressure.

An attempt has been made to estimate the excess pore pressures at the changing levels of the piezometer tips. The excess pore pressures are calculated in relation to a changing "normal ground water condition" where the artesian excess pore pressure has a constant gradient between the interface between the compressible layer and the sand and a point that was originally located 2.0 m below the ground surface, Figs. 62 and 63. The estimated values should be treated with great caution. Much of the pore pressure dissipation is due to the fact that the pore pressure tips are gradually pushed out of the zones with maximum excess pore pressures. The lower filter tips are even pushed right out of the compressible soil and into contact with the underlying sand. A number of conclusions can be drawn, however.
Fig. 59. Location of the BAT piezometers under the test embankments.
Fig. 60. Measured pore pressures (u) under Test Embankment No. 1.
Fig. 61. Measured pore pressures (u) under Test Embankment No. 2.
Fig. 62. Estimated excess pore pressure $\Delta u$ under the centre of Test Embankment No. 1, without vertical drains.

Fig. 63. Estimated excess pore pressure $\Delta u$ under the centre of Test Embankment No. 2, with vertical drains.
The greatest increases in pore pressure were observed under the centre of the embankments. Due to the pushing of the piezometers, pore pressure responses significantly higher than the vertical load increase were recorded during the loading phase. These peak pore pressures rapidly disappeared after the loading was concluded. These effects are believed to be due to pushing alone, as they were most pronounced for the piezometers subjected to most pushing and considerably less in the zones where the shear deformations were greater but the pushing effects smaller.

The excess pore pressures in the peat layer became relatively small and rapidly dissipated, indicating short drainage paths and a high permeability in the upper zone with cracks and root channels.

The variation in the external ground water conditions clearly affected the pore pressures in the soil profile under as well as outside the embankments.

No pore pressure equalization has been obtained in any of the loading stages. At the end of the first stage, there were remaining pore pressures of the order of 10 kPa. At the end of the second stage, which lasted for a year, there were excess pore pressures of the order of 20 kPa and at the end of the third stage there remained high excess pore pressures two years after the final load application. In the middle of the calcareous soil layer, they were probably of the order of 20 – 30 kPa, but there was no piezometer left at that level at that time. The pore pressure development during the first loading stage was not followed in detail. For the subsequent two stages, there was no very significant difference in the measured developments and dissipations of pore pressures under the two embankments. Minor differences cannot be interpreted due to the uncertainty of the effects of pushing and the final ground water conditions to which the excess pore pressures should be related.
6.3 Shear strength increase

The undrained shear strength of the soil was measured by field vane tests in situ. An investigation was first carried out in order to evaluate the influence of various factors in equipment and testing procedure on the shear strength values obtained in field vane tests. This investigation also comprised comparative tests according to both Swedish and Polish standard procedures and equipments (see Chapter 3.2). The differences in results were small, but only results obtained in tests according to the Polish standard method, which was most frequently used, will be considered here.

During the consolidation process, vane shear tests were performed at different locations under the test embankments before each new construction stage and also during the second and third stage (Figs. 2 and 3). At each location, tests were performed at every 0.5 m depth. The uncorrected measured strength values under the embankments are shown in Figs. 64 and 65.

The profiles of shear strength values obtained by the field vane tests show a considerable increase in undrained shear strength due to the loading and subsequent consolidation. The highest strength increase was measured under the centre of the embankment and the increase was most evident in the peat layer. A smaller increase in undrained shear strength was obtained under the slope of the embankment, while the measured shear strength values under the toes of the slopes and outside the embankments remained practically unchanged.

The distribution of the vane shear strength values before the second and third stages of construction is shown in Fig. 66. It can be observed that at the beginning of the consolidation process, zones with increased shear strength are created close to the ground surface and at the sand layer. Due to the drainage conditions, these two zones do not expand towards the centre of the entire layer of soft soil but towards the middle of the layer of calcareous soil. The magnitudes and distributions of the shear strength increases are similar for the two embankments.

The effective vertical stresses estimated from calculations of total stress distribution and measured pore water pressures under the embankment without drains are shown in Fig. 67, together with initial effective stresses and initial preconsolidation pressures.

The relation between estimated effective stresses and measured shear strength values shows large increases in shear strength values when the effective stresses exceed the initial preconsolidation pressures. A small increase in measured strength values may be detected also in the stages where the effective stresses increase but still remain below the initial preconsolidation pressures.
Fig. 64. Profiles of uncorrected vane shear strength values under Embankment No. 1.
Fig. 65. Profiles of uncorrected vane shear strength values under Embankment No. 2.
Fig. 66. Distribution of vane shear strength values under Test Embankment Nos. 1 and 2.
Fig. 67. Estimated effective stresses and shear strength values obtained in field vane shear tests at different stages of the loading and consolidation process.
The increase in undrained shear strength is usually predicted from some relation where the normalized shear strength is a function of the normalized effective vertical stress. The normalized effective vertical stress is usually expressed by the overconsolidation ratio \( \text{OCR} = \frac{\sigma'_p}{\sigma'_v} \). In the overconsolidated stress range, the overconsolidation ratio changes with the current stress because the preconsolidation pressure is constant, but in the normally consolidated stress range \( \text{OCR} \) is always 1 as the preconsolidation pressure changes with the current effective vertical stress.

For a better description of the change in stress state, especially in the normally consolidated state, a normalized effective stress level \( \text{ESL} \) has been proposed instead of \( \text{OCR} \), (Bergdahl et al 1987). The normalized effective stress level is calculated from

\[
\text{ESL} = \left( \frac{\sigma'_p}{\sigma'_v} \right)_o
\]

where

\( (\sigma'_p)_o = \text{initial preconsolidation pressure} \)

The shear strength values obtained in the field vane tests (FVT) have been normalized against the estimated effective vertical stresses. The same has been done with the undrained shear strengths estimated by correction of the vane shear strength values with respect to the liquid limit (cFVT). The corrections have been made according to the SGI recommendations (Larsson et al 1984). The correction factors were thus 0.5 for values obtained in the peat and between 0.6 and 0.7 for values obtained in the calcareous soil.

The relations between normalized shear strength values from field vane tests and normalized effective stress level for peat and calcareous soil are shown in Fig. 68 a. The relations between normalized undrained shear strengths from corrected vane shear tests and normalized effective stress level for peat and calcareous soil are also shown.

The results from laboratory tests performed on peat and calcareous soil specimens have been normalized in the same way. The results from anisotropically consolidated undrained triaxial compression tests (CK \(_o\) UTC) and consolidated undrained direct simple shear tests (DSS) are shown in Fig. 68 b.

The results indicate that the ratio between normalized undrained shear strength and normalized effective stress level changes not only in the overconsolidated state but also in the normally consolidated state.
a) Field vane shear tests.

![Normalized undrained shear strength versus normalized effective stress level](image)

b) $C_{k0}$ UTC and DSS tests.

![Normalized undrained shear strength versus normalized effective stress level](image)

Fig. 68. Normalized undrained shear strength versus normalized effective stress level from in situ and laboratory tests.
The field vane test cannot be considered a very good tool for measuring shear strength increases under embankments with limited widths. The measured strength values are mainly influenced by the horizontal stresses which do not increase as much under narrow embankments as under wider loadings. The current embankments, however, were relatively wide in relation to the thickness of the compressible layers. The trend in the field vane tests is also confirmed qualitatively, if not absolutely quantitatively, by the results from the laboratory tests.

The prediction of increase in undrained shear strength with effective stress level can be made according to the following relations (Bergdahl et al 1987):

\[
\tau_{fu} = \sigma'_{v} S(ESL)^{m_{nc}} \quad \text{ESL} \leq 1
\]

\[
\tau_{fu} = \sigma'_{v} S(ESL)^{m_{oc}} \quad \text{ESL} > 1
\]

where:

- \( S = \) Undrained shear strength at ESL=1 normalized against the initial preconsolidation pressure. Thus S varies with change in initial preconsolidation pressure.

- \( m_{nc} = \) Slope of the relation between \( \log (\tau_{fu}/\sigma'_{v}) \) and \( \log (\text{ESL}) \) in the normally consolidated state (ESL\( \leq 1 \)).

- \( m_{oc} = \) Slope of the relation between \( \log (\tau_{fu}/\sigma'_{v}) \) and \( \log (\text{ESL}) \) in the overconsolidated state (ESL\( > 1 \)).

The parameters \( S, m_{nc} \) and \( m_{oc} \) have been evaluated from the laboratory and field tests for both types of soil. The value of \( m_{oc} \) was about 0.8 for both types of soil. The values of \( S \) were fairly high and were about 0.4 - 0.5 for peat and 0.35 - 0.45 for calcareous soil. The decrease in normalized undrained shear strength with decreasing normalized effective stress level (i.e. at increasing preconsolidation pressure) is most pronounced in peat with a value for \( m_{nc} \) of about 0.15 - 0.3. The corresponding value for calcareous soil is about 0.1 - 0.2.

The variation in undrained shear strength with stress level for a soil with strength parameters \( S=0.5, m_{nc}=0.8 \) and \( m_{oc}=0.2 \) is shown in Fig 69. A comparison between predicted shear strengths calculated from the relations suggested by Jamiołkowski et al (1985), Larsson (1980) and the present relation shows that up to the initial preconsolidation pressure the undrained shear strength predicted from the relation of Jamiołkowski et al and the present relation is the same.
For stresses increasing above the initial preconsolidation pressure, however, the undrained shear strengths calculated by the present relation become lower than those which the previously suggested relations would have given. The relative difference increases with the increase in stress. The present relation is in accordance with the curved relation between shear strength and normal stress found for most soils.

Fig. 69. Predicted undrained shear strength versus effective vertical stress. $\tau_{uu}$=undrained shear strength at the initial preconsolidation pressure, reference strength.
6.4 Influence of vertical drains

Under one of the embankments vertical prefabricated drains had been installed in a 1.2 m square grid. Drains with paper filters and plastic cores were used. As shown by the special laboratory investigation (Chapter 4.4) the paper filters deteriorated rather quickly and the long-term function of this type of drain in the actual environment could be questioned.

