Consolidation of soft soils

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PREFACE

This research report deals with the consolidation of soft soils. The topic has been of major concern in the research activities at the Swedish Geotechnical Institute since it started in 1944. The first test fill was constructed in 1945 and this, and successive fills, have been under observation since then. Parallel to this, the techniques and equipments for testing, instrumentation and sampling in the field as well as testing in the laboratory have been developed. The results have served to increase the understanding of soil behaviour during consolidation and to increase the ability to predict it.

The report includes results from a number of interrelated research and development projects at the Swedish Geotechnical Institute as well as experience gathered from consulting activities.

It also includes updating of the long-term observations at the old test fields at Skå-Edeby and Lilla Mellösa.

The report contains results obtained in ongoing joint research between the Department of Geotechnics at the Agricultural University of Warsaw and the Swedish Geotechnical Institute.

The calculation method CONMULT originates from Laboratoires des Ponts et Chaussées in Paris. It was modified at Université Laval in Quebec and this version came to the Swedish Geotechnical Institute as part of the research cooperation between the Institute and Université Laval. Further modifications have been made at the Institute.

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Rolf Larsson
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SUMMARY

Consolidation characteristics for soft soil are normally determined in small-scale oedometer tests in the laboratory. The technique for such tests has been developed and continuous stress-strain and strain-permeability relations can be obtained. The stress-strain relations, however, are time/strain rate dependent as the soft soils exhibit creep effects.

The creep effects in terms of coefficients of secondary consolidation have empirically been found to be a function of the void ratio. In mineral soils, the coefficient of secondary consolidation and its variation with compression can for most applications be calculated from the natural water content with sufficient accuracy. In a similar way, the creep effects in peat can be evaluated from the degree of humification. Calculation of settlement and the course of consolidation has classically been made according to Terzaghi's (1923) theory for one-dimensional consolidation. This theory assumes, however, that the modulus of compression and the permeability are constants and are independent of time.

Bjerrum (1967 and 1972) developed a general model for the effect of creep in terms of settlements and development of quasi preconsolidation pressures and increase in shear strength. This type of soil behaviour was experimentally demonstrated by Berre and Iversen (1972) and a mathematical solution for a single uniformly loaded layer was worked out by Garlanger (1972).

Today, advanced multilayer computer programmes are available which can take into account time effects as well as the variation of the compression characteristics during the consolidation process.

Initial "elastic shear" deformations have usually been estimated by crude empirical relations based on the undrained shear strength. Based on the investigations by Foott and Ladd (1981) an improved empirical relation for the modulus of elasticity is proposed.
In this relation, the modulus of elasticity is estimated from the plasticity of the soil, the undrained shear strength as determined by field vane tests and the factor of safety against undrained failure.

The field observations at the old test fields at Mellösa and Skå Edeby have been updated and new samplings and laboratory tests have been performed. Results from an instrumented embankment on a peat bog and test embankments on organic soil have also been studied.

These studies have shown that the behaviour of the soil is in general agreement with the behaviour postulated by Bjerrum in that creep effects occur during the process of consolidation and that quasi preconsolidation pressures and related increases in shear strength develop.

A simple approach is used to apply the results from small scale and relatively rapid oedometer tests to full-scale field conditions. In this way, the empirical observation that the pore pressure development in the field is directly related to the preconsolidation pressure measured in the laboratory is used.

A revised version of the multilayer computer programme CONMULT has been used to calculate the courses of consolidation for the different cases. It is found that the course of consolidation and all measured factors such as settlement distribution, pore pressure dissipation, horizontal deformations and development of quasi preconsolidation pressure and shear strength can be estimated with fairly good accuracy in the actual extensively investigated cases.

It is found that the largest remaining difficulties in the predictions of settlement are associated with the estimation of permeability in nonhomogenous layers and determination of the drainage paths. Another problem is to define what the relevant "normal" ground water conditions are and how to account for the changes and fluctuations in ground water level.

It is also found that the permeabilities and drainage conditions are very important also for the development of creep deformations. This may not be sufficiently apparent from the usual simplified presentation of Bjerrum's model where the creep effects are related to time for sustained load only.
Measured and calculated factors during the course of consolidation for the φ35 m test fill without vertical drains at Skå Edeby.

The calculations have been made at the centre of the loaded areas where conditions similar to the oedometer case prevail. Calculations of the course of consolidation at the outer parts of the loaded areas also have to take into account the horizontal water flow and the change in deformation characteristics when the shear stresses increase and the principal stresses rotate.
NOTATIONS AND SYMBOLS

\( a_s \)  Swelling index
\( b \)  Load factor
\( C_c \)  Compression index \( \Delta e / \Delta \log \sigma' \)
\( C_\alpha \)  Secondary compression index \( \Delta e / \Delta \log t \)
\( C \)  Correction defined in Fig. 7 Secondary swelling parameter.
\( c' \)  Effective cohesion intercept
\( \text{CRS-test} \)  Oedometer test, constant rate of strain
\( E \)  Modulus of elasticity (Young's modulus)
\( e \)  Void ratio
\( \dot{e} \)  Rate of void ratio change
\( e_0 \)  Initial void ratio
\( f \)  Function
\( g \)  Gravity (9.81 m/s^2)
\( H \)  Degree of humification. Height
\( I_p \)  Plasticity index
\( k_{onc} \)  Coefficient of earth pressure in normally consolidated state
\( k \)  Permeability
\( k_i \)  Initial permeability
\( M \)  Oedometer modulus
\( M_L \)  Oedometer modulus in linear range defined in Fig. 7
\( M_0 \)  Initial oedometer modulus
\( M_{r1} \)  Oedometer modulus at reloading
\( M_s \)  Oedometer modulus at swelling
\( M' \)  Modulus number
\( m_j \)  Modulus number
\( p' \)  Isotropic effective stress \( (\sigma_1' + \sigma_2' + \sigma_3')/3 \)
\( S \)  Swelling
\( s \)  Second
\( S_t \)  Sensitivity
\( t \)  Time
\( u \)  Pore pressure
\( u_b \)  Pore pressure at undrained end of oedometer specimen
\( u_c \)  Pore pressure due to creep effects
\( w_0 \)  Initial water content
**W_L**  Liquid limit  
**W_N**  Natural water content  
**W_P**  Plastic limit  
**Z**  Depth  
**α_s**  Coefficient of secondary consolidation, \( \Delta \varepsilon / \Delta \log t \)  
**β**  Stress exponent  
**β_k**  Coefficient of permeability change, \( - \Delta \log k / \Delta \varepsilon \)  
**β_α_s**  Coefficient of change in secondary compression, \( \Delta \alpha_s / \Delta \varepsilon \)  
**e**  Compressive strain  
**e_2**  Compression index  
**e_c**  Compression due to creep effects  
**e_CR**  Critical compression  
**ν**  Poisson's ratio  
**p**  Unit weight  
**p_w**  Unit weight of water  
**σ**  Total stress  
**σ_{oct}**  Octahedral stress \( (\sigma_1 + \sigma_2 + \sigma_3)/3 \)  
**σ_v**  Vertical stress  
**σ'**  Effective stress  
**σ_0**  Initial effective stress  
**σ_c**  Preconsolidation pressure  
**σ_h**  Effective horizontal stress  
**σ_j**  Reference stress  
**σ_L**  Effective stress defined in Fig. 7  
**σ_u**  Effective vertical stress to which unloading has been made  
**σ_v'**  Effective vertical stress  
**τ_{fu}**  Undrained shear strength  
**φ'**  Effective angle of friction  

**CONMULT**  Multilayer computer programme for calculation of consolidation  
**DG**  Department of Geotechnics, Agricultural University of Warsaw  
**SGI**  Swedish Geotechnical Institute
1. INTRODUCTION

One of the major tasks in soil mechanics is the prediction of settlements and their development with time. The classical theory of one-dimensional consolidation was developed by Terzaghi in 1923. In this theory, the relation between void ratio and effective stress is considered as a unique function independent of time. Other soil properties such as compressibility and permeability have often been treated as constants independent of stress level and change in void ratio.

That the assumption of an effective stress-strain relation independent of time is often a rather crude assumption can readily be observed in laboratory tests on high plastic clays which exhibit large amounts of time-dependent creep. Numerous observations have also been made in the field of creep settlements continuing after complete pore pressure equalization has occurred and thus at constant effective stress. Long-term observations in Sweden also show settlements and pore pressures that widely deviate from the classical Terzaghi theory in terms of magnitude as well as their course with time, even when changes in compressibility and permeability are accounted for.

The time-dependency of the effective stress-strain relation has been given many names, such as creep, secondary compression, plastic resistance to compression, time-resistance or strain rate effects, which are all used to describe the same phenomenon. Many attempts have been made to incorporate these time effects in the calculations of settlement. In the simplest forms, Terzaghi's theory has been assumed to be valid during primary consolidation, i.e. until all excess pore pressures caused by the load increment have dissipated. Secondary compression has thereafter been assumed to commence and proceed at a continuously decreasing rate as the secondary settlement is assumed to be a linear function of the logarithm of time. (Buisman 1936, Haefeli and Schaad 1948, Koppejan 1948).

The first theory where secondary creep effects were at least partly involved in the process of primary consolidation was presented by Taylor and Merchant (1940) and a first model for a general variation of void ratio versus effective stress and time was outlined by Taylor in 1942, Fig. 1.
A more general model, where the void ratio - effective pressure relation continuously changes with rate of deformation, was presented by Suklje (1957). In this model, there is no distinction in nature between the primary and secondary compressions other than the hydrodynamic delay during the primary phase Fig. 2a. Another presentation was made by Bjerrum (1967, 1972) in order to illustrate the effect on void ratio of loads sustained for long periods, partly to explain overconsolidation effects in natural clays due to geological "ageing" and partly to explain creep effects occurring with time under constructions in spite of the fact that the preconsolidation pressure had not been exceeded, Fig. 2b.

Bjerrum's model of 1972 also includes increase in undrained shear strength due to creep effects as the ratio between undrained shear strength and preconsolidation pressure is assumed to remain constant also when the preconsolidation pressure is a "quasi preconsolidation pressure" created by creep.

Theories for calculation of consolidation with different kinds of rheological models attempting to incorporate varying compressibility, permeability and creep have been presented by Tan (1957) McNabb (1960) Gibson and Lo (1961), Wahls (1962), Barden (1965) and Garlanger (1972), among others, while theories for 3-dimensional consolidation have been proposed by Biot (1941 and 1956), Tan (1957) and Gibson et al (1970), among others.
Fig. 2a. Equal strain rates versus vertical pressure and void ratio (from Suklje (1957)). $e = \text{rate of void ratio change } de/dt$. 

Fig. 2b. Effects of secondary compression on void ratio and preconsolidation pressure. (Bjerrum (1967, 1972)).
The practical use of these methods seems to have been limited as much of the input data in many cases has been difficult to determine and the calculations have been rather complex. Corroboration for the theories from field cases has in most cases also been lacking. A method of estimating settlements, including creep effects on the basis of primary compression from oedometer tests and an empirical relation between secondary and primary consolidation, which depends on load increment ratio, was suggested by Leonards (1968). This or other empirical methods may have been in wider use among engineers.

Computer development during the last decade has diminished the difficulty of complex calculations and the need for simplifications of the soil models. The soil can thus today be divided into a great number of layers with detailed descriptions of a number of varying parameters and the consolidation process can still be calculated in a few minutes.

This has also brought a swift development in advanced numerical methods for calculation of consolidation, e.g. Runesson (1978), Runesson and Booker (1982), and Small and Booker (1982). Advanced multi-layer computer programmes for one-dimensional consolidation allowing for time effects and variations in soil parameters during the consolidation process have also been developed (Magnan et al 1979, Mesri and Choi 1985).

At the same time, the models for general soil behaviour have been improved. (Schofield and Wroth 1968, Tavenas and Leroueil 1977, Larsson 1981). New testing techniques to accurately determine consolidation parameters in the laboratory have been introduced (e.g. Larsson and Sällfors 1985) and empirical knowledge of creep behaviour has improved (e.g. Mesri 1973, Mesri et al 1983 and Larsson 1981). The actual field behaviour of clays at loading has been studied in a number of well-instrumented cases, e.g. Sällfors (1975), Leroueil et al (1978) and Tavenas (1979). At SGI, long-term observations of the behaviour of instrumented fills, the oldest now 38 years, have continued (Chang 1981, Hansbo 1960, Holtz and Broms 1972, and Holtz and Lindskog 1972). Results from new field cases have also been obtained.

The present study has been performed in order to evaluate the accuracy with which the actual field behaviour can be predicted using modern computer programmes, results from modern testing techniques and incorporating empirical knowledge of soil behaviour.
2. DETERMINATION OF CONSOLIDATION PROPERTIES BY OEDOMETER TESTS

Calculation of settlements is usually based on results from oedometer tests. These can be performed either as incrementally loaded tests or as continuously loaded tests.

The conventional test procedure with incremental loading, each increment being equal to the previous load and a new increment every 24 hours, was suggested by Terzaghi in 1925 and has been widely used since then. During the test, the sample is drained from both ends and readings of the compression are taken in a time sequence enabling a plot of the time-settlement curve for each increment.

The results from incremental oedometer tests are usually presented in a diagram where the strain at the end of each step is plotted against the consolidation pressure. In this plot the vertical effective pressure is in log-scale, Fig. 3.

For determining the preconsolidation pressure $\sigma_c$ the Casagrande method illustrated in Fig. 3 has been widely used (Casagrande, 1936).

![Diagram showing the Casagrande method for evaluating the preconsolidation pressure.](image)
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.

Apart from the preconsolidation pressure, the compression index $C_c$ or $\varepsilon_2$ is also evaluated from the oedometer curve. The presentation of the oedometer curve in a diagram with strain in linear scale and pressure in log scale was chosen as the curve then appeared to be a straight line after the preconsolidation pressure.

The compression index $C_c$ is determined as

$$
\frac{\Delta \varepsilon}{\log \frac{\sigma' + \Delta \sigma'}{\sigma'}} \quad \text{or} \quad \frac{\Delta \varepsilon}{\Delta (\log \sigma')}
$$

To avoid the determination of the void ratio, the compression index $\varepsilon_2$ is used in Sweden where $\varepsilon_2$ is the relative compression of the sample at a doubling of the vertical pressure, Fig. 4b.

Fig. 4a. Evaluation of Compression index $C_c$.

4b. Evaluation of Compression index $\varepsilon_2$. 
The relation of these compression indexes is

\[ e_2 / \log 2 = C_C / (1 + e_0) \]

Unfortunately the assumption that the oedometer curve should be a straight line for stresses higher than the preconsolidation pressure in this plot is not valid in soft Swedish clays and this method of describing the compressibility is therefore an approximation valid within a small stress range only.

Soil compressibility is therefore often expressed by tangent modulus \( M \), Odhe (1951), Janbu (1967), Brinch-Hanssen (1966) and others

\[ M = m_j \sigma' \left( \frac{\sigma'}{\sigma_j^0} \right)^{1-B} \]

where

\( m_j \) = modulus number

\( B \) = stress exponent

\( \sigma' \) = effective vertical stress

\( \sigma_j^0 \) = reference stress (usually 100 kPa)

This method of describing compressibility of soft clays is not general either, but the approximation can be used for a larger stress interval than the compression index.

Alternatively, the compression index can be expressed as a function of the effective stress (Mesri and Godlewski 1977). The results from incremental oedometer tests can also be expressed by the same parameters as those used for the constant rate of strain test.

The different time-settlement curves for each load increment are plotted with deformation versus the logarithm of time. Fig. 5.
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 either

\[ \alpha_s = \frac{de}{d\log t} \]

or

\[ C_\alpha = \frac{de}{d\log t} \]

Previously, a coefficient of consolidation has been evaluated from the time-settlement curves and from this coefficient also, indirectly, the permeability. This has, however, been done under the assumption that Terzaghi's theory of consolidation is valid. Comparisons of permeabilities indirectly evaluated in this way from time-settlement curves and directly measured permeabilities have shown that the former are often grossly in error (Tavenas et al 1983).

The oedometers for incremental tests should therefore be modified to enable direct measurements of permeability at different stages of the test if this property is to be measured in this type of test.

The coefficient of consolidation is not a separate property but the product of compression modulus and permeability divided by two constants, and has thus not to be determined.
In Sweden, the incremental tests have mainly been replaced by automatic continuous consolidation tests which are performed with a constant rate of strain, CRS-tests. CRS-tests are in reality performed with a constant rate of deformation, but the notation CRS-test for this type of test is internationally used.

In the CRS-test, the sample is placed in an oedometer where drainage is allowed at the top surface only. The oedometer is placed in a compression test machine and the sample is compressed with a constant rate of deformation. During the compression of the sample the deformation, the applied load and the pore pressure at the lower undrained end of the sample are continuously recorded. Fig. 6.
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. 7.

**Fig. 7.** Results from CRS-test and evaluation of compression and permeability properties.
The results from CRS-tests in terms of stress-strain relations are dependent on the rate of deformation at which the test has been performed. For tests performed at rates lower than a critical rate, the results can be evaluated with consideration to rate effects as follows (Larsson and Sällfors 1985):

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 \( \sigma_C \). 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 \( \sigma_C \) 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 \( \sigma_C \). At \( \sigma_C \) the modulus is assumed to drop instantaneously to the second constant modulus \( M_L \). 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 \( \sigma_0 - \sigma_C \) where \( M=M_0 \)
2. The part in the stress interval \( \sigma_C - \sigma_L \) where \( M=M_L \)
3. The part in the stress region where \( \sigma' > \sigma_L \) and where \( M=M_L + M' (\sigma' - \sigma_L) \).
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 $M_0 = 250 \tau_{fu}$ or $M_0 = 50 \sigma_C^c$. To obtain a useful value of $M_0$ in the laboratory, the sample has to be unloaded when $\sigma_C^c$ is just exceeded to the "in situ" effective vertical stress $\sigma_0^e$. 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 $e=0$ and the decrease in permeability with compression is expressed by the parameter $\delta_k = - \Delta \log k / \Delta e$.

Tests on dry crusts, silts and remoulded clays can be evaluated using the same parameters but 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 is probably 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.

The results in terms of stress-strain relations from all types of tests are rate-dependent. At incremental loading tests, the results depend on the load increment ratios used, the duration of each load increment and the time during this period at which the stress-strain relation is evaluated. In CRS-tests, the measured stress-strain relation is a function of the rate of deformation at which the test was performed. The evaluation used in Sweden compensates for these effects down to a certain rate but the stress-strain relations at even lower rates become
different. In the conventional evaluation of incremental tests, the deformation 24 hours after load application is plotted against effective stress. At this stage, all excess pore pressure has normally disappeared and the sample is deformed at a rate governed by the coefficient of secondary consolidation $\alpha_s$. Considering the concept of Suklje (1957) that each void ratio-effective pressure relation corresponds to a certain rate of deformation, the stress-strain relation evaluated from 24-hour values in incremental tests corresponds to a rate of deformation of $\alpha_s \cdot 5 \cdot 10^{-6} \text{1/s}$, where $\alpha_s$ is expressed as a decimal number.

The stress-strain curves from CRS-tests evaluated according to Swedish practice have been shown to be practically identical to curves from conventionally evaluated standard incremental tests. This means that the stress-strain relations evaluated from CRS-tests also correspond to a strain rate of $\alpha_s \cdot 5 \cdot 10^{-6} \text{1/s}$. The coefficient of secondary consolidation is not a constant during the test but varies with deformation/effective stress level and the stress-strain relations from both types of oedometer tests thus represent a correspondingly varying strain rate.

The swelling properties of a clay at unloading are best determined by stepwise unloading test. These properties are even more time dependent than the compression characteristics as the rate of secondary swelling is much higher in relation to the swelling modulus than the corresponding relation in compression (Mesri et al. 1978). The procedure in the stepwise unloading tests is principally the same as in incremental loading tests, but instead of doubling the applied load as in loading tests the load is reduced to half of the previous load.

Other decrements may be used and when only the recompression modulus is of interest the load is reduced to the lowest "in situ" effective stress at unloading in a single step.

Readings of the deformation are taken in a time-sequence enabling a plot of the time-deformation curve for each unloading step. The results are then plotted as deformation versus the logarithm of time. Swelling after 24 hours and the rate of secondary swelling are evaluated in the same way as the corresponding values in compression. The permeability of the soil is a function of void ratio only and can be taken from the
relation determined in CRS-tests during compression or determined by permeability tests before and after the swelling process. The permeabilities during the swelling procedure can then be interpolated using 
\[ \Delta \log k = - \varepsilon_k \Delta \varepsilon. \]

Swelling tests may also be performed in advanced automatic oedometer tests where the vertical stress can be kept constant.

The recompression properties of the soil are determined in the same way as in first loading, but the test starts at the pressure to which the soil has been unloaded and has been allowed to swell for.
3. EMPIRICAL SOIL BEHAVIOUR

3.1 Observed behaviour in laboratory tests

3.1.1 Secondary consolidation

The coefficient of secondary consolidation $a_s$ is not a constant.

From incremental oedometer tests it can be observed that $a_s$ varies from load step to load step and also in some stress intervals varies significantly within the load step. This is illustrated in Fig. 8.

Fig. 8. Types of time-settlement curves from standard incremental test.
The first two curves in Fig. 8 represent loads well below the preconsolidation pressure and the values of $\alpha_s$ are very small. The third curve represents a load slightly below the preconsolidation pressure. Here the coefficient of secondary consolidation has a low value just after the pore pressure equalization has occurred but this value does not remain constant and increases with time. The fourth curve represents a load where the preconsolidation pressure is reached but not much exceeded. In this curve, there is no typical reversed s-shape but the curve bends downwards more and more. Pore pressure equalization occurs somewhere along the curve, but this cannot be seen from its shape. With standard increments, curves 3 and 4 never appear in the same test and sometimes, if the preconsolidation pressure happens to be in the middle between two loads, neither of them will occur. The curves representing loads well over the preconsolidation pressure become the reversed s-shape, but $\alpha_s$ is not a constant and decreases with time and further compression.

From tests on Swedish clays, it has been shown that the coefficient of secondary consolidation can be related to the relative compression, or void ratio, whereby a unique curve is obtained, Fig. 9. (Larsson 1981).

The coefficient of secondary consolidation is very low until a certain compression is reached, after which it increases very rapidly up to a maximum value and then slowly decreases with further compression.

The critical compression where $\alpha_s$ starts to increase corresponds to an effective vertical stress of about 0.8 $\sigma_C$ for clays.

The variation of the coefficient of secondary consolidation with deformation resembles the inverse to the relation between compression modulus and deformation. Mesri and Godlewski (1977) proposed that the coefficient of secondary consolidation could be expressed as a constant relation to the compression index $C_C$. This is a fairly good approximation, but the relation varies somewhat with stress level within the test and it varies considerably between different soils.

Mesri (1973) collected results from reported tests and showed that the coefficient of secondary consolidation is mainly dependent on water content, Fig. 10.
Fig. 9. Coefficient of secondary consolidation versus relative compression. Bäckebol clay, 8 m.

Fig. 10. Coefficient of secondary consolidation for soil deposits (from Mesri 1973) $\varepsilon_0 = \alpha$
This finding was further supported by tests on Swedish clays (Larsson 1981) and data have systematically been collected since then. The accumulated data for maximum coefficient of secondary consolidation evaluated from oedometer tests and associated water contents at the actual deformations are shown in Fig. 11.

**Fig. 11.** Relation between maximum coefficient of secondary consolidation and water content.
From Fig. 11 it can be seen that there is a clear relation between the coefficient of secondary consolidation and the associated water content. This relation, however, varies with different types of soils. A correlation with void ratio instead of water content may give a more unified relation, but the void ratio is seldom determined in Sweden and especially not for organic soils. A relation between void ratio and coefficient of secondary consolidation was proposed by Wahls (1962) and this relation agrees well with the values found for inorganic Swedish clays.

If the decrease in coefficient of secondary consolidation with further compression after the maximum value is studied for the different tests, it is found that the decreasing coefficient follows the same relation for $\alpha_s$ versus water content (void ratio) as the maximum values. The variation of $\alpha_s$ during compression can be written as

$$\alpha_{s_{\text{max}}} = \alpha_{s_{\text{max}}} - \beta_{\alpha_s} \cdot \Delta \varepsilon$$

where

$\alpha_{s_{\text{max}}} =$ coefficient of secondary consolidation at relative compression $\varepsilon$

$\alpha_{s_{\text{max}}} =$ maximum coefficient of secondary consolidation

$\beta_{\alpha_s} =$ constant $= \Delta \alpha_s / \Delta \varepsilon$

$\Delta \varepsilon =$ relative compression between the actual compression and the compression where $\alpha_{s_{\text{max}}}$ was measured.

For peats, however, the maximum coefficient of secondary consolidation seems to be related to the degree of humification rather than the water content, Fig. 12.

The decrease in $\alpha_s$ with compression follows the decrease in water content also for peats.

A fairly good idea of the magnitude of the coefficient of secondary compression and its variation during the consolidation process can thus be obtained on the basis of the type of soil and its water content.
At first loading in the oedometer of a soil sample exhibiting a preconsolidation pressure, the stress-strain relation becomes almost linear up to a certain deformation where a break occurs in the curve. The pressure at this point depends on the loading procedure but the deformation seems to be the same for "identical" samples (Leonards 1979). This critical deformation, however, is not a unique value but depends on the load and deformation history of the soil (Larsson 1981). In a wider aspect, it also depends on what stress-path is followed (Tavenas and Leroueil 1977).

It has also been observed that secondary compressions are very small up to a deformation somewhat lower than the critical deformation. From this deformation, the coefficient of secondary consolidation $\alpha_s$ increases linearly with increasing deformation to reach a maximum at the critical deformation. When modelling soil behaviour, consideration thus has to be taken to the stress and deformation history of the soil in situ.
Soils exist in situ with varying degrees of overconsolidation from practically normally consolidated soils, which are frequent some distance below the ground surface in Sweden, to heavily overconsolidated soils. The overconsolidation often depends on a reduction of previous effective stress due, for example, to erosion, deglaciation, removed constructions or rising ground water levels. The overconsolidation effects may also depend on chemical bonding or "ageing" due to secondary consolidation (Bjerrum 1967).

Other causes of overconsolidation are drying-out of the soil, fluctuating ground water levels, suction from roots, cyclic loads and for coarser soils, compaction and vibrations. There are still many soils for which the overconsolidation effects cannot be readily explained.

3.1.2 Swelling

If the effective stresses in a soil without bonding effects are decreased sufficiently, the soil starts to swell. If a clay has just consolidated for a load increase over the previous preconsolidation pressure, secondary compression will proceed at a relatively high rate. A small reduction in load will halt this compression for a while or there may be a small swelling before compression starts again at a reduced rate. If the load reduction is large enough, secondary compression stops completely and for even larger load reductions the clay will swell. Like compression, this swelling is time-dependent. At unloading, the pore pressure in the clay drops. There is a time-lag in the swelling while there is a hydraulic gradient in the clay and water is absorbed as fast as the permeability and access to water permits. After the gradient has evened out, there is secondary swelling at unloading similar to secondary compression at loading.

This is illustrated in Fig. 13, where two oedometer tests with stepwise loading and unloading are shown. The time-swelling curves can be seen to show the same shape as the time-settlement curves. The swelling modulus is high close to the preconsolidation pressure, but decreases as the effective stress decreases and is very low at low effective stresses.
Fig. 13. Oedometer tests with incremental loading and unloading on Norsholm clay.
No detailed study of the secondary swelling rates has been performed for Swedish clays but Mesri et al (1978) have reported a comprehensive study on shales. Certain similarities between the results from this study and observed behaviour for Swedish clays can be seen and a tentative model for deformations during unloading and swelling can be outlined.

The swelling modulus at unloading decreases continuously with decreasing effective stress. The swelling modulus $M_s$ can be written as

$$M_s = \sigma' / a_s$$

where

- $M_s$ = swelling modulus
- $\sigma'$ = effective vertical stress
- $a_s$ = swelling index

When a normally consolidated soil is unloaded by only a small load increment, the secondary compression will continue but at a reduced rate. When the load reduction becomes large enough the direction of the secondary deformations will change and secondary swelling starts. The load at which this turning point occurs can be written as $b \cdot \sigma_c'$ where $b$ is the load factor. The secondary swelling at effective stresses lower than $b \cdot \sigma_c'$ can be expressed as $c \cdot S$ where $c$ is a constant and $S$ is the accumulated swelling.

The net effect of these processes for the 24 hour reading in the oedometer test is that for small load reductions there is a small compression in spite of the unloading. The deformation at the load $b \cdot \sigma_c'$ is often about the same as when the unloading started and for larger load reductions the soil will actually swell, Fig. 14.

From tests on Swedish soils, the following simple equation can be suggested for swelling in incremental oedometer tests for the first unloading cycle with 24 hours' duration of the decrements.
S = a_s \ln \frac{b \cdot \sigma_c'}{\sigma_u'}

where
S = total relative increase in sample height
a_s = swelling index
\sigma_c' = preconsolidation pressure
b = load factor when swelling overcomes secondary compression
\sigma_u' = effective pressure after pore pressure equalisation

This equation has been used for a variety of soils and found to fit the laboratory test results satisfactorily.