The field observations showed a pronounced effect of the vertical drains during the first load stage where the horizontal deformations became smaller and the settlements became faster under the embankment with drains as compared to the embankment without drains. In this load stage the applied loads were exactly the same. In the following load steps and for most of the consolidation process, however, there was no significant effect of the drains and it seems likely that they have been clogged after a short time.

This is not a measure of the effectiveness of vertical drains as a method. That sand drains work very well in similar soils has previously been found in projects carried out at SGI and drains with polyester filters would, according to the special investigation, have functioned well for the whole construction period. The limited effects obtained by the drains in this particular case can be attributed to the choice of drains with paper filters in combination with the long construction period due to loading in stages. It is quite possible that even the paper filters would have worked sufficiently well in a more normal loading case where the loading is applied in a single step. The choice of the particular drains, however, was made deliberately as the durability of paper filters in severe environmental conditions has been debated for a long time.

The results show that although drains with paper filters have been found to function very well in many soft clays they should rather be avoided in the harsh environmental conditions in organic and calcareous soils. In these soils more resistant filters, such as polyester filters or sand drains, should be used.
7. PREDICTION OF DEFORMATION AND STABILITY

7.1 Prediction of deformations and course of consolidation

7.1.1 General

Predictions of deformations and stability can be carried out with different degrees of sophistication depending on the nature of the problem. In the simplest forms of settlement analysis, the soil conditions are simplified to one or a few layers with uniform and constant deformation properties. The deformations are then calculated as purely elastic shear deformations or as vertical compression disregarding horizontal deformations. The two types of deformation can also be calculated separately and thereafter added.

The elastic parameters are often estimated by an empirical relation coupled to the undrained shear strength of the soil. Alternatively they can be determined by more elaborate field or laboratory tests.

In peat, the compression characteristics are often estimated from an empirical relation which is usually related to the natural water content of the soil. For more accurate calculations and in all other types of soft soils the compression parameters are usually evaluated from oedometer tests.

Elastic deformations are here considered to be instantaneous and the effective stress-strain relations in compression are independent of time. As compression entails pore water being squeezed out of the pores, the consolidation process takes a certain time due to the hydraulic flow resistance. This time is often calculated by assuming that the modulus of compression is constant and that the permeability of the soil is constant. (Terzaghi 1923, 1924). These types of calculations are usually called 'conventional settlement analysis'.

The calculations can then be made more detailed taking soil variability into consideration by dividing the soil profile into more sublayers and also by taking the variation in deformation properties with stress level into account. In the latter case, the modulus of elasticity is not assumed to be a constant, but a variable where the modulus decreases with increasing shear stresses. The compression modulus also varies with stress, so that there is a higher modulus at vertical stresses lower than the preconsolidation pressure, a drop in modulus at the preconsolidation pressure and a gradually increasing modulus at even higher stresses. The stress-strain relation in compression is usually evaluated from oedometer tests.

When large deformations occur, there are more aspects to take into consideration. The applied load may vary as parts of it or the upper crust
become submerged into the ground water due to the settlements. The drainage paths change as the geometry changes during consolidation and the permeability changes as the void ratio and porosity change. An analytical solution of this complicated process requires very complicated mathematical equations.

Improvements of the Terzaghi equation has been suggested by Gibson et al (1981) and Young and Ludwig (1984), among others.

The consolidation analysis can be improved by performing the calculations in time steps. The initial modulus and permeability are used in the first step and these parameters, together with load and geometry, are then updated for the actual stresses and deformations after each time step (Helenelund, 1951).

Further theoretical elaboration is possible taking two- and three-dimensional water flow into account, (e.g. Biot, 1941, Tan, 1961).

The division of the deformations into immediate shear deformations and time-bound vertical compression is artificial since they are both components of a continuous process where shear deformations occur also during consolidation. There are a number of more elaborate ways of estimating the magnitude and distribution of deformations in different directions under constructions.

The deformations can be estimated by using finite elements and theory of elasticity, whereby also elasto-plastic soil models can be used. The soil models can be linear elastic, linear elastic-plastic, hyperbolic-strain hardening (or softening) depending on the degree of sophistication.

The soil models can also be of the critical state type (Schofield and Wroth, 1968) with a stress-space within a yield envelope where the deformations are elastic. When passing this yield envelope, the strains become plastic and strain hardening or softening depending on the combination of shear stresses and normal stresses at which the yield surface was passed. The plastic volume strains can then be estimated using the compression characteristics of the soil and the shear strains by an associated flow rule.

In most critical state models, the soil is assumed to have isotropic properties but also anisotropic models exist (e.g. Runesson, 1978, Larsson, 1981, Magnan and Lepidas, 1987).

The consolidation process with time can be calculated assuming one, two or three-dimensional water flow, with constant or varying permeability and with constant or changing compressibility. An elaborate calculation method using finite differences and an anisotropic soil model has been described by Runesson et al (1980). In this method two and three-dimen-
sional water flow is accounted for and the calculations are made in small steps with continuous updating of all soil parameters as well as the geometry of the problem at the end of each step.

Such calculation methods, however, require a large amount of input data on soil characteristics that are relatively difficult to determine. Normally, the present programs of this type do not account for creep effects. Recently finite element programmes taking two and three-dimensional consolidation, anisotropy and creep into account have been developed at Laboratoire des Ponts et Chaussees and Ecole National des Ponts et Chaussees in Paris (Magnan 1987).

The assumption of stress-strain relations that are independent of time is a gross oversimplification, especially in soft soils. The stress-strain relations are highly dependent on the strain rate and the deformations increase with time even after the hydraulic flow resistance has ceased to be of importance. Moreover, the stress-strain relations are time-dependent also during the time for so-called primary consolidation when there are still excess pore pressures due to the hydraulic time lag. The simplest way to account for the creep deformations, which is to add them after the excess pore pressures have dissipated, is thus inadequate in an elaborate analysis.

An accurate description of the consolidation process taking time effects on compressibility into account, leads to very complex differential equations which can only be solved by numerical methods (e.g. Garlanger, 1972, Szymanski et al, 1983).

The creep effect can be taken into account by using calculations in short time steps with updating of the compressibility of the soil with consideration to time effects as well as all other properties and load and geometry after each load step. Such calculation programs have been developed by Magnan et al (1979) and Mesri and Choi (1985).

The CONMULT-program developed by Magnan et al has been revised at SGI to take new models of soil compressibility and empirical observations into account (Larsson, 1986).

Calculations of settlements taking creep effects into account have so far been almost restricted to one-dimensional consolidation. Calculation programs of the CHALFEM C type (Runesson et al, 1980) have the capability to incorporate creep effects, (and the new French programmes do) but the soil models and the determination of soil parameters are complex and not fully developed.

Consolidation of soils with vertical drains is predicted by assuming one-dimensional vertical settlements and two-dimensional axi-symmetrical horizontal pore water flow. Well-known theories of this type were put
forward by Kjellman (1949), Barron (1949) and Hansbo (1979, 1981), among others. All these theories assume linear stress-strain relationships and constant soil parameters in the consolidation process. The combined effect of horizontal water flow towards the drains and vertical water flow towards horizontal drainage boundaries (and in the case of drains with limited depth also the additional water flow towards the lower ends of the drains) can be calculated by integration or with finite element programs. Effects of aspects such as disturbance at insertion of the drains (smear effects) and well resistance due to limited discharge capacity in the drains can be taken into account in the calculations, but not changing soil parameters and time effects. Calculation programs with short time step and updating of all parameters similar to the programs for one-dimensional consolidation without drains are reportedly under way but not yet at hand.

A special form of settlement prediction is often used in connection with stage-loading and preloading with surcharge. In both cases, the settlements are followed up during the consolidation process and the predicted course of consolidation can be checked and improved. For this purpose, a method developed by Asaoka (1978) has often been used. The method is, however, very sensitive to measuring errors in the early stages of the consolidation process (Eriksson and Fallsvik, 1984). I also assumes that Terzaghi's theory for consolidation is valid. As all changes in parameters during the consolidation process are thereby neglected, the prediction changes depending on the length of the observation period. This period has to be fairly long if a prediction of any use is to be obtained.

In very special cases where not only the settlements but their distribution and the pore pressures and sometimes also the horizontal deformations are measured, the initial assumptions on preconsolidation pressure, modulus of elasticity and drainage boundaries etc. can be checked. The initial, more elaborate predictions can then be adjusted accordingly.
Predictions of settlements and courses of consolidation for the embankments at Antoniny have been made at both DG and SGI and according to different methods with varying degrees of sophistication. In the one-dimensional consolidation analysis, the settlements have been calculated as initial shear deformations and settlements due to consolidation. The total settlements are the sum of these two parts.

The initial settlements have been calculated according to the theory of elasticity. The equations for a rectangular load on a layer with limited depth according to Steinbrenner (1936) have been used. The moduli of elasticity for the different soil layers have been evaluated with consideration to the undrained shear strength and plasticity of the soil and to the shear stress level in terms of calculated factor of safety against undrained shear failure. The following formula has been applied:

\[
E = \frac{\tau_{fu} 215 \ln F}{I_p}
\]

where:
- \( E \) = Modulus for initial deformation
- \( \tau \) = Undrained shear strength from vane shear tests or direct simple shear tests
- \( fu \) = Undrained shear strength from vane shear tests or direct simple shear tests
- \( F \) = Calculated factor of safety against shear failure
- \( I \) = Plasticity index
- \( P \)

This formula has been derived from the results presented by Foott and Ladd (1981) coupled with Swedish and international empirical experience (Larsson, 1986). The harmonic mean of the moduli for the different sub-layers has been used in the predictions of initial shear deformations.