For soft clays, the swelling index a_s has been found to vary between 0.007 and 0.012 and the load factor b is about 0.8. a_s decreases with increasing grain size and is about 0.001 for sand and gravel and b increases with increasing grain size to become 1 for gravel.

a_s-values for a number of Swedish soils are plotted against average grain size d_{50} in Fig. 15.
At the top of the figure, ranges for approximate b-values are given. A study by Inganäs (1978) indicates that the load factor b may be about 0.8 also for peats.

For clays, there is probably some correlation between plasticity index, swelling index and the load factor. Possible correlations are sketched in Fig. 16, but as Ip has not been determined in most cases the points obtained so far are too few to give a definite relation.
This method of calculating swelling is confined to what actually occurs in an oedometer test with a certain loading-unloading sequence. The deformations in an actual case will, among other things, depend on how long the soil was allowed to develop secondary deformations before it was unloaded, the unloading sequence and what amount of secondary swelling the soil has developed.

To calculate these deformations, information is required on how the secondary deformations vary with stress and deformation. A tentative assumption can be made that the rate of secondary compression during unloading decreases linearly with decreasing stress within the stress interval $\sigma'_c$ to $b \cdot \sigma'_c$ according to

$$\alpha_s = \alpha_{s_{\text{max}}} \left( \frac{\sigma'_c - b \sigma'_c}{(1-b)\sigma'_c} \right)$$

where

- $\alpha_s$ = rate of secondary compression
- $\alpha_{s_{\text{max}}} = \text{rate of secondary compression during compression at loading}$
- $\sigma'_c$ = effective vertical stress
- $b$ = load factor
- $\sigma'_c$ = preconsolidation pressure

Secondary swelling rates at effective vertical stresses lower than $b \cdot \sigma'_c$ can be assumed to be governed by

$$\alpha_s = -c \cdot S$$

where

- $\alpha_s$ = rate of secondary swelling
- $c$ = constant
- $S$ = total relative swelling

3.1.3 Recompression

The modulus at recompression after swelling is dependent on the magnitude of the swelling. Samples that have had only a small load reduction have not swelled at all and have a relatively high recompression modulus.
A characteristic feature of samples that have undergone swelling is that when they are reloaded they will show an almost constant compression modulus up to the effective pressure $b \cdot \sigma_c$ and the compression at this pressure will be equal to the previous maximum compression. $b$ is the same load factor as the factor $b$ that determines at which pressure the sample starts to swell.

This is illustrated in Fig. 17, where a number of tests on the same clay have been preconsolidated for the same effective stress and then unloaded to different loads. The curves have been slightly adjusted along the $e$-axis to give the same compression at the end of the first loading.

Fig. 17. Effect of swelling on recompression. Curves slightly adjusted in the vertical direction.
Fig. 18 shows results from a number of tests on the same clay where the samples have been consolidated to different effective stresses, unloaded to low stresses and reloaded after swelling. The curves have been slightly adjusted along the ε-axis to fall on the same virgin compression curve. The tests all start to swell at the same b-value and the recompression curves all intersect the swelling curve at \( \sigma' = b \cdot \sigma'_C \).

\[ \text{Fig. 18. Swelling and recompression after different preconsolidation. Curves slightly adjusted.} \]

The modulus at recompression \( M_{r1} \) in the stress interval \( \sigma'_u \) to \( b \cdot \sigma'_C \) thus becomes

\[
M_{r1} = \frac{b \cdot \sigma'_C - \sigma'_u}{a_S \cdot \ln \left( \frac{b \cdot \sigma'_C}{\sigma'_u} \right)}
\]
where $\sigma_u^i$ is the effective pressure to which the sample was unloaded and $a_s$ and $b$ are the swelling parameters. This formula is also a simplification valid for the standard procedure in the oedometer.

If the effect of secondary swelling is incorporated, the modulus at recompression becomes

$$M_{r1} = \frac{b \cdot \sigma_c^i - \sigma_u^i}{S}$$

where $S$ is the total swelling.

The modulus at recompression for soils which have been unloaded only to stresses higher than $b \cdot \sigma_c^i$ is more difficult to evaluate. If the soil unloaded to stresses in the interval $b \cdot \sigma_c^i$ to $\sigma_c^i$ is assumed to respond in a way similar to soil unloaded to lower stresses, the modulus of recompression for these cases can be written as

$$M_{r1} = \frac{\sigma_u^i}{a_s} \quad \sigma_u^i \geq b \cdot \sigma_c^i$$

The modulus of recompression calculated without regard to secondary swelling becomes a function of overconsolidation ratio, swelling index and load factor. The modulus at recompression calculated with a typical value of $a_s=1\%$ and $b=0.8$ is shown in Fig. 19.

![Graph](image-url)

**Fig. 19.** Calculated recompression modulus versus overconsolidation ratio.
From Fig. 19 it can be seen that empirical values of recompression modulus of $M_r \approx 50 \sigma_C$ (or $M_r \approx 250\tau_{fu}$) are averages of a wide range of values. This range becomes even wider considering that $a$ and $b$ vary with type of soil and that the amount of secondary swelling varies.

The compression modulus in the stress interval $\sigma_0$ to $\sigma'$ might be different if the overconsolidation is an effect of bonding (cemented clays, structured clays) or "ageing" due to long-term secondary consolidation. In Scandinavia, however, the same type of empirical relations for compression modulus at stresses lower than the preconsolidation pressure have been used regardless of the cause of overconsolidation.

Overconsolidation effects in clays in eastern Canada are often attributed to cementing or "structure" and extreme care has to be taken to preserve these effects during sampling. Still, careful investigations have given values of compression modulus for the actual stress range that are of the same order as for Scandinavian soils which have been unloaded (Leahy 1980).

The creep effects vary during reloading. The secondary swelling does not stop immediately a small load increment is applied, but continues within a small stress range at rates that depend on how much secondary swelling had developed before the load was increased. This may somewhat affect the modulus at recompression. Within the stress interval where this swelling process continues at reloading, the net effect of compression and secondary swelling may be that no significant deformation or even a small swelling occurs.

At further reloading there are no significant creep effects until the effective stress $b\sigma_C$ is reached. From this stress and associated deformation, the rate of secondary deformation increases linearly with increasing deformation until the maximum value, which is identical to the value for the actual deformation at first loading, is reached.

3.1.4 Tentative model for moduli and creep

A tentative model for the variation of modulus and creep during loading - unloading - reloading can be summarized as in Fig. 20 and Table 1.
Fig. 20. Schematic variation of modulus and creep during loading, unloading and reloading.

Table 1. Schematic variation of modulus and creep during loading, unloading and reloading.

<table>
<thead>
<tr>
<th>MODE</th>
<th>CONDITION</th>
<th>STRESS INTERVAL</th>
<th>MODULUS</th>
<th>COEFF. OF SECONDARY CONSOLIDATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loading</td>
<td>(\sigma'<em>o &lt; b \sigma'</em>{c_1})</td>
<td>(\sigma'<em>o - b \sigma'</em>{c_1})</td>
<td>(M_o)</td>
<td>Negligible</td>
</tr>
<tr>
<td></td>
<td>(\sigma'<em>o &gt; b \sigma'</em>{c_1})</td>
<td>(\sigma'<em>o - \sigma'</em>{c_1})</td>
<td>(b \sigma'<em>{c_1} - \sigma'</em>{c_1})</td>
<td>(\frac{\sigma_{\text{max}}}{\sigma_{\text{c_1}}(1-b)/M_o} \leq \alpha_{\text{max}})</td>
</tr>
<tr>
<td></td>
<td>(\sigma &gt; \sigma'_{c_1})</td>
<td>(\sigma'<em>o - \sigma'</em>{c_1})</td>
<td>(M_L)</td>
<td>(\frac{\sigma_{\text{max}}}{(\sigma'<em>o - \sigma'</em>{c_1})/M_o} \leq \alpha_{\text{max}})</td>
</tr>
<tr>
<td></td>
<td>(\sigma &gt; \sigma'_o)</td>
<td>(\sigma'_o - \sigma'_o)</td>
<td>(M_L + M'(\sigma'_o - \sigma'_o))</td>
<td>(\alpha_{\text{max}} = \alpha_{\text{max}} - \rho \sigma'<em>o (\sigma</em>{\text{c_1}} - \sigma'_o))</td>
</tr>
<tr>
<td>Unloading</td>
<td>Eventual small interval</td>
<td>Eventual small interval</td>
<td>(M_o = \sigma'<em>o / \sigma</em>{c_5})</td>
<td>(\frac{\sigma'<em>o - b \sigma'</em>{c_2}}{(1-b) \sigma'_{c_2}})</td>
</tr>
<tr>
<td></td>
<td>(\sigma'<em>o &lt; b \sigma'</em>{c_2})</td>
<td>(\sigma'<em>o - b \sigma'</em>{c_2})</td>
<td>(b \sigma'<em>{c_2} - \sigma'</em>{c_2})</td>
<td>(\frac{\sigma_{\text{max}}}{\sigma_{\text{c_2}}(1-b)/M_{t_1}} \leq \alpha_{\text{max}})</td>
</tr>
<tr>
<td></td>
<td>(\sigma &gt; \sigma'_o)</td>
<td>(\sigma'_o - \sigma'_o)</td>
<td>(M_{t_1} = \sigma_{c_2} / \sigma_{c_5})</td>
<td>(\frac{\sigma_{\text{max}}}{\sigma_{\text{c_2}}(1-b)/M_{t_1}} \leq \alpha_{\text{max}})</td>
</tr>
</tbody>
</table>
In this model, the modulus $M_0$ should preferably be determined by oedometer tests with unloading from $o'_C$ to $o'_0$ and reloading. $M_0$ should then be evaluated from the reloading curve. Alternatively, $M_0$ is estimated taking the stress-history and swelling-compression characteristics into account.

The critical deformation $\varepsilon_{CR1}$ is calculated as $(o'_C - o'_0)/M_0$. $o'_C$ is preconsolidation pressure of the soil as measured in the oedometer test.

$\sigma'_C$ is the new preconsolidation pressure developed if the soil is compressed more than $\varepsilon_{CR1}$. $\sigma'_C$ is the pressure on the oedometer curve for first loading corresponding to the maximum compression (minimum void ratio) that has been reached. $\sigma'_C$ thus increases with prolonged secondary consolidation.

The new critical deformation $\varepsilon_{CR2}$ depends on the previous deformation history and can be calculated from

$$\varepsilon_{CR2} = \varepsilon_{b} + \frac{\sigma'_C}{\sigma'_u} (1-b)/M_r$$

and

$$\varepsilon_{CR2} = \varepsilon_{b} + \frac{(\sigma'_C - \sigma'_u)}{\sigma'_u} - \frac{a_s}{\sigma'_u}$$

Deformations calculated by the various moduli, except $M_5$ in the interval $\sigma'_C - b \cdot \sigma'_C$ and $M_r$ in the first small stress interval where secondary swelling may occur, include secondary consolidation occurring at deformation rates faster than $a_s \cdot 5 \cdot 10^{-6}$ l/s. For slower processes, the effects of secondary consolidation or swelling have to be taken into account. In the stress intervals where deformations from moduli and from secondary consolidation occur in opposite directions, the deformations from moduli can be considered as instantaneous and independent of time, except for the hydrodynamic time lag, while the creep process goes on as before the stress change, but at a reduced rate.
3.1.5 "Undrained" modulus

When a load with limited lateral extension is applied, there will be shear deformations and lateral deformations in the soil. These deformations are assumed to occur instantaneously when the load is applied. The soil is further assumed to be undrained during the time for load application and the deformations are often calculated by using \( \nu = 0.5 \) and an undrained modulus of elasticity. This modulus is often estimated from empirical correlations with the undrained shear strength. Values for \( E \) from \( 80 \tau_{fu} \) to \( 2000 \tau_{fu} \) have been suggested. There is a clear trend that the highest values have been suggested for low-plastic clays and the lower values for organic soils.

Tavenas (1979) has pointed out that the soil during normal load application usually does not respond in a totally undrained way. Partial drainage can be expected until the effective stresses in the soil reach the preconsolidation pressures. For soils with some degree of overconsolidation, it is therefore difficult to distinguish between initial deformations and consolidation. Poulos (1972) has shown that the initial horizontal movements are far less than what corresponds to the vertical movements for stiff, overconsolidated soils. This discrepancy is reported to diminish when the soil becomes softer and more normally consolidated. Tavenas (1979) showed that once the normally consolidated stage is reached, the further initial horizontal deformations correspond to the further initial settlements. If \( \tau_{fu} \) and \( E \) are determined from triaxial tests the effect of anisotropy must be considered. If the values are obtained from direct simple shear tests or the shear strength is obtained from field vane tests, this effect is probably smaller. An undrained modulus of elasticity is not a constant but the stress-strain curve is a hyperbolic function. Nor are the deformations elastic, but become more and more plastic as failure is approached.

Foott and Ladd (1981) presented moduli determined in direct simple shear apparatus for a wide variety of soils and also recommendations for how the calculated initial deformations should be corrected for plastic deformations. The results from this study, combined with the recommendation, indicate that the modulus for initial deformations \( E \) for normally consolidated or slightly overconsolidated soil can be written
\[ E = \frac{\tau_{fu} \cdot 215 \cdot \ln F}{I_p} \]

where

- \( E \) = Modulus for initial deformation
- \( \tau_{fu} \) = Undrained shear strength from vane shear tests or direct simple shear tests
- \( F \) = Calculated factor of safety against shear failure
- \( I_p \) = Plasticity index

In this formula, the introduction of \( I_p \) takes the effect of plasticity into account and the factor \( \ln F \) accounts for the hyperbolic stress-strain relation. The formula is also in general agreement with the suggested empirical relations.

The initial deformations are calculated using theory of elasticity with \( \nu = 0.5 \) and \( E \) as an elastic Young's modulus.

When the soil is unloaded, the swelling causes a softening of the soil. The values indirectly determined in direct simple shear tests by Foott and Ladd (1981) indicate that the modulus for overconsolidated clays within a limited range (1.25<OCR<8) could be written as

\[ E = \frac{\tau_{fu} \cdot 215 \cdot \ln F}{I_p} (1.08 - 0.37 \ln OCR) \]

In these cases, however, it should be considered that the soil usually does not respond in a totally undrained way and that \( \nu \) is probably less than 0.5. In Swedish practice, the elastic moduli for overconsolidated soils are usually assumed to be 50 to 70% of the empirical relations for normally consolidated soils (Hansbo 1972). The settlements thus calculated are, however, assumed to include long-term consolidation settlements and no real reduction of the elastic modulus is normally made for overconsolidation.

A certain reduction occurs indirectly, though, as the elastic modulus is based on results from vane shear tests and the strength values obtained in these tests decrease somewhat when the over burden pressure decreases and the soil swells.
3.1.6 Permeability

The permeability of a soil is a function of its void ratio. Tavenas et al (1983) found that for the working range of compressions the permeability, as in Swedish practice, could be simplified to a straight line in the log permeability - compression plot. The logarithm of the permeability can thus be written as

\[
\log k_\varepsilon = \log k_i - \beta_k \cdot \varepsilon
\]

where

- \(k_\varepsilon\) = permeability at relative deformation \(\varepsilon\)
- \(k_i\) = initial permeability
- \(\beta_k\) = permeability change index
- \(\varepsilon\) = actual relative deformation

The permeability change index is a simple function of natural void ratio, while the initial permeability was found to be a complex function of a number of soil properties (Tavenas et al 1983).

No difference between horizontal and vertical permeability has been observed for homogenous soft clays in Sweden or eastern Canada (Larsson 1981, Tavenas et al 1983). In many cases, the soil is layered which, apart from variations in vertical permeability, also entails large differences between vertical and horizontal permeabilities. This may also be the case in some non-layered soils where particle orientation creates the same effect. Clays that have been subjected to very high stresses often have a significant particle orientation.

Anisotropy of permeability has been found for Swedish peats (Inganäs 1978) where peats with a low degree of humification had very high horizontal permeabilities as compared to vertical permeabilities. This difference decreased with increasing degree of humification and became negligible for highly decomposed peats.

Also in Swedish gyttjas with pronounced horizontal orientation of the organic particles the permeabilities have been found to be higher in the horizontal direction than in the vertical direction.
Even the most homogenous soils have natural variations in water content and void ratio. For the very homogenous Bäckebo clays, a variation in initial permeability in oedometer samples of ± 25% has thus been found (Larsson and Sällfors 1985). The corresponding scatter for homogenous clay from eastern Canada is about ± 40% (Tavenas et al. 1983). In both cases, the scatter can be directly related to the natural variation in void ratio. This means that accurate measurements of the permeability for a natural clay that can be applied to the field conditions cannot be obtained from a single test on a small sample in the laboratory. For homogeneous soils, a number of tests may be performed to give an average or tests on large samples may be performed. For layered or varved soils, the measurements of permeability should be performed on large samples, include effects of anisotropy and preferably be performed in situ.

3.1.7 Pore pressure response at loading

A number of methods have been proposed for predicting the pore pressure response in a clay mass when the stresses are changed under undrained conditions (e.g. Skempton 1954, Henkel 1960, Hoeg et al. 1968). These early methods have been based on combinations of the responses to changes in compressive stresses and in shear stresses.

A more rational method of predicting the pore pressure development was created by the introduction of the concept of Critical State Soil Mechanics (Schofield and Wroth 1968). This concept has been further developed to include the behaviour of natural soft clays and anisotropy (e.g. Wong and Mitchell 1975, Larsson 1977, Tavenas and Leroueil 1977 and 1979, Larsson and Sällfors 1981).

It has been established that natural clays have a yield surface that is a combination of normal stresses and shear stresses at which the behaviour of the material changes from elastic to plastic, Fig. 21.

The shape of the yield surface can be estimated from the preconsolidation pressure $o_c$, the coefficient of effective earth pressure in the normally consolidated stage $K_{onc}$ and the effective strength parameters $c'$ and $\phi'$. 
In undrained conditions, the pore pressure response in the elastic range will be such that the isotropic effective stress
\[ p = \frac{(\sigma_1' + \sigma_2' + \sigma_3')}{3} \]
remains constant. When the yield surface is reached, the further pore pressure development will be such that the yield surface is followed up to the point of undrained shear failure, Fig. 22.

The yield stresses and the pore pressure developments are somewhat time-dependent. Prolonged undrained conditions will cause the yield surface to shrink and the pore pressures to change. On the other hand, totally undrained conditions do not prevail indefinitely in the field.
3.2 Observed behaviour in situ

The actual behaviour of clay under embankments has been summarized by Tavenas (1979). It has been found that the pore pressure development during the loading process is considerably smaller than that predicted from totally undrained behaviour until the yield stresses are reached. For stresses within the elastic range where the compression modulus is high, a significant degree of consolidation can thus be expected to occur within the time for load application.

For soils with an overconsolidation ratio lower than 2.5 there will be a drastic change in the pore pressure development when the effective vertical stress reaches the preconsolidation pressure. From this stage, the pore pressure will respond directly to the additional increase in vertical load \( \Delta u = \Delta a \) and thus keep the effective vertical stress constant and equal to the preconsolidation pressure, Fig. 23.
This behaviour was first observed by Sällfors (1975) who used the pore pressure development at large-scale field loading tests to establish the preconsolidation pressure in situ. The values of the preconsolidation pressure obtained in this way in situ were used as references to the preconsolidation pressures evaluated from CRS-test by Sällfors' method. This method is thus calibrated versus the pore pressure development under large-scale field loading test. This approach was taken up by Leroueil et al (1978) in their investigation on pore pressure development at embankment construction. They found that the critical pressure where a direct response in the pore pressure to additional load occurred was equal to the preconsolidation pressure determined from standard incremental oedometer tests using the 24 hour values, except for soils with an overconsolidation ratio of 2.5 and higher, Fig. 24.

The difference between the critical pressure and the preconsolidation pressure for soils with a high overconsolidation ratio is caused by the stress-path for these soils being such that the yield surface is reached at critical shear stresses instead of a limiting vertical stress. The same approach to comparing preconsolidation pressures in the field and from laboratory tests has later been used by Leroueil et al (1983) and Samson et al (1983) with the same result. In the latter investigations, other criteria for the preconsolidation pressure in situ such as observed settlements were also used with the same
result, i.e. that the preconsolidation pressures evaluated from standard incremental tests are directly applicable to field conditions. In the investigation by Samson et al. (1983) a large number of CRS tests were also performed.

If the preconsolidation pressures from these tests are evaluated according to the Swedish procedure, they become equal to the preconsolidation pressures from standard incremental tests and those evaluated from the field studies (Larsson and Sällfors 1985). It is thus well established that preconsolidation pressures from oedometer tests with the established procedures for evaluation correspond to the preconsolidation pressures in the field when these refer to yield stresses where pore pressure development and settlements become drastically changed.

Tavenas (1979) also showed that there is a pronounced difference in the lateral displacements of soil measured beneath the toes of the embankments, depending on stress level. Under undrained conditions
Fig. 25a. Lateral displacement $Y_m$ versus settlement $S$ during the construction of four test embankments. (From Tavenas, 1979).

Fig. 25b. Average correlation between $Y_m$ and $S$ during construction (After Tavenas, 1979).
and \( v = 0.5 \) the lateral displacements would theoretically correspond to the vertical settlements. This, however, is not the case until the soil becomes normally consolidated. Until the yield stresses have been reached, the lateral deformations are small in relation to the settlements. On the other hand, the vertical and horizontal movements correspond very well once the soil has entered a normally consolidated state. Figs. 25a and b.

Tavenas (1979) furthermore showed that lateral deformations occur also during the consolidation process. The relative importance of these movements increases with increasing steepness of the slopes and decreasing factor of safety. The only long-term observations available indicate that the horizontal displacements decrease in relation to the vertical settlements with time. This would be in accordance with the increasing factor of safety with time.

Long-term settlements for loads below the preconsolidation pressure have not been studied in detail in Sweden. In the large study of settlements of buildings in Drammen by Bjerrum (1967) the overconsolidation was attributed to secondary consolidation and ageing. From this study, it has been concluded that 50% of the difference between \( \sigma_0 \) and \( \sigma_C \) could be used without causing any significant creep settlements. In Drammen, the average overconsolidation ratio was 1.6 and the safe effective vertical stress thus about 0.8 \( \sigma_C \).

The settlements of the buildings after 10 years are plotted against the stress level in the most compressive layer in Fig. 26.
Fig. 26. Settlements of buildings in Drammen after 10 years versus stress level in the layer of high plastic clay. Data from Bjerrum (1967) and Foss (1969).
THE MULTILAYER CONSOLIDATION PROGRAMME CONMUL T, SGI-VERSION

4.1 History

The programme CONMULT (CONsolidation of MULTilayers) for calculation of one-dimensional consolidation of soils was initially developed in France at Laboratoires des Ponts et Chaussées (Magnan et al. 1978).

The programme was further developed in conjunction with the Laval University of Quebec in Canada. As part of the research cooperation between SGI and the Laval University, a copy of the programme including modifications made at Laval was given to SGI in 1981. In that version of the programme the compressibility of the soil was expressed by compression indexes $C_s$ and $C_c$.

Creep effects were accounted for by the adoption of the simple form of Bjerrum's model where each void-ratio relation was valid for a certain time after load application and the void ratio change with time at constant load was calculated with a constant coefficient of secondary compression.

Canadian experience of the programme was that it was a very good tool for evaluating the processes in small-scale laboratory tests but less satisfactory in applications to field cases where the creep function seemed to become erroneous (Tavenas and Leroueil 1981, personal communication).

At SGI, the programme was rewritten for the compression parameters evaluated from CRS-tests which are used in Sweden and the creep function was omitted. Load reduction due to settlements was also incorporated.

As a start, a consolidation programme was obtained where the soil profile could be divided into a large number of layers with different properties and where the variation of these properties as regards compressibility and permeability, and also the change in applied load during the consolidation process, could be accounted for.
Later, a new creep function based on the empirically observed behaviour of soils, in the laboratory as well as in the field, and adapted to be valid for field cases has been added.

### 4.2 Soil Model

The model of the soil used in the calculations corresponds to the observed behaviour of the soil in the field and laboratory. The compressibility of the soil is expressed by the parameters $M_0$, $o_C$, $M_L$, $o'L$ and $M'$ as previously described. The permeability of the soil is expressed by its initial permeability $k_i$ and the permeability change index $B_k$.

The immediate pore pressure response to a stress-change is calculated by

\[ \Delta u = \Delta \sigma_{oct} \quad \text{when} \quad \sigma'_V < \sigma'_C \]

and

\[ \Delta u = \Delta \sigma_V \quad \text{when} \quad \sigma'_V = \sigma'_C \]

which entails that the effective vertical stress in undrained conditions can reach the preconsolidation pressure but not exceed it.

The creep process is expressed in analogy with the model previously described. The compression parameters used incorporate creep effects occurring at deformation rates faster than $5 \cdot 10^{-6}$ 1/s. This reference rate is compared to the calculated rate of deformation and when the calculated rate becomes lower the creep effects start. In compression, the creep effects will create a larger deformation for a given effective pressure than what is calculated by the compression parameters. To achieve this deformation, the soil has to consolidate further and a corresponding amount of pore water has to migrate out of the soil. As this is a time-dependent process, the immediate result of the creep effect is an increase of the pore pressure corresponding to the creep deformation and current modulus, Fig. 27.
The increase in pore pressure may actually lower the effective stresses so that the rate of the creep process decreases. If the decrease in effective stress is large enough, the compressive creep process will stop altogether.

At swelling, the creep effects will correspondingly create a negative pore pressure in the soil.

The creep effects in this model are thus dependent on the rate at which the permeability and drainage conditions allow them to develop and are not only related to the time after load application.

4.3 Calculation method

The soil profile is divided into a number of main layers with given properties for the middle of the layer. These properties are assumed to be valid for the entire layer. The programme then automatically divides these layers into a number of sublayers. The distribution of the sublayers depends on the consolidation properties of the main layers.
The programme uses Terzaghi's equation for one-dimensional consolidation

\[
\frac{\partial u}{\partial t} = \frac{M}{g \cdot \rho w} \cdot \frac{\partial}{\partial z} \left( k \cdot \frac{\partial u}{\partial z} \right)
\]

where

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

When creep effects create additional pore pressures \( \Delta u_{ct} \), which are also subject to Darcy's law for equalization, the consolidation equation changes to

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

The equation is solved by using of finite differences with small time steps. Continuity between the layers demands that the rate of water flow across the interface between the layers shall be constant

\[
k_n \cdot \left( \frac{\partial u}{\partial z} \right)_n = k_{n+1} \cdot \left( \frac{\partial u}{\partial z} \right)_{n+1}
\]

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

During the calculations, certain records on stress history and which creep effects have been accounted for have to be kept and checked to control the calculated creep process as intended. The stress and deformation history is also required for the updating of swelling and recompression characteristics.
Chart 1. Flowchart of the calculations in CONMUlt.
"Elastic" shear deformations during load application and eventual deformations due to lateral movements during consolidation have to be calculated separately.

In the calculation of stresses in the soil, two models can be used. Either the stress is calculated by a slightly modified Boussinesq-procedure (Brucy 1977) or it is assumed that the increase in vertical stress is equal to the applied surface load throughout the profile. ($\Delta \sigma_v = \Delta q$)

Pore pressure changes at an increase of load are calculated as

$$\Delta u = \Delta \sigma_{oct} \text{ when } \sigma'_v < \sigma'_c \text{, and}$$

$$\Delta u = \Delta \sigma_v \text{ when } \sigma'_v = \sigma'_c.$$ 

At unloading, the pore pressure changes are calculated as $\Delta u = \Delta \sigma_{oct}$.

In cases with high ground water levels, settlements will reduce the load as some masses will be submerged and this can be accounted for. In this case, the difference in effective density above and below water of the submerged masses is given as an input to the programme and the load reduction after each time step in the calculations is taken into account. The calculations are shown schematically in Chart 1.

4.4 Input to the programme

General data

- Number of main layers
- Suggested number of sublayers
  (The programme will automatically adjust this number and distribute the sublayers according to the consolidation characteristics)
- Number of points where the excess pore pressures are to be recorded.
  (The programme selects the interfaces between the main layers and a few points within these layers.)
- Code for how the degree of consolidation should be calculated
- Alpha $\frac{M \cdot k \cdot \Delta t}{g \cdot \rho_w \cdot (\Delta z)^2}$ which is
used for selection of thickness of sublayer and the time step. The value of Alpha should be less than 0.5 to ensure convergence in the calculations. Usually a value of 0.4 is used.

Properties of the main layers

- Thickness of the layers
- Code for the drainage conditions in the layers; normal, impermeable or free draining.
- Effective density of the soil in the layer (used to calculate the effective stress in situ).
- The compression characteristics at the middle of the layers; $M_0$, $\sigma_C^L$, $M_L$, $\sigma_L^L$, $M$, $\alpha_s$, and $b$.
- The creep parameters at the middle of the layers; $\alpha_{s\text{max}}$, $B_{os}$, and $c$.
- Initial permeability $k_i$ and permeability change index $B_k$ for the layers.
- Information whether the lowest boundary should be considered as impermeable or free draining.

(Not all compression and creep parameters may be relevant to the actual loading case and some can in that case be omitted. In fact, the parameter for secondary swelling c has so far not been used). The compression and permeability parameters are usually determined in CRS-tests, while the creep and swelling parameters are in most cases taken from empirical relations.

Initial pore pressure

- The initial pore pressures are given as excess pore pressures at a number of points at different depths.
- Load corresponding to the initial pore pressure.