The initial settlements calculated with this empirical correlation between undrained shear strength, plasticity and safety factor against undrained shear failure were 0.10 m, 0.17 m and 0.13 m for stages 1, 2 and 3.

The initial settlements were measured indirectly by measured settlements after load application and by measurements of horizontal movements by inclinometers some time after load application. Both types of measurements include some movements due to time-dependent consolidation and the elastic deformations should therefore be somewhat smaller. The "measured" and calculated initial deformations are compared in Table 6.
Table 6. "Measured" and calculated initial deformations.

<table>
<thead>
<tr>
<th>Stage</th>
<th>&quot;Measured&quot; by hose settlement gauge</th>
<th>Calculated from horizontal move-empirical</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>&lt; 0.11</td>
<td>0.11</td>
</tr>
<tr>
<td>2</td>
<td>0.15-0.20</td>
<td>&lt; 0.18</td>
</tr>
<tr>
<td>3</td>
<td>0.08-0.10</td>
<td>&lt; 0.09</td>
</tr>
</tbody>
</table>

The total calculated settlements in the three stages amount to 0.40 m. The measurements indicate that the calculated values are of the right order of size, but the initial settlements seem in all three stages to have been somewhat smaller than calculated. The initial settlements had a distribution which reflected the lower factor of safety in the outer parts of the embankment. The maximum settlements in stage 1 thus occurred halfway between the toes and the centre of the fill. This picture has largely remained in later stages but the maximum has moved inwards during stage 2 and 3.

The "FINAL" DEFORMATIONS have been calculated with a number of the simpler methods for one-dimensional consolidation analyses. The soil has thereby been divided into two main layers with uniform properties. The settlements have been calculated separately and added to the elastic deformations:

- With application of various empirical methods for estimations of settlements in peat (Ostromecki, 1956, Niesche, 1977 and Drozd-Zajac, 1968). Only Polish relations have been used, as such relations normally are of use only locally. The type of peat found at Antoniny thus is outside the limits for applicability of corresponding relations used in Scandinavia.

- On the basis of results from oedometer tests performed on samples from the middle of the peat layer and the layer of calcareous soil respectively.

- On the basis of the field observations using Asaoki's method.
The results of these calculations are shown in Tables 7 and 8. The small differences between the two embankments are related to the slightly different loads in stages 2 and 3.

Table 7. Predicted "final" settlements of subsoil under Embankment No. 1.

<table>
<thead>
<tr>
<th></th>
<th>Standard</th>
<th>Niesche</th>
<th>Ostromoecki</th>
<th>Drozd-Zajac</th>
<th>Asaoka</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(m)</td>
<td>(m)</td>
<td>(m)</td>
<td>(m)</td>
<td>(m)</td>
</tr>
<tr>
<td>Stage 1</td>
<td>0.28</td>
<td>0.56</td>
<td>0.49</td>
<td>0.12</td>
<td>0.27</td>
</tr>
<tr>
<td>Peat</td>
<td>0.80</td>
<td>0.84</td>
<td>0.80</td>
<td>0.48</td>
<td>0.72</td>
</tr>
<tr>
<td>Stage 3</td>
<td>1.09</td>
<td>1.14</td>
<td>1.03</td>
<td>0.66</td>
<td>0.96</td>
</tr>
<tr>
<td>Stage 1</td>
<td>0.24</td>
<td>-</td>
<td>-</td>
<td>0.17</td>
<td>0.24</td>
</tr>
<tr>
<td>Gyttja</td>
<td>0.57</td>
<td>-</td>
<td>-</td>
<td>0.36</td>
<td>0.60</td>
</tr>
<tr>
<td>Stage 3</td>
<td>0.85</td>
<td>-</td>
<td>-</td>
<td>0.56</td>
<td>1.03</td>
</tr>
<tr>
<td>Stage 1</td>
<td>0.52</td>
<td>-</td>
<td>-</td>
<td>0.29</td>
<td>0.51</td>
</tr>
<tr>
<td>Total</td>
<td>1.37</td>
<td>-</td>
<td>-</td>
<td>0.84</td>
<td>1.32</td>
</tr>
<tr>
<td>Stage 3</td>
<td>1.94</td>
<td>-</td>
<td>-</td>
<td>1.22</td>
<td>1.99</td>
</tr>
</tbody>
</table>

The results obtained with the empirical methods in the peat differ among themselves and in relation to the other two methods, especially at the smallest load. The initially predicted settlements using oedometer results and the calculations using field observations agree fairly well. It should be observed, though, that the settlements calculated from the field observations using Asaoki's method are by no means a final result. Thus, at the end of stage 2, the total settlements amounted to 85% of the predicted values, while the pore pressures indicated that only about 50% of the excess pore pressures had dissipated and the settlements continued at an appreciable rate. At the end of stage 3, the settlements amounted to 95% of the predicted values, 40% of the excess pore pressures remained and the settlements showed no sign of a slowdown.
Table 8. Predicted "final" settlements of subsoil under Embankment No. 2.

<table>
<thead>
<tr>
<th></th>
<th>Standard</th>
<th>Niesche</th>
<th>Ostro-mecki</th>
<th>Drozd-Zajac</th>
<th>Asaoka</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(m)</td>
<td>(m)</td>
<td>(m)</td>
<td>(m)</td>
<td>(m)</td>
</tr>
<tr>
<td>Stage 1</td>
<td>0.26</td>
<td>0.52</td>
<td>0.46</td>
<td>0.10</td>
<td>0.26</td>
</tr>
<tr>
<td>Peat</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stage 2</td>
<td>0.82</td>
<td>0.86</td>
<td>0.80</td>
<td>0.50</td>
<td>0.60</td>
</tr>
<tr>
<td>Stage 3</td>
<td>1.09</td>
<td>1.14</td>
<td>1.03</td>
<td>0.66</td>
<td>0.91</td>
</tr>
<tr>
<td>Stage 1</td>
<td>0.24</td>
<td>--</td>
<td>--</td>
<td>0.16</td>
<td>0.24</td>
</tr>
<tr>
<td>Gyttja</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stage 2</td>
<td>0.62</td>
<td>--</td>
<td>--</td>
<td>0.36</td>
<td>0.78</td>
</tr>
<tr>
<td>Stage 3</td>
<td>0.82</td>
<td>--</td>
<td>--</td>
<td>0.52</td>
<td>1.05</td>
</tr>
<tr>
<td>Stage 1</td>
<td>0.50</td>
<td>--</td>
<td>--</td>
<td>0.26</td>
<td>0.50</td>
</tr>
<tr>
<td>Total</td>
<td>Stage 2</td>
<td>1.44</td>
<td>--</td>
<td>0.86</td>
<td>1.38</td>
</tr>
<tr>
<td>Stage 3</td>
<td>1.91</td>
<td>--</td>
<td>--</td>
<td>1.18</td>
<td>1.96</td>
</tr>
</tbody>
</table>

The change in the parameters in the Asaoka method during parts of the consolidation process can be studied in Figs. 70 and 71.

Corresponding changes in predicted COURSES OF CONSOLIDATION can be seen in Fig. 72. Further changes would occur if the field observations were made for an even longer period of time.

One-dimensional consolidation analyses have been performed at DG using Terzaghi's method (conventional analysis) and the $c_v$ values from the oedometer tests.
A. FOR PEAT LAYER

![Graph A]

B. FOR CALCAREOUS SOIL LAYER

![Graph B]

Fig. 70. Change in consolidation parameters in Asacka's method during consolidation (Embankment No. 1)
Fig. 71. Change in consolidation parameters in Asaoka's method during consolidation (Embankment No. 2)
Fig. 72. Measured and estimated subsoil settlements of Embankment No. 1 (top figure) and Embankment No. 2.
PREDICTION OF CONSOLIDATION using Young and Ludwig's (1984) approach to account for large strains has also been made at DG.

The governing equation is:

$$\frac{\partial}{\partial t} \left( \frac{k}{\gamma_w} \frac{\partial u}{\partial \zeta} \right) + \frac{1}{1+e} \frac{de}{d\sigma} \frac{Du}{Dt} \bigg|_x = 0$$

where:
- $k$ = coefficient of permeability
- $\gamma_w$ = unit weight of water
- $e$ = void ratio
- $u$ = excess pore water pressure
- $\sigma'$ = effective pressure
- $\zeta$ = convective coordinate
- $\frac{Du}{Dt}\bigg|_x$ = material derivative
- $t$ = time

For application of this method to prediction of settlement and excess pore pressure dissipation, a numerical method is required to solve the equation because of the non-linear nature of the parameters. A piecewise linear iterative calculation procedure is required. In the piecewise linear iterative analysis, the derivation for finite difference consolidation is performed with respect to a convective coordinate system. The excess pore pressure is updated explicitly and proper accounting for surcharge loading and updating of stresses and soil properties is required during the calculation process.

Calculation of the course of consolidation under embankment No. 1 (without drains) has been made with the numerical programme LSCA (Large Strain Consolidation Analysis). This programme was originally created for calculation of one-dimensional consolidation of waste ponds at the Geotechnical Research Centre at McGill University, GRC, (Young and Ludwig, 1984). It was further developed for prediction of settlements of embankments together with DG in conjunction with the joint research between GRC and DG, (Szymanski and Lechowiz, 1987). The soil parameters used in the calculations are the specific gravity of the solids, the relationships between void ratio and effective stress and the void ratio and coefficient of permeability. These parameters were obtained from the laboratory investigations performed at SGI and DG.
The calculated courses of settlement using conventional analysis and the LSCA programme are shown together with the measured settlements in Fig. 73. The calculated initial settlements have been added to the calculated consolidation settlements.

![Diagram showing measured and calculated subsoil settlements of Embankment No. 1.](image)

**Fig. 73.** Measured and calculated subsoil settlements of Embankment No. 1.

The course of settlement predicted by the LSCA programme is fairly close to the observed settlements during the period for observation. However, at the ends of the load stages almost all predicted settlements had occurred, but there still remained large excess pore pressures and the settlements continued at appreciable rates. The settlements predicted by conventional analysis only amounted to about 60% of the measured settlements at the ends of the load stages. None of these methods accounts for time dependency of the compression characteristics.

The course of consolidation of the embankment with vertical drains was predicted with the method originally developed by Barron (1949) and elaborated by Hansbo (1979, 1981). The calculations were made with and without regard of the effects of smear and well resistance. Figs. 74-76. The calculated effects of smear and well resistance were relatively small in this case, but both effects would slow down the consolidation process. There is relatively good agreement between the settlements predicted with the plain method without any smear or well resistance effects.
Fig. 74. Measured and calculated subsoil settlements of Embankment No. 2.

Fig. 75 - 76. Measured and calculated settlement of subsoil with vertical drains in Antoniny site.
However, a comparison between the courses of settlements under the two embankments with and without drains shows that the settlements under the embankment with drains were almost identical and only marginally larger than the settlements under the embankment without drains. The applied load was also slightly larger for the embankment with drains. As in the case of the embankment without drains, almost all predicted settlements had occurred at the end of the load stages, while there were still large remaining excess pore pressures and continuing settlements.

These additional observations show that, instead of being a good prediction of the real behaviour, the prediction of the consolidation with drains is a striking example of the pitfalls of back analyses that may occur unless all aspects of the behaviour are accounted for. Such pitfalls have explicitly been pointed out by Leroueil and Tavenas (1981).

The SETTLEMENTS and the CONSOLIDATION PROCESS under the embankment without drains have also been calculated as one-dimensional consolidation at SGI.

The consolidation process has been calculated with the computer programme CONMULT (CONsolidation of MULTilayers). In this programme, the soil can be divided into a large number of layers and the compression and permeability characteristics of each layer can be described in detail.

The calculations are made in small time-steps using Terzaghi’s equation for one-dimensional consolidation

\[
\frac{\Delta u}{\Delta t} = \frac{M}{g \rho_w} \cdot \frac{\partial}{\partial z} (k \cdot \frac{\partial u}{\partial z})
\]

where
- \(u\) = excess pore pressure
- \(t\) = time
- \(M\) = modulus
- \(\rho_w\) = density of water
- \(z\) = vertical distance to draining surface
- \(k\) = permeability

The compression characteristics of the soil are often time-dependent due to CREEP EFFECTS. This can be taken into account in the calculations by calculating the creep settlements that would have occurred during the time-step if there had not been a hydraulic flow resistance preventing them from developing. To allow for flow resistance these calculated creep settlements are converted to a corresponding pore pressure increase \(\Delta u_{t+\tau}\) by using the compression modulus. The consolidation equation then changes to
The equation is solved using finite differences with small time steps. Continuity between the layers demands that the rate of water flow across the interface between the layers be constant

\[ \frac{\partial u}{\partial t} = \frac{M}{g \cdot \rho_w} \cdot \frac{\partial}{\partial z} \left( k \cdot \frac{\partial u}{\partial z} \right) - \frac{\partial u_{ct}}{\partial t} \]

In each time step, the rate of deformation in each layer is calculated and compared to the reference rate for which the compressibility of the soil has been determined. The pore pressure is then changed according to the creep characteristics. The compression characteristics, the permeability and the applied load are updated for the changes due to the deformation during the time step. The consolidation process during the next time step is then calculated.

The STRESSES are calculated according to the theory of elasticity and the IMMEDIATE PORE PRESSURES resulting from the load increase are calculated using three empirical findings. The first is that, within the "elastic stress range", i.e. before the soil is overstressed by shear stresses or a yield stress such as the preconsolidation pressure is reached, the change in pore pressure under undrained conditions is approximately equal to the change in total octahedral stress. The second finding is that, once the effective vertical stress amounts to the yield stress in a vertical direction, i.e. the preconsolidation pressure, the pore pressure response will be such that the preconsolidation pressure is not exceeded under undrained conditions. The third finding is that this limiting effective vertical stress encountered at full scale loading in the field is equal to the preconsolidation pressure evaluated from oedometer tests in the laboratory, provided that the established procedures for evaluation are used. The immediate pore pressure response at loading (or unloading) is thus calculated as

\[ \Delta u = \Delta \sigma_{oct} \quad \text{when } \sigma' \leq \sigma' \]
\[ \Delta u = \Delta \sigma_v \quad \text{when } \sigma' \leq \sigma' \]

The computer programme CONMULT and the soil model used are described in detail together with their backgrounds by Larsson (1986).
The CONSOLIDATION SETTLEMENTS for the undrained test fill at Antoniny have been calculated with the CONMULT programme ACCOUNTING FOR AS WELL AS DISREGARDING CREEP EFFECTS. The division of the soil into layers follows the oedometer tests. The division and the key consolidation parameters are shown in Fig. 77.

Fig. 77. Parameters used for calculation of consolidation at Antoniny site, Bialosliwie.
The consolidation parameters are as evaluated from the oedometer tests. The initial moduli are usually not taken from oedometer tests, but as it becomes extremely difficult to estimate an empirical modulus when the effective stresses in the ground are close to zero, an unusual degree of reliance has in this case been put on the results from the oedometer tests.

The total "final" settlements disregarding creep settlements but including initial settlements are calculated to be about 0.5 m, 1.3 m and 1.8 m for the three stages. However, no full pore pressure dissipation has occurred in any of the stages, but stages 2 and 3 were applied when there were still relatively high remaining excess pore pressures. The pore pressures dissipation in stage 3 was still in progress when the load stage was ended. The total settlements at the end of stage 1 and 2 were 0.4 m and 1.1 m respectively and the continuing settlements in stage 3 amounted in May 1987 to 1.8 m.

In the calculation of the consolidation process, the upper one and a half metre of peat with vegetation, cracks and root threads has been considered as free draining. This assumption was made first after the piezometers at 2 m depth had shown that the pore pressure dissipation during stage 1 was very rapid and that the piezometers were close to a free draining surface. The assumption of free drainage in the entire layer of 1.5 m is of course exaggerated, but measurements of permeability in the homogeneous amorphous peat had shown that the permeability of the homogeneous peat was ten to one hundred times greater than the permeability of the other layers. The infusion of root threads and cracks would further greatly increase this difference, but to what extent cannot readily be estimated. As there are practical limitations to the variations in permeability that can be put into the CONMULT programme used, the upper layers have been assumed as free draining for simplicity. It should be considered, though, that this is an oversimplification and that the permeability in these layers probably decreases very much with compression as cracks are closed and channels cease to stand open.

The consolidation process has been calculated on the assumption that the initial artesian water pressure in the sand remains constant and that the final pore pressure in each layer will deviate from the hydrostatic pressure in the same way as in the initial stage.

The load reduction due to settlements has been assumed to be 6 kPa per meter of settlement. Here, this corresponds to a load reduction of 10-15% of the total applied load at the end of stages 1 and 2 and at the last observation in stage 3.

The measured and calculated (initial plus consolidation) total settlements are shown in Fig. 78.
The calculated time-settlement curves have been smoothed for the first fifty days after each load application to roughly compensate for the exaggerated assumption of free drainage in the top layers. A comparison with the measured time-settlement curves shows that this compensation was not quite enough for the early stages just after load application. The discrepancy in measured and predicted behaviour in these stages is also partly due to all lateral movements being predicted to occur instantaneously at loading, whereas in reality they are smaller than predicted in the uploading phase but continue for a short period thereafter.

A comparison between calculated settlements with and without creep effects shows that in the relatively short periods of time studied, the creep effects have little influence. The differences increase with time, though, and the trend is that with time the measured time-settlement curve more and more connects to the curve calculated to include creep effects.
The calculated settlements in stage 1 are larger than the measured settlements. In this stage, however, the calculated consolidation is almost entirely based on the rather roughly estimated recompression moduli. Considering the fact that the initial deformations are somewhat overpredicted, the agreement is surprisingly good.

Nevertheless, the discrepancy in this stage affects the comparisons between measured and calculated total settlements in the further stages.

The DISTRIBUTION OF SETTLEMENTS has been measured by special magnetic markers. The course of the settlements with time has been studied only from stage 2 and onwards. Fig. 79 shows the settlements for the two main types of soil; the peaty soil on top and the calcareous soil below. The settlements are shown as further settlements after stage 1.

![Figure 79. Increase of settlements in Stages 2 and 3 under the embankment without drains.](image)

Apart from the discrepancies in the earliest stages already discussed, there is good agreement between the measured and calculated settlements in magnitude as well as distribution. Four magnetic settlement markers are installed under the centre of the embankment and the settlement distribution versus depth at the end of stage 2 is shown in Fig. 80. Problems with buckling of the guiding tubes occurred in stage 3.
The PORE PRESSURE DISSIPATIONS in piezometers located 2 m below original ground surface have been rapid, which indicates a very high permeability in most of the very compressible overlying soil. The excess pore pressure measured at a point which was originally 4.5 m below the ground surface is shown in Fig. 81. This point was located approximately in the middle of the layers with low permeability.

The measured and calculated pore pressures agree quite closely. There is practically no difference in the calculated pore pressures, whether creep is considered or not. The higher rate of dissipation that has been measured in the later stages can, at least partly, be attributed to the fact that the pore pressure tip has been pushed closer to the lower drainage boundary.

At the end of stage 2 and stage 3, new samples were taken close to the centre of the "undrained" embankment and oedometer tests were performed. The preconsolidation pressures measured in these tests are shown in Fig. 82.
Fig. 81. Measured and calculated excess pore pressure at the "middle" of the low permeable soil under the embankment without drains.

The PRECONSOLIDATION PRESSURES measured in oedometer tests correspond very well to those calculated with CONMULT including creep effects. The calculated quasi preconsolidation pressures are the pressures on the original oedometer curves that correspond to the calculated deformations.