(Used to modify the "total stress" and thereby from an actual load-pore pressure condition enable a calculation of the further development with time.)
Fig. 28. Load description.
Load description

- The external load is described by a number of load stages and first the number of load stages is given.
- The change in effective density due to settlements submerging some masses (if any) is given.
- Type of geometry of the load (e.g. circular fill, long embankment).

For each load stage the following data are given Fig. 28
- change of load
- time for load application
- total time for the load stage (load application + sustained load)
- width of the part with full load change and width of the slope.

Additional input includes the type of calculation of stress distribution that should be used and the distance from the centre of the loaded area for which the consolidation process should be calculated.
The test fills at the farm of Lilla Mellösa near Upplands Väsby were constructed by SGI shortly after the Institute was founded. They were constructed in connection with the search for a suitable site for a new airfield outside Stockholm.

At Lilla Mellösa there was a large, almost flat, area with only scattered farms. The subsoil conditions, however, were less ideal with ten to fifteen metres of highly compressible soils.

On the initiative of the Director of the Institute, Dr Kjellman, a test fill was constructed in 1945 in order to investigate if an airfield could be built on the site using preloading and prefabricated vertical drains.

The technique for installing prefabricated vertical drains was not very advanced at the time and only five metre long paper-drains could be installed.

The fill was partly unloaded about two hundred days later. In the meantime, Dr Terzahghi had been invited to Sweden to give his opinion on the suitability of the method. His recommendation was that although the method in itself was sound, there were too many shortcomings in the technique at that time and, as too little was known about the effects of secondary consolidation, the site ought not to be considered for the new airfield for the time being.

Terzahghi also predicted that this type of problem would be recurrent in Sweden and he recommended that the field tests at Lilla Mellösa should continue and be extended. In analogy with this recommendation, a new test fill was constructed in 1947. The new fill had the same load, time for load application and dimensions as the first fill but no drains were installed. The main reason for this fill was to enable an evaluation of the effects of the vertical drains.
Measurements on the fills were made periodically, but after new test fills had been constructed at Skå-Edeby to enable a closer examination of the effects of vertical drains, the interest in the fills at Mellösa waned.

In 1966, the test field at Mellösa came into focus again as Chang (1969) compiled the available earlier data and started up new investigations regarding soil compressibility, pore pressures and settlements using newer and more accurate equipment.

This investigation was completed ten years later when Chang (1981) was invited as a guest researcher to SGI. The report from 1969 was then updated with the accumulated measurements, new pore pressure measurements and further laboratory investigations. From these investigations, Chang concluded that the observations in terms of settlements, decrease in water content, pore pressures and undrained shear strength were completely incompatible with each other on the basis of conventional concepts of the process of consolidation.

The measurements at Mellösa are continuing, even if most of the original instrumentation has ceased to function and the routine measurements are therefore confined to total settlements and a few settlement markers at various depths that still appear to function.

The test field has also been used for a number of other investigations concerning the properties of soft clays. It was thus one of the sites used when the new vane apparatus was tried out (Cadling and Odenstad 1950). Later research concerning the vane shear test (Wiesel 1975), undrained shear strength and creep (Larsson 1977), quality of "undisturbed" samples (Larsson 1981) and permeability (Carlsten and Eskilson 1984) has been carried out at the Mellösa test field. Samples from Mellösa were also included in the study on the permeability of natural soft clays performed at Laval University (Tavenas et al 1983).

5.2 Soil profile

A generalized soil profile at Mellösa is shown in Fig. 29.
<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description</th>
<th>Water content (%)</th>
<th>Density (t/m³)</th>
<th>Organic content (%)</th>
<th>Undrained shear strength (kPa)</th>
<th>Effective vertical stress (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Topsoil Dry crust</td>
<td>20-40</td>
<td>1</td>
<td>1</td>
<td>5-10</td>
<td>10</td>
</tr>
<tr>
<td>2</td>
<td>Green organic clay</td>
<td>40-60</td>
<td>2</td>
<td>2</td>
<td>10-15</td>
<td>20</td>
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<tr>
<td>3</td>
<td>Dark grey organic clay, shells</td>
<td>60-80</td>
<td>3</td>
<td>3</td>
<td>15-20</td>
<td>30</td>
</tr>
<tr>
<td>4</td>
<td>Black organic clay, shells</td>
<td>80-100</td>
<td>4</td>
<td>4</td>
<td>20-25</td>
<td>40</td>
</tr>
<tr>
<td>5</td>
<td>Dark grey organic clay, shells</td>
<td>100-120</td>
<td>5</td>
<td>5</td>
<td>25-30</td>
<td>50</td>
</tr>
<tr>
<td>6</td>
<td>Grey clay</td>
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</tr>
<tr>
<td>12</td>
<td>Grey varved clay</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>13</td>
<td>Sand</td>
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<td></td>
</tr>
<tr>
<td>14</td>
<td>Rock</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fig. 29. Soil profile at Mellösa.
At the top, there is a layer of about 0.3 m of organic topsoil which was scraped off before the fills were constructed. The dry crust is unusually thin and consists of organic soil. The dessicated crust is limited to 0.5 m and underlain with soft clay. The clay has an organic content of about 5% just under the crust which decreases with depth and is less than 2% from 6-7 metres depth and downwards. The colour changes from green to black and becomes grey with depth. The black colour is the result of the presence of sulphides which between 2.5 and 6.5 metres amount to about 0.5% of the dry weight of the soil. The natural water content is about equal to the liquid limit and decreases from a maximum of about 130% to about 70% in the bottom layers. The bulk density increases from about 1.3 t/m³ to about 1.8 t/m³ at the bottom. The undrained shear strength has a minimum of about 8 kPa at 3 m depth and increases thereafter with depth.

The shear strength values were determined by vane tests in 1964 and 1967 and the values have been corrected according to the present SGI recommendation (Larsson et al 1984).

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

The pore water pressure in the ground outside the fills is hydrostatic with a ground water level about 0.8 m below the ground surface.

Preconsolidation pressures have been determined by Chang (1969, 1981) and by Larsson (1977, 1981). The step loaded tests performed by Chang in 1969 were corrected for measured ring friction. This friction was high and especially so in the stress intervals up to and around the preconsolidation pressure. The evaluated preconsolidation pressures from 24-hour curves corrected for friction were lower than the effective vertical in situ stress for large parts of the profile. CRS-tests and new incremental tests in connection with the investigations 1976, 1979 and 1981 have shown that the soil is overconsolidated in the upper two metres due to dry crust effects. In the organic high plastic clay between 2 and 6 m the soil is almost normally consolidated with an overconsolidation of only 2-3 kPa, while the clay in the lower part of the profile has an overconsolidation of about 12 kPa. Creep and swelling characteristics were studied in incremental oedometer tests.
in 1979 and the results have been incorporated in the empirical relations.

The permeability of the soil has been studied in the CRS-tests and in various field tests (Carlsten and Eskilson 1984).

5.3 The undrained test fill

In this study, only the undrained test fill at Mellösa is considered as vertical drains are outside the topic.

The undrained fill was constructed in October-November 1947.

Before the fill was placed 0.3 m of the loose organic topsoil was removed. A 2.5 m high fill of gravel with a density of 1.7 t/m³ was then constructed. The fill had bottom dimensions of 30 x 30 metres and slope 1:1.5. Time for construction was 25 days and no change in load other than natural variations has been made after that. The net load increase was calculated to 40.6 kPa.

A number of settlement markers and piezometers were installed at various depths before the fill was placed. The original markers in the clay have all ceased to function. Newer models have been installed which in turn have also mostly ceased to function with time. The settlement distribution with depth has thus mainly been evaluated from change in water content (Chang 1981). New piezometers were installed in 1968 and the pore pressures in 1979 were measured by retractable piezometers.

5.3.1 Initial settlements

The settlements during the loading period amounted to 0.065 m. The calculated elastic settlement based on empirical relations between undrained shear strength, plasticity index and calculated factor of safety is about 0.10 m. As some consolidation can be expected to have taken place during the 25 days the uploading phase lasted, the discrepancy between calculated and measured initial settlements is somewhat larger.
5.3.2 Consolidation

The computed "final" settlement when creep effects were disregarded was just below 1.40 m. This amount of settlement was reached in 1966. At that time, there were still remaining excess pore pressures in the order of 30 kPa indicating that almost no increase in effective stress had occurred in large parts of the profile. In 1979, the total settlement was 1.65 m and remaining pore pressures were over 20 kPa.

The consolidation settlements have been calculated with the CONMULT programme. They have been calculated both with and without creep effects. The division of the soil in the main layers and the key consolidation parameters are shown in Fig. 30.

![Fig. 30. Parameters used for calculation of consolidation at Mellösa.](image)

In the calculations, the lower boundary and the upper half metre of dessicated crust have been assumed to be free-draining. The load reduction due to settlements has been assumed to be 8 kPa per metre settlement. This value is an average as at first 0.5 m of saturated dry crust is submerged and then the actual fill starts to settle below the ground water level. It may be advocated that the weight of the fill will increase due to increasing moisture content above the ground
water level but this fill consists of gravel with a low capillarity. If the fill is assumed to be fully saturated also above the ground water level the average decrease in load due to settlements would still be 5 kPa per metre settlement. This would in 1979 entail a load reduction of about 8 kPa or 20% of the applied load. A load reduction of 8 kPa per metre settlement is judged as more realistic, though, and this corresponded in 1979 to a load reduction of 13 kPa or one third of the initially applied load.

The measured and calculated (initial plus consolidation) total settlements are shown in Fig. 31.

![Fig. 31. Measured and calculated total settlements for un-drained test fill at Mellösa.](image)
From Fig. 31 it can be seen that the calculated settlements, including creep effects, agree fairly well with the measured settlements, while the settlements calculated without creep effects are only about half of the measured values for most of the consolidation process. It can also be observed that in spite of the fact that more than 120% of the "final settlements" have occurred and that of the original 2.2 m only about 0.5 m is today elevated above the surrounding ground, there is no sign of a slowdown in the settlements versus log time.

A correct settlement prediction demands that not only the total settlements but also the distribution of settlements should be in accordance with what actually happens in situ. As previously mentioned, the settlement markers at various depths ceased to function after some time. Calculated relative compressions of different layers and corresponding measured values from markers are shown in Fig. 32. The markers at 4 and 7 metres were installed 1.5 years after the construction of the fill and may still be functioning. The other markers have probably become stuck with time and now follow the settlement of the fill and penetrate into the clay. This is a very common problem with markers of this type.

Another way to indirectly measure the settlement distribution at large settlements is to use the change in water content in different layers and thereby calculate the relative compression in the layers. This can only be done in cases where the lateral deformations can be assumed to be small in relation to the consolidation settlements. Such measurements were made by Chang (1981) for the conditions in 1967 and this settlement distribution is shown together with the settlement distribution measured by markers and the settlement distributions calculated with CONMULT in Fig. 33.

A comparison between the different settlement distributions shows fairly good agreement, but the measurements on the markers indicate larger deformations in the middle and bottom layers than the changes in water content and the calculated deformations. This discrepancy is further emphasized when the settlement distribution from measurements on markers in 1979 is compared to the calculated settlement distribution, Fig. 34. According to the measurements on the markers no settlements should have occurred in the upper five metres between 1967 and 1979 (see also Fig. 32) and this is hardly
Fig. 32. Relative compression in different layers. Calculated and measured by markers.
likely considering the pore pressure dissipation and the evolution of undrained shear strength.

In the prediction of the course of consolidation, not only settlements but also pore pressures should be predicted. The readings of the older types of piezometers were found to be extremely difficult to interpret. In the investigations in 1966-1969, new and more accurate piezometers were installed under the fills as well as in the ground well outside the influence of the fills. The pore pressures in natural ground were found to be hydrostatic for a ground water level about 0.8 m below the ground surface. Long term observations with durations from a few months to over a year did not reveal any fluctuations.

After these investigations, the new piezometers were withdrawn and no further observations were made until 1979. Then the pore pressure profiles were measured again, whereupon the piezometers were retracted.

The measured and computed excess pore pressures in 1968 and in 1979 are shown in Fig.s 35 a and b.
Fig. 34. Settlement distributions in 1979.

Fig. 35. Measured and calculated excess pore pressures in a) 1968 and b) 1979.
The agreement between pore pressures calculated with creep and the measured values is fairly good. The maximum measured values are somewhat higher than the computed values in spite of the fact that the measured settlements are somewhat larger than the calculated, which suggests that the creep effects may be a little larger than assumed. The differences are in both cases small.

In connection with the investigations in 1979, a few samples were taken under the undrained fill and a few oedometer tests were performed. The preconsolidation pressures are shown in Fig. 36.

![Diagram](image)

**Fig. 36.** Preconsolidation pressures and effective stresses under the undrained fill 1979.

The preconsolidation pressures measured in oedometer tests correspond well to those calculated with CONMULT. The calculated quasi preconsolidation pressures are the pressures on the original oedometer curves that correspond to the calculated deformations. The deformations have been calculated including creep effects.
Except at the drained boundaries, the effective stresses have in all parts of the profile been much lower than the "final" effective stress which would ultimately have been reached if unloading effects are disregarded. In fact, for most of the profile the maximum effective stresses that have acted in the profile hardly exceed the original preconsolidation pressure. In spite of this, the quasi preconsolidation pressures that have developed due to creep effects are considerable and for some parts even exceed the "final" effective stresses calculated without load reduction.

The course of consolidation and the development of stresses and changes of properties during this process are thus full of nuances. The calculated development of load and effective vertical stresses is shown in Fig. 37.

Directly after load application, the effective stresses become close to the preconsolidation pressures, except at the drained boundaries. The effective stresses increase only slowly in spite of the large settlements and even after 20 years there is a portion in the middle of the profile where the effective vertical stresses have not exceeded the original preconsolidation pressures. The pore pressure equalization and the convergence between the applied stress and effective vertical stress are largely due to the load reduction caused by the settlements and submergence of the upper masses. The "final" effective stress profile is shown in Fig 38. "Final" is in inverted commas as even at this stage there are creep settlements still going on and the effective stress will decrease somewhat with time. At this far distant stage, the load reduction due to settlements will amount to about half the applied load. In spite of this large load reduction, the settlements will exceed those predicted without load reduction if creep effects are disregarded in the assumptions. In fact, most of the loaded area will have sunk below the surrounding ground level.