Except at the very boundaries, the effective stresses that had acted in the profile were much lower than the "final" stresses would have been if there had been no excess pore pressures. In the centre part of the layer with low permeability, the effective stresses had not even exceeded the original preconsolidation pressures at the end of stage 2. In those parts of the profile where the preconsolidation pressure had been exceeded, quasi preconsolidation pressures had developed that were considerably higher than the maximum effective vertical stresses. On the other hand, there was no measured increase in preconsolidation pressure in those parts of the profile where the vertical stresses were still lower than the original preconsolidation pressures. At the end of stage 3, the relations between maximum effective stresses and developed preconsolidation pressure was very similar. The original preconsolidation pressure had been exceeded in all parts of the profile, but the degree of consolidation in the centre parts of the profile with low permeability is low as large excess pore pressures remained.

According to the calculations, there was no increase in preconsolidation pressure below 4 m depth in stage 1. The calculated increase in quasi preconsolidation pressure above this level in stage 1 is also shown in Fig. 82.
Fig. 82. Measured and calculated stresses and quasi preconsolidation pressures at the centre of the embankment without drains.
The calculations with CONMULT have thus qualitatively accounted for the observed behaviour of the soil under the embankment. The assumption about the drainage situation used in the calculations could hardly have been made beforehand, but is a result of the observations during the early loading stages. The effect of creep was comparatively small during the relatively short periods of observation in terms of total settlements and pore pressures. The effect of creep on the increase of quasi preconsolidation pressure and undrained shear strength was pronounced in some parts of the profile.

The initial "elastic" deformations predicted with empirical relations were of the right order of size but somewhat too large.

The limited EFFECT OF CREEP is not general for this type of soil. The effect of creep is most pronounced at relatively small load increases and settlements and becomes more evident with time. The creep effects can only be a part of the total settlements. In this particular case where the average settlements amounted to about 25 per cent of the thickness of the compressible layer and occurred within 4 years, the effects during this period were limited. The settlement process, however, was not concluded and if allowed to continue the differences between taking creep into account or not would have become larger. 5 years after the final load application the predicted settlements would have amounted to 2.2 metres. After 10 years they would have been 2.5 metres and after 50 years 2.7 metres. The final settlements calculated without creep effects would have been reached within 25 years and amounted to 2.0 metres.

The discrepancy between the latter figure and the 1.8 metres estimated by more approximate calculations using the same soil parameters is due to the calculation procedure. When the course of consolidation for the various parts of the soil profile is followed, the effect of load reduction due to settlements affects the different layers in different ways. The end result thus differs from a crude calculation where the load reduction is assumed to affect the whole profile by the same amount.

TWO-DIMENSIONAL FINITE ELEMENT CALCULATIONS for the case of plane-strain consolidation have also been made at DG. The calculations have been made for the embankment without drains with the computer programme COPRESS developed at DG by Kaminski (1986). The calculations are made on the following assumptions about the compressibility of the soil:
• piece-wise linear stress-strain relationship for the soil skeleton

\[ \sigma'_{ij} = G(W_{i,j} + W_{j,i}) + (K - \frac{2}{3} G) W_{kk} \delta_{ij} \]

where \( \sigma'_{ij} \) = the effective stress tensor
\( W_{i,j} \) = component of the displacement gradient
\( W_{j} \) = the displacement vector
\( \delta_{ij} \) = Kronecker's delta
\( K = \frac{E}{3(1-2v)} \) and \( G = \frac{E}{2(1+v)} \)
\( E \) = deformation modulus
\( v \) = Poisson's ratio

The following equations are used for calculation of the consolidation process:

• Darcy's law for the pore water flow

\[ q_{j} = k \cdot u_{j} \]

where \( q_{j} \) = the component of the specific discharge vector of pore water
\( u_{j} \) = gradient of pore water pressure
\( k \) = coefficient of permeability

• Terzaghi's formula

\[ \sigma_{ij} = \sigma'_{ij} + u \delta_{ij} \]

where \( \sigma_{ij} \) = total stress tensor

• the de Josselin de Jong storage equation

\[ \frac{\delta \varepsilon}{\delta t} = n \beta \frac{\delta u}{\delta t} - q_{i,j} \]

where \( \varepsilon \) = volume strain
\( n \) = porosity
\( \beta \) = compressibility of the pore water
The calculation procedure in the COPRESS programme is schematically shown in Fig. 83.

Calculations have been made with values of modulus of elasticity, Poisson's ratio and permeability selected with guidance from the laboratory tests. Calculated excess pore pressures and deformation patterns are shown in Figs. 84 and 85.

The calculated DISTRIBUTIONS OF EXCESS PORE PRESSURES AND DEFORMATIONS resemble the measured values, but the calculated horizontal movements and the calculated excess pore pressures under the outer parts of the embankment become too large. This can partly be attributed to the assumption of a constant Poisson's ratio used in the calculations.

Another calculation of the deformations under this embankment has been made at DG with the FINITE ELEMENT programme COVEPP. This programme was also developed at DG by Kaminski (1983). The programme uses an ELASTO-PLASTIC SOIL MODEL AND A YIELD ENVELOPE to distinguish the different phases of soil behaviour. Two dimensional plane-strain conditions are assumed. An isotropic yield surface has been assumed and the parameters have been selected with guidance from the laboratory tests.

Fig. 86 shows the finite element mesh and the calculated deformation pattern together with the measured deformations. Also in this case, the calculated horizontal deformations became too large. This may partly be attributed to the type of yield surface that is used in the programme. Yield surfaces for natural soils are highly anisotropic and usually also change their shape during this type of consolidation process.
Fig. 83. Flowchart of the calculations in COPRESS.
Fig. 84. Measured and calculated excess pore pressures under Embankment No. 1, after the load applications.
Fig. 85. Measured and calculated displacements under Embankment No. 1, after the load applications.
Fig. 86. Measured and calculated displacements under Test Embankment No. 1.
7.2 Stability analyses for embankments on soft soils

7.2.1 General

Stability analyses for embankments on soft soils can be carried out with different degrees of sophistication. For rough calculations in order to make sure that the stability is sufficient, an undrained total stress analysis using circular slip surfaces (Fellenius 1926) is usually used. The total stress analysis almost always gives the lowest factors of safety in soft soils, but in stiff and highly overconsolidated soils an additional effective stress analysis may be required.

The circular slip surface usually corresponds fairly well to the critical slip surface. The assumption of a circular slip surface also makes the calculations fairly simple. In soil profiles containing weaker layers, the critical surface may significantly deviate from the circular shape and this has to be accounted for.

The undrained analysis is often based on undrained shear strengths obtained from corrected vane shear tests. The estimation of undrained shear strengths can be further elaborated by using an ADP-analysis, whereby the slip surface is divided into zones of active shear, direct simple shear and passive shear. The values of the shear strengths in the different zones are then obtained from corresponding tests in the laboratory, Fig. 87.

![Diagram of shear zones](image)

Fig. 87. Relevance of laboratory shear tests to shear strength in the field.
The ADP analysis was originally developed by Ladd (1969) and has been extensively used since then, especially at Massachusetts Institute of Technology and at the Norwegian Geotechnical Institute. It becomes especially useful in stage-construction of embankments where the increase in shear strength, which is different in different zones, is taken into account.

In rough estimates of stability, the shear strength in the fill material is often neglected. In more elaborate analysis, it has to be taken into account. It then has to be considered that the strength properties in the fill, which usually consists of granular material may change due to the deformations in the soil below. It is very doubtful whether any strength significantly higher than the strength at critical density (i.e. no dilatancy effects) can be accounted for in fills on soft soils, even if the fill material has been compacted to some degree. This is especially valid when the stability is low and the initial deformations become large.

In stage construction, the increase in shear strength due to consolidation can be utilized. A procedure for taking this shear strength increase into account, which was originally outlined by Aas (1976), was recommended by SGI in 1984, (Larsson et al 1984).

In this procedure, the assumed shear surface is divided into three parts. In the part of the shear surface under the embankment where the inclination is steep, i.e. the active zone, the increase of the shear strength is assumed to correspond to the increase in shear strength due to consolidation in plane-strain or triaxial compression. This increase in shear strength is actually larger than that normally evaluated from the vane shear tests. For a large number of Scandinavian soft soils and also for many other soft soils, the increase in active compression shear strength amounts to about one third of the increase in preconsolidation pressure. Similar values were found for the soils at the Antoniny site.

In the part of the assumed shear surface under the embankment which is horizontal or almost level, the increase in shear strength is assumed to correspond to the increase in shear strength due to consolidation at direct simple shear. The shear strength at direct simple shear is normally close to the shear strength evaluated from vane shear tests.

The shear strength values measured by vane shear tests are mainly influenced by the horizontal stresses in the ground and increases in shear strength under narrow embankments are therefore not measured fully by the vane test, (Law 1985).
According to the SGI recommendations, the increase in shear strength is assumed to be confined to the extent of the embankment and the shear strength outside this limit is assumed to be unchanged. The embankment area is here normally assumed to end at the middle of the slope, Fig. 88. The comprehensive shear strength testing in the later stages at the Antoniny site has confirmed the general validity of this assumption.

As an alternative to the ADP analyses the corrected shear strength values from field vane tests can be used, but the full shear strength increase is thereby often not utilized.

Fig. 88. Simplified estimation of increase in shear strength due to consolidation below an embankment.

More elaborate stability analysis normally use some method where the soil above the assumed slip surface is divided into slices. All forces acting on the individual slices; external forces, interslice forces and normal and shear forces on the slip surface, are taken into account, as well as the interaction between the slices. The simplest and internationally probably most commonly used method is the simplified Bishop method, (Bishop 1955). A large number of other slice methods have been proposed, e.g. Morgenstern and Price (1965) and Janbu (1973). Normally, the safety factors calculated with the different methods differ only marginally. The most commonly used method in Scandinavia is Janbu's "General Procedure of Slices", which also enables a closer study of the interaction between forces and stresses in the soil mass (e.g. Svanå 1981, Larsson 1983). The slip surfaces can be of arbitrary shape and can thus be assumed to follow weaker layers in the soil. The calculations with the slice methods can be performed as undrained, partly drained or drained analyses.
For calculation of stability of embankments on soft soils, the normal procedure is that the fill material is assumed to be drained and the soft soil undrained. Partial drainage during load application may occur in overconsolidated soils, but practically stops when the effective stresses reach the preconsolidation pressures, (Tavenas 1979). The effect of this partial drainage on the shear strength is very limited, except for heavily overconsolidated soils.

As also the shear strength in the undrained cases is due to effective stresses, it may be advocated that it would be more correct to predict the pore pressures and use an effective stress analysis for all parts of the slip surface. However, the undrained shear strength is simply the net result of the effective strength parameters, the original effective stress in the soil, the stress changes and the pore pressures resulting from these at failure for a given undrained loading case. At low factors of safety, there is therefore no significant difference in the calculated safety factors for embankments on soft soils calculated with total stress analysis or effective stress analysis taking pore pressure responses into account. This assumes that both the undrained shear strengths are relevant and that the pore pressures have been correctly predicted.

At higher safety factors, differences occur as the total stress analysis takes the pore pressures at failure into account, while the effective stress analysis only considers the pore pressures developed at the actual stress level. The calculated safety factors using effective stress analysis thereby generally become higher in soft soils and their physical significance is rather a measure of the current stress state in the soil. It is more difficult to interpret from these "safety factors" how close the embankment is to failure. The safety factors calculated assuming drained conditions in the fill and using undrained total stress analyses in the soft soils, on the other hand, are direct comparisons between the shear stresses that can be mobilized at failure and the actual shear stresses, (Svanö 1981, Larsson 1983, 1984, Ladd 1985).
7.2.2 Predictions of increase in shear strength due to consolidation

To predict the increase in shear strength due to consolidation, the stress distributions and courses of consolidation have to be accurately predicted as the shear strength is a function of the preconsolidation pressure and the stress level.

The most commonly used relations for prediction of shear strength assume that the shear strength in normally consolidated soil is a direct function of the preconsolidation pressure

$$\tau_{fu} = c \sigma_p$$

For MINERAL SOILS, it has been found that $c$ is about 0.23±0.04 (Larsson 1980). In laboratory tests it has been found that the undrained shear strength at unloading decreases with increasing overconsolidation ratio. Ladd (1977) and Jamiolkowski et al (1985) suggested the relation

$$\tau_{fu} = c' \sigma' V (OCR)^m$$

Also in this expression $c$ is 0.23±0.04 and the value of $m$ found in laboratory tests is about 0.8.

The $c$ values of 0.23±0.04 are valid for the average of the strengths at direct simple shear and active and passive shear. This average shear strength is close to both the shear strength at direct simple shear and the corrected shear strength from field vane tests. These $c$ values have also been found in back-calculation of a number of full scale failures of constructions on natural soft clays.

The corresponding constant for the shear strength at active shear in mineral soils is about 0.33, while the constant for passive shear is much lower and highly variable with the plasticity of the soil. Data for these relations can be found in Larsson et al (1984) and Jamiolkowski et al (1985).

In field vane tests, the value of $m$ has been found to be close to 1, which for this type of test makes the relation

$$\tau_{fu} = c \sigma'_p$$

valid also in overconsolidated clays.

These relations have been obtained for mineral soils and the investigations of the soils at Antoniny have shown that they have to be modified for ORGANIC SOILS and possibly also for other soils with very low initial preconsolidation pressures.
The predictions of increase in undrained shear strength can be made from the increase in effective stress and preconsolidation pressure due to calculated increases in total vertical stress and dissipation of excess pore pressures. The pore pressure dissipation can be checked by pore pressure measurements. The values thus calculated do not account for the increase in shear strength due to creep effects. The creep effects create quasi preconsolidation pressures that are higher than the maximum effective vertical stresses and thereby also corresponding increases in undrained shear strength. (Bjerrum 1972, 1973, Larsson 1986). These effects can be calculated but verification usually requires that samples be taken and tested in the laboratory.

Calculated and measured quasi preconsolidation pressures are presented in Chapter 7.1.2.

The undrained shear strengths were measured by corrected field vane tests at various stages during construction of the embankments. Measurements were performed under the centre of the embankment, under the middle of the slopes and at the toes of the slopes.

The effective stresses calculated for the same times as the field vane tests were performed have been used to predict the field vane strengths that would be measured. The predicted and measured values are shown in Figs. 89 and 90.

The predictions using relations normally used in mineral soils yield higher strengths than predictions with the relations found in the present investigation when the initial preconsolidation pressures are exceeded. The measured values are of the same order as the predicted values. The shear strength under the embankment would have been somewhat overpredicted in the later stages if the shear strength is assumed to be related to the preconsolidation pressure only. Considering the normal scatter in the vane results, no further conclusions can be drawn from this comparison.
Fig. 89. Corrected vane shear strength and predicted shear strength values under Embankment No. 1, before Stage 2 (April, 1984).

Legend:
- corrected field vane test
- Jamiolkowski et al. (1985)
- SGI-DG (Bergdahl et al., 1987)

Fig. 90. Corrected vane shear strength and predicted shear strength values under Embankment No. 1, before Stage 3 (May 1985).
7.2.3 Predictions of stability for the embankments at Antoniny

The stability of the embankments in the various stages has been calculated taking into account the initial shear strength and the increase in shear strength due to consolidation. The sand in the embankment has been assumed to have a friction angle of 30°. This relatively low value was chosen because of the low density and the expected large deformations. At large deformations, only the frictional forces corresponding to the friction angle at critical density can be mobilized.

Rough estimates of the stability in Stage 1 using undrained shear strength and circular slip surfaces and ignoring the shear strength in the fill material yielded safety factors between 2 and 3. Because the geometry with wide embankments placed on soft soil with limited thickness and with dense sand underneath, the lowest safety factors were obtained for local slip surfaces in the outer parts of the embankments.

The stability has then been calculated at DG with the simplified Bishop method using a computer programme designated SSA-1. The soil was divided into three zones with different magnitudes of shear strength increase; A - below the part of the embankment with full height, B - below the slopes of the embankments and C - outside the embankments, Fig 91. The soils in these zones were divided into layers with characteristic undrained shear strengths and the shear strength in the fill material was also taken into account.

The change in geometry, bulk density and shear strength during the consolidation was taken into account at calculation of the stabilities in stages 2 and 3.

Calculations were made using the corrected shear strengths obtained from the field vane shear tests and also with the predicted shear strengths using the relations between undrained shear strengths and stress levels found in the present investigation. (Chapter 6.3). Here the calculated stresses and the measured pore pressures at the end of each stage were used.
TEST EMBANKMENT NO. 1

Fig. 91. Undrained total stress analysis of embankment stability.

142
For embankment No. 1 (without drains) the stability calculations with corrected vane shear strengths yielded the following safety factors:

Stage 1  F = 2.30
Stage 2  F = 1.49
Stage 3  F = 1.35

In the stability analyses using strengths estimated from the stress levels the safety factors became somewhat lower:

Stage 1  F = 1.96
Stage 2  F = 1.18
Stage 3  F = 1.22

The location of the critical circular slip surfaces is also shown in Fig. 91. In stage 1 the critical surface is a local slip surface at the outer parts of the embankment. In stage 2 it comprises most of the embankment, but mostly due to the limited thickness of the soft layers not quite all of the embankment crest. In stage 3 the embankment crest has become narrower and the critical slip surface passes through the back of the crest.

The results of the calculations for embankment No. 2 were very similar.

Stability analyses have also been made at SGI using Janbu's "General Procedure of Slices". The calculations have been made for stages 2 and 3. The soil has been divided into four zones with consideration to mode of shear and shear strength increase. Circular and non-circular slip surfaces have been assumed. In accordance with the procedure recommended by SGI, the steepest part of the shear surfaces under the embankment has been assumed to be a zone of active shear. The almost horizontal parts of the shear surfaces have been assumed to be zones of direct simple shear and the steeper parts of the shear surface where it goes up outside the embankment to be zones of passive shear. The increase in shear strength has been assumed to be confined to the soil below the embankment within the centres of the slopes. Quasi preconsolidation effects on the increase in shear strength have been accounted for. The calculated factors of safety against undrained failure in stages 2 and 3 were both about 1.2.

The shape of the critical shear surface deviated from the circular in that it was steeper down to the weakest layer located about 2 m above the sand layer. It then followed this layer to below the centre of the slope, where it started to go upwards.
The formation of a distinctly weaker layer in the later stages at about 2m above the sand layer originated from the lack of consolidation there.

The stability has also been calculated by DG using effective stress analyses. The effective strength parameters for both types of soil had been measured to be $c' = 2$ kPa and $\phi' = 30^\circ$.

The initial pore pressure pattern had been measured with a large number of piezometers and the excess pore pressures due to the loading were measured in selected points under and outside the embankments. The pore pressure distributions at various times were thus estimated and the safety factors in terms of effective stresses were calculated.

The safety factors were calculated just after full load application for stages 2 and 3. In stage 1 the corresponding measurements of excess pore pressures were lacking. For comparison, the safety factors at the end of stages 1 and 2 and one year after load application in stage 3 were calculated. The calculations were made with circular slip surfaces and Bishop's simplified method of slices. The same slip surfaces as in the total stress analyses were used. The geometries, pore pressure distributions and locations of the piezometers are shown in Figs. 92 and 93.

The calculated factors of safety in stages 2 and 3 just after full load application were 1.07 and 1.12 respectively. The total stress analyses had given the corresponding factors of 1.18 and 1.22. In general, this confirms that there is no great difference between safety factors calculated with undrained total stress analyses or effective stress analyses considering excess pore pressures in such cases and if the safety factors are low. On the other hand, the safety factors calculated with undrained total stress analyses should normally be lowest. The measured excess pore pressures, however, are probably somewhat too high due to the pushing effects at loading. The artesian water pressure and very low effective stresses outside the embankments also have some influence on the relation between the calculated safety factors.