The creep effects will have created quasi preconsolidation pressures which, in spite of the load reduction, will show a profile of preconsolidation pressures almost as if no load reduction had occurred.
Fig. 37. Development of load and effective stresses with time. Undrained fill at Mellösa.
The consolidation process is often checked by measurements of increase in undrained shear strength, especially at stage construction where this strength increase is to be utilized at the application of the next load step. This strength increase is often measured by vane shear tests in spite of the problems encountered in this special case. It has been shown by Law (1979, 1985) that the increase in undrained shear strength measured in vane tests is almost exclusively due to the increase in effective horizontal stress. In loading cases such as embankments with limited width and steep slopes, this is a serious drawback as the increase in horizontal stress and thereby the increase in undrained shear strength as measured by vane tests under the embankment is small. The vane shear test thus does not always reflect the relevant increase in active shear strength under the embankment. The wider the loaded area is, the more relevant the vane shear test becomes.
Vane shear tests were performed in two profiles near the centre of the undrained fill in 1967 and vane tests to a limited depth were performed in 1979.

The relation between undrained shear strength determined by vane shear tests corrected according to SGI 1984 and the preconsolidation pressure in normal ground \( (\tau_{fu}/\sigma'_C) \) varies between 0.2 for low plastic clays and 0.3 for high plastic organic clays. The increase in undrained shear strength and the calculated increase in preconsolidation pressure including creep effects is shown in Fig.s 39 a and b.

![Graph showing increase in undrained shear strength and preconsolidation pressure](image)

- ○ Calculated increase in \( \sigma'_C \)
- × Measured increase in \( \tau_{fu} \), corrected vane shear tests

**Fig. 39.** Increase in undrained shear strength as measured by vane shear tests corrected according to SGI and increase in calculated preconsolidation pressure under centre of undrained fill at Mellösa a) 1967 b) 1979.

As can be seen from the figure, the increase in undrained shear strength corresponds fairly well to the increase in preconsolidation pressure and is in this case of about the same order as for a very large loading area.
6. THE TEST FIELD AT SKÅ EDEBY

6.1 General

After the initial tests at Mellösä this site was disregarded for the new airfield. Construction was started at a place called Halmsjön (today Arlanda) in 1946. This construction was halted after a while as the new airfield was postponed.

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

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

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

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

The location of the test area was not quite ideal as the depth to firm bottom varied from 12 to 15 m for the fill locations. The area was selected bearing in mind that if an airfield was to be constructed the test area had to be out of the way. The layout of the test area is shown in Fig. 40.

Fig. 40. Main test areas and depth from ground surface to firm bottom in the test field at Skå Träddy.
Parallel to the following field observations, supplementary field investigations and an extensive laboratory investigation were carried out.

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

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

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

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

A large number of other research activities have been carried out at the test field at Skå Edeby. This was one of of the sites where the effects of various factors on the quality of clay samples was studied in connection with the standardization of piston sampling in Sweden.
(Kallstenius 1963). A new test fill has been constructed to compare the effects of sand drains and prefabricated drains (Torstensson 1976). Other test fills have been constructed to evaluate the effect of lime columns. The stabilizing effect of lime columns at excavations has also been tested (Boman and Broms 1975). At one of the old drained fills, a part of the fill has been removed and the sand drains partly excavated. Condition of the drains, distribution of water content and zones affected by the installation of the drains were studied (Holtz and Holm 1972). A large study of the effect of size of piston samplers on the quality of the samples was performed at Skå Edeby after Norwegian and Canadian tests had shown large size effects. No significant practical differences between the Swedish standard piston sampler and piston samplers with larger diameters were found, however (Holm and Holtz 1977).

The test field has also been used in research concerning the vane shear test (Wiesel 1975) and in testing various piezometers and the piezometer sounding method.

6.2 Soil profile

The soil under the test fills consists of soft clay with a thickness of 12 to 15 metres on top of till or rock. Also in other respects the conditions are not quite uniform. The water contents and liquid limits in the upper two metres vary between the different test locations, as does the level of a high plastic clay layer in the upper clay profile. These variations are, however, not much larger than the scatter of results within the separate test areas when different investigations for the test fields are collated. The same observation is valid also for strength and deformation characteristics, so that a general soil profile is fairly representative for the entire area. A geological profile and grain size distribution determined at the largest test fill is shown in Fig. 41.

The dessicated dry crust is only about half a metre thick. Below the crust, there is a layer of grey-green organic clay which in spite of the relatively low clay content is very high plastic due to the organic content. This layer is affected by the closeness to the ground surface and is thus overconsolidated with a relatively high shear strength and water contents lower than the liquid limits.
The underlying postglacial clay is slightly organic and high plastic. The postglacial clay, as well as the glacial clay beneath it, is coloured by or banded by iron sulphides. The contents of iron sulphide may be assumed to be of the same order as at Mellösa.

The glacial clay is varved. The varves are thin at the top but become thicker with depth. Near the bottom, occasional seams of silt and sand are found. Bedrock or dense till is found below the clay. Both can be considered as free draining. The geology of the site has been discussed in detail by Pusch (1970) who investigated the microstructure and chemistry of the clay. Some leaching of salts has occurred in the clay. The general properties of the clay are shown in Fig. 42.

The water contents in the clay are well above the liquid limit except for the upper two metres which are affected by dry crust effects. The water contents decrease from about 100% at the top to about 60% in the lower layers. The bulk density of the clay increases from 1.3 t/m³ at the top to about 1.7 t/m³ at 12 m depth.

The shear strength has been determined by vane shear tests, fall cone tests and unconfined compression tests. The averages of the results are in good agreement but the scatter in the results is unusually high.
<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description</th>
<th>Water contents, %</th>
<th>Undrained shear strength, kPa</th>
<th>Effective vertical stress, kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>50</td>
<td>100</td>
<td>100</td>
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<tr>
<td>1</td>
<td>Dry crust</td>
<td></td>
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<tr>
<td>2</td>
<td>Grey-green slightly organic clay, sulphide flecks</td>
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<td>3</td>
<td></td>
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<tr>
<td>4</td>
<td>Grey varved clay, sulphide flecks and bands</td>
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<td>5</td>
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<td>6</td>
<td>Grey-brown varved clay, sulphide bands</td>
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<td>8</td>
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<tr>
<td>9</td>
<td>Occasional sand and silt seams</td>
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<tr>
<td>10</td>
<td></td>
<td></td>
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<tr>
<td>11</td>
<td>Rock or till</td>
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<tr>
<td>12</td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

Fig. 42. General soil properties at the Skå Edeby test field.
for this type of clay. In the original stability calculations a cautious value of 5 kPa was used. The compiled shear strength measurements show, however, that a minimum shear strength of 6 kPa at a depth of 3.5 m and an increase with depth of 1.2 kPa per meter thereafter is a more realistic value. Still, the clay is very soft even for Swedish conditions.

The sensitivity was found to vary from about 5 in the upper layers to about 15 at the bottom in the early investigations. In the investigations in 1971, the sensitivity in the ground outside the fills was found to be about twice as high as in the early investigations, or about 10-30. Both sets of values are compatible with normal values for clays of this type and no obvious explanation for the discrepancy has been found as all other parameters from the different testing times agree quite closely.

The ground water level and pore pressure in the ground have been found to vary seasonally. The maximum variation seems to be ± 0.5 m. The pore pressure in natural ground can generally be assumed to be hydrostatic for a ground water level which can vary from the ground surface to 1 m below.

The preconsolidation pressures have been measured in incremental oedometer tests; originally by Hansbo (1961) then in 1971 by Holtz and Broms and again in a large number of tests by Holm and Holtz (1977). Finally, the preconsolidation pressures have been measured in CRS-tests in 1984 in connection with the present study. The results are unanimous in showing overconsolidation effects in and just under the dry crust, after which the soil becomes practically normally consolidated for a ground water level one metre below the ground surface.

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

The undrained test fill was constructed in June-July 1957. It was made up of gravel and given a bottom diameter of 35 metres and slopes 1:1.5. The density of the gravel fill was checked continuously and was 1.79 t/m³ as an average. The total height of the fill was 1.5 m and the imposed load intensity thus 27 kPa. No changes of load except for natural variations have been made after that.

6.3.1 Initial deformations

The settlements during the construction period were measured and found to be about 0.06 m. Calculated elastic settlements based on empirical relations between undrained shear strength, plasticity index and calculated factor of safety are about 0.03 m. As some consolidation can be assumed to have occurred during the month for load application the agreement between measured and calculated deformations is better than the numbers indicate.

6.3.2 Consolidation

The computed "final" settlements disregarding creep effects are about 0.75 m. This amount of settlement was obtained in 1972. At that time, there were still excess pore pressures in the order of 20 kPa. This means that fifteen years after load application there was practically no increase in effective vertical stress in large parts of the clay profile. Twenty-five years after the load application, the settlements amounted to 0.95 m and there were still excess pore pressures in the order of 12 kPa in the middle of the clay profile.

The distribution of settlements from the centre of the fill and outwards shows that the initial deformations were evenly spaced and decreased towards the edges of the fill. There were thus no excessive shear deformations at the edges. The distribution of settlements with time showed, however, that there was some effect of three-dimensional consolidation and horizontal waterflow. The maximum settlement rate thus occurred at a distance of about 10 metres from the centre of the fill where the load concentration is still high but the horizontal drainage path relatively short, Fig. 43.
Another observation was that there was a pronounced seasonal variation in settlement rate. These variations in themselves varied for different layers. The external factors causing the variations are varying ground water levels, varying water contents in the fill, varying temperatures, snow cover and freezing. As these factors affect different layers differently and sometimes work in opposite directions, there is no simple way to account for them.

The consolidation process for the centre of the fill has been calculated with the CONMULT programme. It has been calculated with as well as without creep effects. The division of the soil into main layers and the consolidation parameters from oedometer tests are shown in Fig. 44.
Fig. 44. Consolidation parameters for undrained circular fill at Skå Edeby.
In the calculations, the lower boundary and the upper metre of dessicated crust and clay with an abundance of root channels have been considered as free draining. The ground water level has been assumed to be stationary at one metre below the ground surface. Associated with this assumption, the load reduction due to settlements has been assumed to be 10 kPa per metre of settlement as it is saturated clay that becomes submerged for the first metre of settlement. Eventual changes in density of the gravel fill have been disregarded.

According to these assumptions, the initially applied load has been reduced by about one third due to the settlements after 25 years.

The measured settlements and the settlements calculated including creep effects are shown in Fig.s 45 a and b.

The total measured and calculated settlements agree well. The measured and calculated settlements for markers at various depths after 25 years agree well but there are some pronounced differences at the beginning of the consolidation process. Thus, the compression of the upper one and a half metres is initially much larger than predicted and remains so throughout the process. On the other hand, the early deformations in the bottom layers are much smaller than calculated, but they converge with time. As can be seen in Fig. 45 b, there is no sign of an upward turn of the time settlement curve in the semi log plot even after 25 years, when 125 per cent of the primary consolidation predicted in the classical way has occurred.

Measured settlements and settlements calculated without creep effects are shown in Fig 46 a and b. The correspondence between the predicted and calculated settlements is in all aspects rather poor. The "final settlements" have been calculated without regard to the consolidation process. Due to this process, the clay adjacent to the drainage boundaries will consolidate for the total imposed load while the clay at the middle distance between the drainage boundaries will consolidate for the applied load reduced by the total settlement effects. The reduction in the rest of the profile will have a hyperbolic distribution. In estimations of "final settlements" these differences in consolidation load are usually disregarded and a full reduction for load decrease due to settlements is applied throughout the profile. The error in terms of predicted settlements is small.
Fig. 6a and b. Measured and calculated settlements of undrained fill at Skå Edeby. The calculations include creep effects.
Fig. 46 a and b. Measured settlements and settlements calculated without regard to creep effects for the undrained circular fill at Shd Edeby.
The distribution of measured and calculated settlements with depth after 24 years is shown in Fig. 48. The agreement between the measured settlements and the settlements calculated including creep effects is very good at this point, but the poorer agreement in earlier stages as shown in Fig. 45 should be kept in mind.

The excess pore pressures after load application were reported by Hansbo (1960). They show that the pore pressure response to the load was almost identical to the increase in vertical stress, which confirms that the soil was almost normally consolidated, Fig. 47.

![Diagram showing observed excess water pressure distribution at a depth of 5 m below ground surface in Test Area No. IV one year after load application compared with distribution of theoretical stresses produced at the same depth by overloading. (From Hansbo, 1960.)](image)

Fig. 47 Observed excess water pressure distribution at a depth of 5 m below ground surface in Test Area No. IV one year after load application compared with distribution of theoretical stresses produced at the same depth by overloading. (From Hansbo, 1960.)

The pore pressures at the centre of the fill as well as in natural ground were carefully measured in 1971 and in 1982. The excess pore pressures at those times are shown together with calculated excess pore pressure profiles in Fig. 48.

The measured pore pressures agree fairly well with the calculations. There is no great difference in calculated pore pressures, whether creep effects are taken into account or not, and the measured values mainly fall between them.

New samples were taken under the fill in 1985 and oedometer tests were performed. The preconsolidation pressures are shown in Fig. 49.
The preconsolidation pressures measured in the oedometer correspond well to the quasi preconsolidation pressures calculated with CONMUL7, including creep effects. The latter preconsolidation pressures are the pressures on the original oedometer curve that correspond to the calculated deformations.

The effective stresses have in most parts of the profile only exceeded the preconsolidation pressure by a few kPa. In spite of this, there is a pronounced increase in measured preconsolidation pressures throughout the profile. In the upper parts, there are increases in preconsolidation pressure of over 40 kPa although the total load increase was only 27 kPa and unloading and remaining excess pore pressures have limited the real increase in effective stress in these parts to 10 to 15 kPa.

**Fig. 48.** Measured and calculated pore pressure profiles and settlement distribution under the undrained circular fill at Skå Edeby.
The permeability of the clay was also measured in the CRS-tests. The measured permeabilities and the permeabilities calculated with CONMULI, starting with initial permeability and taking calculated deformations into account, are shown in Fig. 50.

The measured and calculated permeabilities agree very well. The increase in undrained shear strength under the fill was checked by Holtz and Broms in 1971. As in the initial investigations there was a certain scatter in the results. The averaged shear strength profile with special reference to the results from vane shear tests corrected according to SGI 1984 is shown in Fig. 51, together with the general strength profile from 1957.
Fig. 50. Permeability under undrained circular fill at Skå Edeby.
The undrained shear strength had increased throughout the profile, and especially so at the bottom and in the upper parts of the profile, except for the crust. The increase in shear strength and the increase in preconsolidation pressure corresponding to the calculated deformations in 1971 are shown in Fig. 52. The deformations have been calculated with CONMULT and include creep effects.

Fig. 51. Undrained shear strength profiles under the undrained circular fill at Skädeby.
The scatter in the measured undrained shear strength should be kept in mind, but the figure shows that the profiles are very similar and that the increase in undrained shear strength is in the order of 0.2 times the increase in quasi preconsolidation pressure. The calculations with the CONMULT programme including creep effects have thus satisfactorily accounted for the long-term behaviour of the undrained circular fill at Skå Edeby. There is a discrepancy in the measured and calculated settlement distribution with depth in the early stages. Part of this discrepancy may be due to the varying ground water and climatic conditions, whose effect will even out with time, but to what degree it is not possible to estimate.

6.4 The test embankment

The test embankment was constructed in May 1961. At that time, the idea of locating an airfield at Skå Edeby had been abandoned but the results of the tests were considered very important, especially for road construction. The test embankment was therefore constructed to study the influence of geometry and lateral deformations. It was sponsored by the Swedish Road Administration and the Building Research Council.
The embankment was constructed with the surplus gravel from the partial unloading of the most heavily loaded circular fill. This is a rather narrow embankment with a crest width of 4 metres and slopes 1:1.5. The height of the embankment was 1.5 metres and the total length 40 metres. The density of the fill material is 1.8 t/m³ and the maximum load increase 27 kPa. The safety factor against failure was estimated to be about 1.5. The fill was instrumented with settlement markers and piezometers at different locations and depths under and outside the fill.

Flexible pipes for a newly constructed inclinometer measuring system were also installed at the toes of the embankment slopes, one on each side, and one pipe was installed further out from the embankment. Location of the instrumentation is shown in Fig. 53. The construction of the fill was made in stages during 22 days and thereafter no changes have been made.

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

Fig. 53. Mid section of embankment showing location of instrumentation. (From Holtz and Lindshög 1972.)
6.4.1 Initial settlements

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

Fig. 54. Vertical movements during the construction of the embankment of the test embankment at Skåledenby. (From Osterman and Lindskog 1963.)
An extrapolation of the calculated vertical settlements due to lateral displacements indicates that about 0.05 m of settlement is due to "initial" shear deformations directly after full load application. Calculated elastic settlements based on empirical relations between undrained shear strength plasticity index and calculated factor of safety are about 0.05 m.

6.4.2 Consolidation

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

Some horizontal movements are still in progress 25 years after construction but they are now barely detectable and the relative importance for the total settlements is steadily decreasing. The "final" consolidation settlement calculated in the ordinary way and disregarding creep effects is about 0.60 m (0.80 m if unloading due to settlements is disregarded). An earlier figure of 1.2 m and 1.5 m respectively referred to by Holtz and Lindskog (1972) must have been based on a more pessimistic assumption of the preconsolidation than further studies have shown, combined with a neglect of the load distribution with depth that occurs due to the narrow embankment and great depth to firm bottom. Even when correction for limited depth to firm bottom is applied, the load intensity as an average for the profile will still only be about half of the stress applied at the surface.

The calculated "final" settlements were reached in 1974 when corrections for the lateral deformations are made. The settlements are continuing and there is still no sign of an upward bend in the settlement - log time curve in spite of the decreasing influence of lateral deformations.

As for the undrained circular fill, a seasonal variation of the settlement rate was observed right from the beginning. Later slowdowns in the settlement rate have been observed for longer periods of a couple of years in 1968-1969 and in 1973-1974. Apart from these irregularities, which cannot be readily explained, the consolidation process has followed a smooth course.
The two functioning piezometers under the fill in 1971 showed pore pressures which indicated that the increase in effective stress was small for most of the profile.

The consolidation process, including the initial elastic deformations, has been calculated with the CONMULT programme. The process has been calculated with as well as without creep effects. As a certain influence of three dimensional water flow can be expected with this type of geometry also for the centre of the fill an additional calculation has been made with the programme GEOMEC (Runesson et al 1980) at Chalmers University. With this programme three dimensional waterflow can be accounted for, but not creep effects.

The division of the soil into main layers and the consolidation parameters are shown in Fig. 55.

The profile and parameters are identical to those for the undrained circular fill, except for the depth to firm bottom. In the same way, the upper metre of dessicated dry crust and clay with numerous root channels has been assumed as free draining. The lowest two metres of clay with increasing infusion of silt seams have in this case also been regarded as free draining.

The ground water level has been assumed to be stationary at one metre below the ground surface. Measurements with a piezometer outside the embankment located very near the firm bottom indicate that it has been at this level for about half the time between 1961 and 1971. For the rest of the time it has been higher and occasionally it has reached the ground surface.

Associated with the assumption about the ground water level, the load reduction due to settlements has been assumed to be 10 kPa per metre of settlement. According to these assumptions the initially applied load has been reduced by about one quarter during the first 20 years of consolidation.

The settlements calculated including creep effects are shown in Fig. 56 together with the measured settlements, as well as measured settlements corrected for measured time-dependent lateral deformations which have occurred after the loading was completed.
Fig. 55. Consolidation parameters for test embankment at Skå Edeby.
The calculated and the corrected measured settlements agree fairly well. As for the circular test fill, there are some discrepancies in the settlement distribution in the early stages, but these even out with time. It can be observed that the rate of vertical deformation in the field is 10-15% higher than the calculated rate, even when the field rate is corrected for lateral deformations. The settlements calculated without creep effects are shown together with the measured settlements in Fig. 57.

The consolidation settlements calculated without creep effects are less than half of the measured settlements, even if the latter are corrected for lateral deformations.

The consolidation process has also been calculated with the finite element programme GEOFEM C to investigate the effect of three-dimensional water flow. Care has been taken to give as similar input parameters as possible, but the computations are not directly
Fig. 57. Measured settlements and settlements calculated without creep effects for the test embankment at Skån Edeby. The calculated settlements include initial deformations.

calculable as the soil models differ and the GEOFEM C programme also allows for horizontal deformations. The calculations with GEOFEM C and CONMUL, both without creep effects, are shown as degree of consolidation in terms of settlement versus log time in Fig. 58.

As pointed out, the calculations are not directly comparable but it can be observed that, apart from initial discrepancies that follow from different methods of modelling the loading stage, the curves become very close with time. The consolidation process is somewhat faster as calculated with GEOFEM C, but the degree of consolidation never becomes more than about 10 per cent larger than that calculated with CONMUL. The discrepancy of more than 100 per cent between consolidation settlements calculated without creep effects and measured settlements can thus only to a minor part be explained by the effect of three-dimensional water flow. This effect seems more to be in the order of the discrepancy between calculated consolidation, including creep effects and the measured settlements.
The relatively small influence of three-dimensional water flow is of course only valid for the consolidation process at the centre of the fill. It can be explained when the distribution of settlement with depth in 1979 is studied, Fig. 59.

Measurements as well as calculations show that the bulk of the deformations have occurred within the upper five metres. The agreement between measured and calculated settlement distribution was very good at this stage.

The horizontal movements at the toes of the slopes and also some 4 m outside the embankment are shown in Fig. 60.
The maximum horizontal movements have occurred at a depth of 2 metres and amount to about 0.12 m in each direction. The relative maximum of the ongoing horizontal movements has been located further down with time. At present, the movements at 2 metres depth seem to have stopped and the maximum movements that occur now are located at a depth of 4 to 5 metres. The horizontal movements have also shown seasonal variations and, at the slowdown of settlements in 1968-1969 the inclinometer tubes seemed to move inwards in the upper 3-4 metres. This effect vanished though and, apart from these irregularities and variations associated with measuring accuracy, there has been a smooth decrease in the rate of horizontal deformations. The influence of the horizontal movement on the vertical settlements has steadily decreased and amounts at present to about 20 per cent of the total settlements, Fig. 61.
Fig. 60. Horizontal movements at the test embankment at Skå Edeby. Tubes H2 and H3 are located at the toes of the embankment slopes; one each side at the middle section. Tube H1 is located in the same section but about 4 m outside the embankment.
The present rate of horizontal movements is so small that the corresponding vertical deformations only amount to about 5% of the total rate of vertical deformations.

The initial pore pressure response to the applied load was measured by piezometers located at the centre of the embankment at depths of 2.5, 5 and 10 metres. The pore pressure response to the applied surface load of 27 kPa was about 20, 18 and 13 kPa respectively. The calculated stress increase at the same levels was 25, 20 and 13 kPa. This confirms the general picture of a practically normally consolidated soil profile with depth and an overconsolidation of a few kPa at 2.5 m depth. It should be noted, though, that there were large and sudden variations in the readings of all piezometers during the first years and the pressure in the reference piezometer located outside the loaded area and probably in contact with the draining bottom layers rose about 5 kPa during the loading phase. Part of this variation may have affected the readings further up in the profile. The long-term variations in the readings of the piezometers have all clearly shown a trend similar to the readings of this reference piezometer, Fig. 62.
Only two piezometers were still in apparent function in 1971. The excess pore pressures measured then are shown together with the calculated excess pore pressure profiles and the remaining increase in vertical stress in Fig. 63. If the pore pressure readings are correct, there seems to have been practically no increase in effective stress between 5 and 10 metres depth in the profile. At this time, about 90% of the "final" consolidation settlements calculated without creep effects had occurred.

Fig. 62. Applied surface load and pore pressure readings at the test embankment. (From Holtz and Lindskog, 1972.)
The division of settlements into vertical consolidation and vertical movements corresponding to horizontal deformations is not correct as these are one and the same process. The CONMULT program cannot take horizontal movements into account and the simple calculations of initial shear deformations do not consider time-dependent lateral deformations. There is thus a large discrepancy in calculated and measured settlements in the early stages of the consolidation process. There is no simple way to account for this and a correct prediction would require a finite element programme type GEOFEM C including creep effects. However, the discrepancies decrease with time as the influence of horizontal movements decreases and the calculations with the CONMULT programme including creep effects give a fairly good prediction of the long-term behaviour at the centreline of the embankment. The influence of horizontal movements in this case is unusually large due to the narrow embankment and the low factor of safety against undrained failure.
7. DALARÖVÄGEN - EMBANKMENT ON PEAT

7.1 General

The road Dalarövägen connects the urban district of Jordbro with highway 73 to Stockholm. The road was built in 1979-1981 and was designed as a motorway with a crest width of 24 m. For 850 m the road is located on a peat bog. The peat layers are 2-3 m thick.

In the early stages of planning, the possibility of constructing the embankment on top of the peat instead of excavating the peat and replacing it with other masses was discussed. A test embankment was constructed (Lindskog and Nordstrand 1978) and based on the results the motorway was built on top of the peat.

The road was designed with surcharge in order to quickly obtain most of the settlements in the peat and underlying compressible layers and on the assumption that an unloading large enough to halt further settlements could be made. The surcharge was intended to remain for one year. It was dimensioned on the basis of results from the test fill and the requirement that the unloading should be at least 0.5 m of fill. Before the embankment was constructed, the soil was instrumented in three sections within the peat bog and the behaviour of the embankment has been studied during and after construction.

The instrumentation comprised settlement markers at various depths and locations, horizontal flexible tubes for measurements with hose settlements gauge, inclinometer tubes, horizontal tubes with external loose magnetic rings for measuring horizontal surface movements and piezometers.

The observations showed that the settlements became larger than predicted and the surcharge was increased after one year. The increased surcharge was applied for a further half year, whereupon it was removed.

In connection with this and other projects concerning construction on peat, a new peat sampler was constructed. The new sampler was not ready until about a year after the constructions at Jordbro had started and thus the only good quality samples of the peat are those taken in section 1/360 and outside the loaded area. The peat samples had
a diameter of 0.1 m and a length of 0.8 m. Selected specimens with a height of 45 mm were tested in incremental loading tests in compressiometers, with a diameter of 100 mm. In the compressiometers, the samples are placed between two highly permeable filter stones and are surrounded by rubber membranes on the sides. The diameter is kept constant by thin rings evenly spaced on the outside of the rubber membranes. The results from the tests and measurements have been reported in detail by Carlsten (1985).

7.2 Soil profile

The surface layer with vegetation is about a quarter of a metre thick and is stiffer than the underlying peat. The upper 2-3 m of the soil profile consists of fibrous peat. The degree of humification varies between H2 and H4 and increases somewhat with depth. Under the peat there is a thin layer of about 0.1 m of gyttja and organic clay on top of a 0.5 to 2.0 m thick layer of sand.

Below the sand, there is another compressible layer of clay and silt which is about 3 m thick. This layer is slightly overconsolidated. The observed total settlements in this layer amounted to 0.2 m.

No instruments were placed in the clay, however, and only the behaviour of the soil down to the sand layer has been studied in detail. The ground water level is near the ground surface and the density of the organic soil is close to unity. The shear strength of the peat was measured by field vane tests and the corrected undrained shear strengths were around 5 kPa. The relevance of field vane tests in this type of material is doubtful, though.

The total width of the embankment was 45 m. The thickness of the peat layer in section 1/360 varied between 2.1 and 2.8 metres across this distance. The most detailed measurements have been taken at a point located 8 m from the centreline and the profile deduced from the samples taken on both sides of the embankment is shown in Fig. 64.
7.3 Construction of the embankment

Construction of the embankment started in April 1979 when a first load of 21 kPa was applied. The load was checked by measurements of the thickness and density of the fill. After about 50 days, when almost all excess pore pressures had disappeared, another 34 kPa were added. The embankment had a crest width of 24 metres and the slopes were made flat with an inclination of about 1:3 to provide stability and with a desire to minimize horizontal movements. The maximum width at the base of the embankment was about 45 m.

The density of the fill was measured as 2.1 t/m³ in the dry state and the saturated density was 2.3 t/m³. At the beginning of June 1980, the surcharge was increased by another 10.5 kPa and the unloading of the surcharge took place at the end of November the same year. The unloading amounted to about 23 kPa.

The events described above are a simplification, as construction meant that a transport road had to be kept open all the time. This transport road had to be moved to different locations across the road during construction and the detailed loading history is much more complicated than the simplified main process described.

Fig. 64. Soil profile at Dalarövägen. Section 1/360, Left 8 m.
7.4 Horizontal movements

The measurements of horizontal movements showed that the central parts of the embankment within the crest width have moved practically vertically. The horizontal surface movements under the slopes increase toward the toes. The maximum horizontal movements were about 0.15 m and occurred under the slope at a distance of about one quarter of the width of the slope from the toe.

7.5 Consolidation

The total settlements just before unloading amounted in section 1/360 Left 8 m to 1.2 m, of which 1.0 m had occurred in the peat. The settlements in the peat just before the additional surcharge were about 0.94 m.

The corresponding calculated "final" settlements in the peat disregarding creep effects were 1.10 m and 1.03 m respectively. The predicted settlements were thus about 10% larger than the actually measured settlements. Measured and predicted excess pore pressures were small on both occasions.

The consolidation process at the point considered has been calculated with the CONMULT programme with and without creep effects. The division of the soil in the main layers and the key consolidation parameters are shown in Fig. 65. The parameters were evaluated by Carlsten in 1985.

![Fig. 65: Parameters used for calculation of consolidation at Dalarövägen.](image-url)
In the calculations, the sand under the peat layer and the surface layer have been assumed to be free draining. The load reduction due to settlements has, in accordance with the density measurements, been assumed to be 8 kPa per metre of settlement. The measured and calculated settlements for the peat layer are shown in Fig. 66.