This can also be observed in the calculation of the safety factor in terms of effective stresses at the end of stage 1, which was 2.3. This is practically identical with the safety factor in the undrained total stress analyses after load application. In this load step the load was low and the critical shear surfaces located to the outer parts of the embankments, so that the effect of the artesian water pressure and the stress conditions outside the embankments had a large influence on the results.
The calculations for the end of stage 2 and one year after load application in stage 3 showed that the safety factor in terms of effective stresses rapidly increases with the dissipation of excess pore pressure.

**TEST EMBANKMENT NO. 1**

**STAGE 2**

- **F = 1.07**
- Equal pore pressure lines
- **STAGE 3**
- **F = 1.12**
- Equal pore pressure lines

*Fig. 92. Effective stress analysis of embankment stability with in situ pore pressures at the beginning of Stages 2 and 3.*
Fig. 93. Effective stress analysis with in situ pore pressures at the end of construction stages 1, 2 and 3.
8. SUMMARY AND CONCLUSIONS

Two test embankments have been built on top of 8m of soft organic and calcareous soil. The embankments have been constructed in stages and the increase in shear strength due to consolidation in the different stages has been utilized in the construction of the subsequent stages. Vertical prefabricated drains were installed under one of the embankments.

A comprehensive programme of field and laboratory tests was carried out before the construction of the embankments. A large amount of monitoring equipment was also installed in the ground under and outside the embankments. The behaviour of the embankments in terms of settlements, horizontal displacements and pore pressures has been followed and the changes in soil properties have been measured. The behaviour and the changes in properties have been compared to predictions using various methods of prediction. Special investigations have been carried out concerning the increase in shear strength at consolidation and the durability of prefabricated drains in harsh environmental conditions.

THE MONITORING EQUIPMENTS were selected with consideration to the soft soil, the large expected deformations and the long period of observation.

The deformations were very large. The settlements amount to almost 2 metres which is half of the thickness of the fills and corresponds to 25 per cent of the thickness of the compressible layer. The horizontal displacements were up to 0.55 metres. This amount of movement proved to be the limit for the equipments with vertical tubes, which were buckled or else deformed in such a way that the measuring devices could not be inserted. This was the case for one of the inclinometer tubes and the magnetic settlement screws. The piezometers used were of the BAT type with filter tips connected to a pipe going up to the ground. This type of equipment is excellent for long-term observations as the electronic parts are removed between the measurements and can be checked. The pipes to the ground, however, entail a risk for pushing of the piezometers further into the soil due to settlements in the layers above. In spite of precautions, there was considerable pushing of the piezometers under the embankments at Antoniny. It can thus be concluded that:

• All equipments with vertical tubes under embankments have limitations as to their working range. Any studies of further movement would require these equipments to be duplicated with new equipments installed when the working range for the first equipments was approached.

• The selection of piezometers is often a compromise between the desire to measure rapid processes at loading and long term stability and accuracy. To measure a rapid process the piezometers should be free from
rigid connections to the ground and to ensure long term stability and accuracy they should be provided with some means for check of the calibration and preferably the measuring device should be retractable. There is also an economical aspect as filtertips that can be measured by the same retractable instrument become relatively cheap when a large number of piezometers are installed. However, when piezometers with pipes to the ground are installed the risk for pushing must be observed and eventual displacements must be measured and accounted for.

Apart from these aspects all the monitoring equipments functioned very well.

THE SITE INVESTIGATIONS showed the necessity of careful documentation not only of the layering of the soil and its mechanical properties, but also the ground water conditions and in this case also the environmental conditions.

The ground water pressure in the sand layer below the compressible layers proved to be artesian, which resulted in an upward pore pressure gradient and extremely low effective stresses in the soil profile. The conditions also varied seasonally. The environment with organic and calcareous soil also proved to be very aggressive to some types of filter material. Different samplers were used and it was observed that, although the peat was highly decomposed, a special peat sampler had to be used to obtain "undisturbed" samples. The Swedish standard piston sampler compressed the peat so much that it clearly showed in the oedometer tests in the laboratory. The only difference that could be observed between samples taken with the Borro 60 mm diam. piston sampler and the peat sampler was a small difference in natural water content. The field vane tests yielded high strength values. After correction with respect to the liquid limit according to SGI, the shear strengths reduced considerably and became similar to the shear strengths obtained in laboratory tests.

The shear strength values obtained in the field vane tests proved sensitive to the testing rate. In peat, the results also proved sensitive to the size of the vane. The cone penetration tests showed very low point resistances that were highly affected by the measuring accuracy. Pore pressure soundings were made, but the measured excess pore pressures were very low or negative.

From the results the following conclusions can be drawn:

- It is very important to measure the ground water conditions and their variation in detail.
- The chemical environment may be important for installations in the ground.
• Sampling in peat requires special samplers even if the peat is highly decomposed.

• Field vane tests in organic and calcareous soils have to be corrected. The corrections recommended by SGI yield reasonable results also in these types of soil.

• It is important that the standard procedure is followed in field vane tests.

• The relevance of field vane tests in peat is questionable as the results are sensitive to the size of the vane.

• Cone penetration tests in peat do not give any detailed information when standard equipments designed for stiffer soils are used.

• Pore pressure soundings in overconsolidated soft soils do not yield any information, except possibly that the soil is overconsolidated.

THE LABORATORY TESTS showed that some but not all methods and equipments used for soft mineral soils could be used for determination of the properties in the organic and calcareous soils.

In the routine tests it was found that the organic content could not be estimated with any accuracy by loss on ignition, due to the high content of carbonates. The relatively simple colorimetric method (Schollenberger 1945, Larsson et al 1985) or some more advanced method should be used for this type of soil.

The compression characteristics of the soil were determined in incremental oedometer tests and oedometer tests with constant rate of strain. The results in terms of compressibility were equal for the two types of tests. Tests on samples taken with different samplers indicated that in the peat the samples taken with the special peat sampler and with Borro 60 mm diam. sampler were less disturbed than samples taken with the Swedish standard piston sampler. Tests on specimens with different sizes also indicated that the upper peat layer was not completely saturated.

The shear strengths were measured in triaxial tests and direct simple shear tests. The averages of the shear strengths measured in active and passive (compression and extension) triaxial tests and direct simple shear tests were of the same order as the corrected strengths obtained in field vane tests.

The effective strength parameters in undrained tests could be expressed as $c' = 2 \text{ kPa}$ and $\phi' = 30^\circ$ for the stress range of interest. Similar parameters were obtained in drained tests corrected for dilatancy effects to
correspond to the case of no volume change. The effective stresses in the ground and the preconsolidation pressures were very low and below the working range of some equipments. Some tests were therefore run at elevated stresses and the test results were normalized towards the preconsolidation pressure according to the SHANSEP-procedure (Ladd and Foott 1974). In the following investigation on the increase in shear strength due to consolidation, however, it was found that the increase in shear strength was not linear with the preconsolidation pressure. A more complex function had to be used, taking the current stress level as well as the initial preconsolidation pressure into account. This was found in all types of tests.

The shape of the yield envelope was investigated and was found to be highly anisotropic. The isotropic Cam-clay model poorly fitted the real shape and an anisotropic model as suggested by Tavenas and Leroueil (1977) or Larsson and Sällfors (1981) has to be used.

In a special testing programme, the discharge capacity of prefabricated drains which had been subjected to the environmental conditions in the ground for different periods of time was investigated. It was found that paper filters completely deteriorated within 250 days. Some discharge capacity remained as the channels in the plastic core had not been filled with soil, but also this rapidly decreased with time and increasing horizontal stresses. The discharge capacity in drains with plastic filters also decreased with time and increasing stresses, but by a much smaller degree.

The following conclusions can be drawn from the laboratory tests:

- Most of the testing methods and equipments used for soft mineral clays can be used also for organic and calcareous soils.

- Loss on ignition is not a satisfactory method for determination of organic content in soils containing carbonates.

- Peats are often not completely saturated and the parameters measured in the laboratory to estimate the time for consolidation may not be relevant unless the degree of saturation can be accurately measured.

- The procedure for estimating undrained shear strength properties by normalization towards the preconsolidation pressure alone cannot be used in organic and calcareous soils with very low initial preconsolidation pressures.

- In these soils, a normalized effective stress level has to be used, where both the initial preconsolidation pressure and the current preconsolidation pressure are taken into account.
• The shape of the yield surface in natural soils is a function of the consolidation stresses and is normally highly anisotropic.

• Paper filters can deteriorate rather quickly in harsh environmental conditions and should be avoided in organic and calcareous soils.

THE OBSERVATIONS OF THE TEST EMBANKMENTS showed that large settlements as well as large horizontal displacements occurred. The behaviour of the two embankments was almost identical, except for the first stage, where the horizontal deformations were smaller and the vertical compressions somewhat larger and faster under the embankment with vertical drains. The observations support the findings from the special investigation on the durability of prefabricated drains that paper filters quickly deteriorate in this type of environment. The horizontal deformations were not immediate, but continued for some time after full load application, whereupon they practically stopped. The vertical settlements were large and continued at the end of all three stages.

The measured pore pressure responses in the ground were often larger than the total vertical stress increase. This can be attributed to dynamic excess pore pressures as the piezometers were pushed further into the soil. The pore pressure dissipation in the middle layers was slow and in spite of the fact that most or all of the settlements predicted with conventional analyses had occurred, there were large remaining excess pore pressures at the end of all load stages. The measured pore pressures in the soft soil were clearly affected by variations in the water pressures in the sand layer below and the position of the free ground water level in the upper peat layer. The increase in shear strength measured by field vane tests confirmed the laboratory tests in that the shear strength values increased relatively slower than the increase in effective vertical stress.

The following conclusions can be drawn from the field observations:

• The deterioration of the paper filters and the subsequent reduction in effect of the drains, as well as the conclusion that this type of filter should rather be avoided in harsh environmental conditions, was confirmed.

• This finding does not reflect on the usefulness of vertical drains which has been repeatedly proven in other projects and also in organic soils.

• Horizontal deformations in embankment construction are not quite immediate but practically stop after some time.
Excess pore pressures of the same magnitude as the total vertical stress increase are developed at load application when the soil is in a normally consolidated state.

Large excess pore pressures remain when all settlements predicted with a conventional analysis have occurred.

Vertical deformations continue at an appreciable rate also when all settlements predicted with a conventional analysis have occurred.

At observations of pore pressure dissipations also the variation in boundary conditions at the ends of the drainage paths have to be observed and taken into account.

The pattern of shear strength increase due to consolidation found in laboratory tests was qualitatively confirmed by the field vane tests.

Predictions of deformations were carried out by a number of methods. In most calculations the predicted deformations were divided into initial "elastic" deformations and one-dimensional consolidation settlements.

The initial settlements and horizontal displacements were somewhat lower than the predicted movements. On the other hand, the horizontal movements continued for a short time. The final settlements corresponding to the horizontal displacements therefore became close to the predicted values, even if they did not occur quite instantaneously at the load application.

Various empirical methods were used to estimate the "final" consolidation settlements in the peat layer. The spread in the results was large. The final settlements calculated by conventional analyses and using the results from oedometer tests were close to those later predicted by Asaoka's method. This method uses field observations during the consolidation process to estimate the further course of consolidation and the "end result". These predictions of "final" deformation appeared reasonable, but for the fact that high excess pore pressures remained and the settlement rates in the field were still considerable when almost all predicted settlements had occurred. It was also observed that the parameters in Asaoka's method change during the consolidation process and a long observation period was required to obtain a "reasonable" prediction. Asaoka's method is based on Terzaghi's consolidation theory where all parameters are constant and independent of time and deformation.

Conventional analyses of the course of consolidation using the Terzaghi equation also showed that the course of consolidation predicted in that way deviated widely from the field behaviour.
Two types of analysis were performed, where the variation of consolidation parameters, load and geometry during the consolidation process was taken into account. Both types of analysis predicted the observed time-settlement processes fairly well.

A third type of analysis was also carried out, where all the above mentioned factors and also the effect of time on the compressibility of the soil was considered. The predicted settlements thereby became slightly larger and the excess pore pressures marginally larger within the period for observation. All observations of the soil behaviour in the field such as settlements with time, settlement distribution with depth, pore pressure generation and dissipation and even the development of quasi preconsolidation effects due to creep processes could be accounted for.

The differences in the predictions when taking creep effects into account or not were relatively small within the time period for observation. They increased with time, however, and would have become of importance for the long-term behaviour of permanent embankments. The division of the deformations into initial shear deformations and one-dimensional consolidation is artificial. A correct method of prediction should account for the continuous two-dimensional (or three-dimensional) shear and consolidation process.

Two types of finite element analyses for the case of plane-strain have been carried out. In one calculation, an elastic soil model was used and in the other calculation, the soil was assumed to be elasto-plastic with an isotropic yield surface. For both types of calculation it was found that the soil models used were too simple to account for all aspects of the soil behaviour.

From the predictions and the observed soil behaviour can be concluded that:

- For the embankments which were wide in relation to the thickness of the compressible layers the behaviour could be predicted by dividing the deformations into "elastic" initial deformations and following one-dimensional consolidation.

- The "elastic" shear deformations could be calculated with reasonable accuracy using empirical correlations between undrained shear strength, plasticity and calculated factor of safety against undrained shear failure.

- The course of consolidation can be estimated only if the variability of the consolidation parameters, the load and the geometry during the consolidation process is accounted for. A "conventional" analysis does not give satisfactory predictions.
The effect of creep processes cannot be ignored, especially not in the long-term perspective.

The method of predicting "final" settlements from field observations during the consolidation process suggested by Asaoka requires a long period of observation to yield "reasonable results".

Finite element calculations taking two-dimensional (or three-dimensional) deformations and water flow into account are desirable. They require very sophisticated soil models, however, to give better results than the combination of initial shear deformations and one-dimensional consolidation.

Predictions of stability were carried out using total stress analysis as well as effective stress analysis. The calculated factors of safety in the total stress analysis differed somewhat, depending on whether corrected vane shear strength was used or an ADP-analysis using results from triaxial tests and direct simple shear tests was made. The safety factors calculated with corrected vane shear strengths were about 20 per cent higher. This can be attributed to the uncertainty of the relevance of vane shear tests in peat and how adequate the correction factors are for the organic calcareous soil.

Calculations were made with the simplified Bishop method with circular slip surfaces and with Janbu's "General procedure of slices" with slip surfaces of arbitrary shape. In the latter case, the strength increase due to creep was included. This effect, like the effect of non-circular slip surfaces, was small and the two effects counteracted each other. The results of the two types of calculations thus became equal. Calculations were also made in terms of effective stresses using the measured pore pressures after load application. The safety factors calculated in this way were compatible with those calculated with total stress ADP-analyses.

No failure occurred, but the initial deformations at loading in the second and third stage indicated that the embankments were close to failure. The calculated safety factor was then about 1.2. The methods used for calculation of stress increase due to consolidation and for calculation of stability thus seem to have been fairly relevant.

Calculations of stability at various stages after the load applications showed that the safety factors in terms of effective stresses rapidly increased with the dissipation of excess pore pressures.
The following conclusions may be drawn from the results of the calculations:

• The suggested method for prediction of shear strength increase under embankments, coupled with one of the calculation methods with slices and using an undrained ADP-analysis, yields reasonable results.

• The safety factors in terms of effective stresses rapidly increase with dissipation of excess pore pressure. This is probably the main reason why the horizontal deformations practically stopped after a short time.


157


<table>
<thead>
<tr>
<th>No.</th>
<th>Rapport/Report</th>
<th>År</th>
</tr>
</thead>
<tbody>
<tr>
<td>15.</td>
<td>Kartering och klassificering av lerområdens stabilitetsförutsättningar. L. Viberg</td>
<td>1982</td>
</tr>
<tr>
<td>17.</td>
<td>Symposium on Slopes on Soft Clays.</td>
<td>1983</td>
</tr>
<tr>
<td>20.</td>
<td>Porttrycksvariationer i leror i Göteborgsregionen.</td>
<td>1983</td>
</tr>
<tr>
<td>23.</td>
<td>Geobildtolkning av grova moräner. L. Viberg</td>
<td>1984</td>
</tr>
<tr>
<td>25.</td>
<td>Geoteknisk terrängklassificering för fysisk planering. L. Viberg</td>
<td>1984</td>
</tr>
<tr>
<td>26.</td>
<td>Large diameter bored piles in non-cohesive soils. Determination of the bearing capacity and settlement from results of static penetration tests (CPT) and standard penetration test (SPT) K. Gwizdala</td>
<td>1984</td>
</tr>
<tr>
<td>27.</td>
<td>Bestämning av organisk halt, karbonathalt och sulfidhalt i jord. R. Larsson, G. Nilson, J. Rogbeck</td>
<td>1985</td>
</tr>
<tr>
<td>No</td>
<td>Title</td>
<td>Year</td>
</tr>
<tr>
<td>----</td>
<td>----------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>27E</td>
<td>Determination of organic content, carbonate content and sulphur content in soil</td>
<td>1987</td>
</tr>
<tr>
<td></td>
<td>R. Larsson, G. Nilson, J. Rogbeck</td>
<td></td>
</tr>
<tr>
<td>28</td>
<td>Deponering av avfall från Kol- och torveldning.</td>
<td>1986</td>
</tr>
<tr>
<td></td>
<td>T. Lundgren, P. Elander</td>
<td></td>
</tr>
<tr>
<td>28E</td>
<td>Environmental control in disposal and utilization of combustion residues.</td>
<td>1987</td>
</tr>
<tr>
<td></td>
<td>T. Lundgren, P. Elander</td>
<td></td>
</tr>
<tr>
<td>29</td>
<td>Consolidation of soft soils.</td>
<td>1986</td>
</tr>
<tr>
<td></td>
<td>R. Larsson</td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>Kalkpelare med gips som tillsatsmedel.</td>
<td>1987</td>
</tr>
<tr>
<td></td>
<td>G. Holm, R. Tränk, A. Ekström</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Användning av kalk-flygaska vid djupstabilisering av jord.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>G. Holm, H. Åhnberg</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Om inverkan av härningstemperaturen på skjuvhälstighet hos kalk- och cementstabiliserad jord.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>H. Åhnberg, G. Holm</td>
<td></td>
</tr>
<tr>
<td>31</td>
<td>Kalkpelarmetoden.</td>
<td>1986</td>
</tr>
<tr>
<td></td>
<td>Resultat av 10 års forskning och praktisk användning samt framtida utveckling.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>H. Åhnberg, G. Holm</td>
<td></td>
</tr>
</tbody>
</table>
The Swedish Geotechnical Institute is a government agency dealing with geotechnical research, information and consultancy. The purpose of the Institute is to achieve better techniques, safety and economy by the correct application of geotechnical knowledge in the building process.

**Research**

Development of techniques for soil improvement and foundation engineering. Environmental and energy geotechnics. Design and development of field and laboratory equipment.

**Information**

Research reports, brochures, courses. Running the Swedish central geotechnical literature service. Computerized retrieval system.

**Consultancy**

Design, advice and recommendations, including site investigations, field and laboratory measurements. Technical expert in the event of disputes.

**STATENS GEOTEKNISKA INSTITUT**  
**SWEDISH GEOTECHNICAL INSTITUTE**

S-581 01 Linköping, Sweden  
Tel. 013·11 51 00, Int + 46 13 115 100  
Telex 50125 (VTISGI S)  
Telefax 013·13 16 96, Int + 46 13 13 16 96