![Compression of 2.4 m of peat](image)

**Fig. 66.** Measured and calculated settlements for the peat layer at Dalarövägen. Section 1/360, Left 8 m.

As already mentioned, the calculations without creep overpredicted the "final" settlements by about 10%. The introduction of creep effects naturally did nothing to improve this discrepancy. Instead, the difference in this respect increased to about 16%. On the other hand, the shape and general trend of the curves show better agreement when the creep effects are included. This is especially valid for the ongoing creep deformations at the end of the load steps with increasing load. A similar effect would in this case have been obtained if the simple approach of adding creep deformations after full pore pressure equalization had been used.

Both predictions are quite acceptable, considering the type of material and the fact that the consolidation parameters are extrapolated from samples taken some distance away. The most striking result of the
calculations is how little influence the creep effects in this case have on the early consolidation process. The calculated pore pressures also became practically identical, whether creep effects were included or not. The peat in question with a high compressibility, high initial permeability and moderate creep effects may be considered as an extreme case for soils in general, but this type of soil is common in Sweden. Another feature to observe in this type of soil is the $\beta_k$ value of about 6 ($\beta_k = \Delta \log k / \Delta \varepsilon$). This means that in the actual case the permeability decreased by about 1000 times during the consolidation.

The pore pressures were measured at various times during the construction of the embankment. The excess pore pressures measured at the middle of the highly compressible peat layer are shown together with the calculated excess pore pressures in Fig. 67. The measured and calculated pore pressure developments are in close agreement.

Fig. 67. Applied load and excess pore pressures at Dalarövägen. Section 1/360, Ligt 8 m.
The undrained shear strength increased considerably during the consolidation process. The corrected vane shear strength before loading was in the order of 5 kPa.

After 94 days when the designed surcharge was fully applied but there still were excess pore pressures in the order of 20 kPa, the corrected vane shear strengths had increased to about 15 kPa. Two years after the start of construction, when the soil had consolidated for the additional surcharge and then been unloaded, the corrected vane shear strength was in the order of 26 kPa.

In both the loaded cases, this corresponds to an undrained shear strength about half of the preconsolidation pressure. The scatter is considerable, though, and no further conclusions can be drawn, especially in view of the uncertainty about the relevance of the vane shear test in peat.

The settlement observations have continued and after the small heave at unloading, the total settlements during the following 4 years after paving and opening for traffic have been about 0.01 m.

The course of the consolidation in the peat has thus been successfully calculated with the CONMULT-programme. The main advantage with the programme in this case is that it takes variations of modulus and permeability into consideration. The permeability of the soil decreased by about 1000 times during consolidation while the variation in modulus was less than 10 times. The classical \( c_v \) value (\( c_v = \frac{k \cdot M}{9 \cdot p_w} \)) thus did not just drop at the preconsolidation pressure but decreased more than 100 times further during the consolidation process.
8. ANTONINY SITE, BIALOSLIWIE - STAGE CONSTRUCTED EMBANKMENT ON ORGANIC CALCAREOUS SOIL IN POLAND

8.1 General

Discussions on a joint research project between The Agricultural University of Warsaw and The Swedish Geotechnical Institute concerning the construction of embankments and dykes on organic soils started in 1981.

Several test embankments had previously been constructed by the Department of Geotechnics at the Agricultural University of Warsaw (DG). No less than 10 test embankments were built in the period 1976-1982. The embankments were built on various deposits with 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 have been reported by Professor Wolski and his co-workers in internal reports, doctoral theses and conference papers, e.g. Fürstenberg et al (1983), Szymanski et al (1983) and Lechowicz et al (1984).

The aim of the cooperation between DG and SGI was to combine the resources and experience of DG in construction of embankments on organic soils and the experience and capabilities of SGI in investigation and instrumentation in very soft soils.

Two embankments have been constructed at the Antoniny site near Bialosliwie in north-western Poland. The embankments have a base width of 35 m and a length of 50 m. Vertical prefabricated drains have been installed under one of the embankments. The embankments have been constructed in stages in order to achieve an increase in shear strength and ensure stability.

Both embankments are instrumented with settlement markers, horizontal tubes for measurement of settlements with hose settlement gauge, inclinometer tubes, magnetic settlement gauges for measurement of settlement distribution with depth and piezometers. Prior to the instrumentation, a field investigation program was carried out. This program involved soil sampling with Swedish standard 50 mm piston sampler as well as sampling with a piston sampler taking 60 mm samples. In the upper peat layer, the new peat sampler with 100 mm was also used.
A comprehensive series of vane shear tests was carried out. These comprised tests with varying sizes of the vane blades and varying rates of rotation. There were no obvious size effects on the results and the rate effects were similar to those normally observed in Scandinavian soils (e.g. Larsson et al. 1984).

Some cone penetration tests and pore pressure soundings were also made. The shear strength of the soil is very low, though, and the standard cone penetration equipment is not sensitive enough for this type of soil. The results from the cone penetration tests generally confirmed the results from the sampling and vane shear tests, but did not give any additional information. The pore pressure soundings gave negative excess pore pressures in the peat and positive but rather low excess pore pressures in the calcareous soil. It was later found that the soil had a very high overconsolidation ratio which gives relatively low excess pore pressures in this type of sounding (Jamiolkowski et al. 1985).

The first sampling operations were made by DG in 1982 and the samples were tested in the laboratories at DG and SGI. More samples were taken in the summer of 1983 in connection with the field tests and the instrumentation. The samples were brought to SGI for further tests.

The results from the field and laboratory tests have been reported in detail, together with previous experience in preliminary reports (DG 1983 and SGI 1984).

The first load stage of the embankments was applied in November 1983. New field tests and a thorough check of the instrumentation were made from the end of March to the beginning of April 1984 and the next stage was started directly thereafter. A new series of field tests including sampling and instrument check was carried out in May 1985, whereupon the third load stage was applied. New load stages are planned, including further loading of one embankment and partial unloading of the other. Parallel to these tests, the long-term deterioration of prefabricated drains in organic soil is being studied.

The follow-up of the course of the consolidation process during the first stage was not quite successful in every aspect, due to unfamiliarity with the equipment and insufficient instruction. This problem
was soon overcome and further observations have been performed very satisfactorily.

The results of the measurements have been reported by DG in annual reports in 1984 and 1985.

8.2 Soil profile

The test site is situated near the Notec river, where a large project with construction of dykes is being carried out. The soft soil consists mainly of sediments with a very high content of calcium carbonates. The upper two metres consist 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 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 there is about 5%. At about 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.

The soil properties determined in routine laboratory investigations are shown in Table 2. The undrained shear strength of the soil was measured in a large number of vane shear tests and by fall cone tests during the routine investigations in the laboratory.

Series of active and passive triaxial tests and direct simple shear tests were run on samples from two levels. The undrained shear strength profile is shown in Fig. 68.

The relation between undrained shear strength determined in direct simple shear tests and preconsolidation pressure was about 0.33 for samples from 1.5 as well as 5.8 m depth.

The preconsolidation pressures and compression characteristics have been determined in incremental oedometer tests as well as CRS-tests. A large number of tests have been performed and samples taken with different types of samplers have been tested.
Table 2. Soil properties at Antony, Bielskowicze.

<table>
<thead>
<tr>
<th>Depth m</th>
<th>Soil</th>
<th>Density t/m³</th>
<th>Water content %</th>
<th>Liquid limit %</th>
<th>Plastic limit %</th>
<th>Plasticity index %</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>11.6(5.8¹)</td>
</tr>
<tr>
<td>3.0</td>
<td>Dark brown very calciferous dy-bearing gyttja</td>
<td>1.19</td>
<td>227</td>
<td>221</td>
<td>17</td>
<td>12</td>
<td>17 (8.5¹)</td>
</tr>
<tr>
<td>4.0</td>
<td>Yellowwhite calcareous soil (marl)</td>
<td>1.31</td>
<td>134</td>
<td>129</td>
<td>16</td>
<td>12</td>
<td>16 (9.8¹)</td>
</tr>
<tr>
<td>5.0</td>
<td>Grey calcareous soil</td>
<td>1.38</td>
<td>109</td>
<td>96</td>
<td>10</td>
<td>13</td>
<td>10 (7.0¹)</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>12.9(8.7¹)</td>
</tr>
<tr>
<td>6.0</td>
<td>Grey calcareous soil</td>
<td>1.41</td>
<td>108</td>
<td>97</td>
<td>12</td>
<td>13</td>
<td>12 (8.3¹)</td>
</tr>
<tr>
<td>7.0</td>
<td>Green gray calcareous gyttja</td>
<td>1.33</td>
<td>148</td>
<td>138</td>
<td>12</td>
<td>11</td>
<td>12 (7.1¹)</td>
</tr>
</tbody>
</table>

Samples from 1.47-1.64 m and 5.67-5.84 m are taken with Borro ®60 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.
There was no obvious difference in evaluated preconsolidation pressure in the different tests or the different samples. 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 than the other samples. The average results from the oedometer tests are shown in Table 3.

The preconsolidation pressures showed an unusual preconsolidation profile, Fig. 69. 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.
Table 3. 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>$\sigma_C^t$ kPa</th>
<th>M_L kPa</th>
<th>$\sigma_I^t$ kPa</th>
<th>M' kPa</th>
<th>a kPa</th>
<th>k m/s</th>
<th>$\beta_k$</th>
<th>$c_v$ m²/s</th>
<th>$\alpha_S$ %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.47-1.64</td>
<td>CRS</td>
<td>φ50</td>
<td>5</td>
<td>14.8</td>
<td>122</td>
<td>31.8</td>
<td>6.8</td>
<td>13.9</td>
<td>1.7·10⁻⁸</td>
<td>4.2</td>
<td>10⁻⁵⁻¹⁰⁻⁷</td>
<td>2.6-2.0</td>
</tr>
<tr>
<td>1.47-1.64</td>
<td>Step</td>
<td>φ60</td>
<td>3</td>
<td>=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>φ100</td>
<td>3</td>
<td>=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>φ50</td>
<td>5</td>
<td>15</td>
<td>120</td>
<td>28</td>
<td>8.1</td>
<td>13.0</td>
<td>1.7·10⁻⁸</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>φ50</td>
<td>3</td>
<td>17</td>
<td>240</td>
<td>65</td>
<td>7.3</td>
<td>32</td>
<td>4·10⁻⁹</td>
<td>3.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.0</td>
<td>CRS</td>
<td>φ50</td>
<td>2</td>
<td>23</td>
<td>382</td>
<td>50</td>
<td>8.9</td>
<td>7</td>
<td>4·10⁻¹⁰</td>
<td>2.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.0</td>
<td>CRS</td>
<td>φ50</td>
<td>3</td>
<td>21</td>
<td>260</td>
<td>39</td>
<td>10.5</td>
<td>14</td>
<td>5·10⁻¹⁰</td>
<td>2.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.67-5.84</td>
<td>CRS</td>
<td>φ60</td>
<td>5</td>
<td>19.4</td>
<td>288</td>
<td>41.2</td>
<td>11.7</td>
<td>16.6</td>
<td>8.3·10⁻¹⁰</td>
<td>2.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.67-5.84</td>
<td>Step</td>
<td>φ60</td>
<td>3</td>
<td>=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>φ50</td>
<td>5</td>
<td>19.8</td>
<td>320</td>
<td>44</td>
<td>11.9</td>
<td>17</td>
<td>8.8·10⁻¹⁰</td>
<td>2.2</td>
<td>5·10⁻⁹⁻²·10⁻¹⁰</td>
<td>2.0-1.4</td>
</tr>
<tr>
<td>7.0</td>
<td>CRS</td>
<td>φ50</td>
<td>2</td>
<td>23</td>
<td>145</td>
<td>40</td>
<td>9.3</td>
<td>24</td>
<td>1.6·10⁻⁹</td>
<td>3.4</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
The preconsolidation profile indicated a non-hydrostatic pore pressure. There was much concern about the quality of the "undisturbed" samples obtained in this type of soil, which is why different samplers were used. 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.

After the piezometers had been installed and stabilized and a standpipe had been driven into the sand layer, it became clear that an unusual stress situation was at hand. The water pressure in the sand layer below 7.8 m depth proved to be artesian with a water head 1.5 m above the ground surface. With some fluctuations this situation has prevailed during the whole of the observed period. Measurements of the pore pressure in the sand have shown that the pressure has varied.
seasonally, and for a large part of the time the artesian pressure head has been about half a metre lower than the initial values. At the start of measurements the ground water level at the top was slightly below the ground surface, which has been the normal situation, but the area has been seasonally flooded. The piezometer readings showed a pore pressure distribution such that the effective stresses in the natural ground are very low and in most of the profile amount only to about 2 kPa, Fig. 70.

These low effective stresses entail that the soil has a high overconsolidation ratio. If the stress history of the site is such that the deposit has consolidated for higher effective stresses and these have later been reduced by the artesian water pressure, then the low recompression moduli measured in the oedometer tests become quite plausible. Recompression moduli calculated from overconsolidation ratio and swelling characteristics become of the same order as the measured values.

The compression characteristics measured in the oedometer tests can thus be expected to be fairly representative for the soil in situ.
8.3 The undrained test embankment

As consolidation with vertical drains is outside the scope of this study only the embankment without drains is considered here.

The embankment was constructed with the base dimensions 35 x 50 m. The slopes were made with inclination 1:3. The fill material is sand with a unit weight of 1.8 t/m³ at a natural water content of about 10%.

Before construction, the soil below and outside the embankment was instrumented. The layout of the fill and the position of the instruments are shown in Fig. 71.

Construction began on 7th November 1983, when 1.2 m of sand was applied. This operation took about five days. The first stage lasted for five months and the next load of another 1.3 m of sand was applied between 11th and 18th April 1984. The second stage lasted about thirteen months. In the third stage another 1.4 m of sand was applied between 25th May and 12th June 1985. The third stage is still continuing.

The stages were chosen with consideration to stability and finally decided when the shear strengths at the end of each stage had been determined. The calculated factor of safety for stage 1 ranged from about 2 to 3 with the lowest factors for local stability of the outer parts of the embankment. The calculated factors of safety against undrained failure in stage 2 and 3 were both about 1.2.

8.3.1 Initial deformations

Due to problems with proper handling of the instrumentation the course of the consolidation was not followed in detail in stage 1. The inclinometer readings in stage 2 and 3 showed, however, that the majority of the horizontal movements occurred during uploading. The total horizontal movements during stage 1 correspond to a vertical movement of about 0.12 m and the initial deformations should be somewhat less.
Fig. 71. Layout of test embankment and position of instrumentation. Antoniny site, Bialos³wie.
The settlements during uploading in stage 2 were measured by the hose settlement gauge and varied in the central part between 0.15 and 0.20 m. The horizontal movements measured 9 days after uploading corresponded to a vertical movement of about 0.18 m.

In stage 3, the settlements during uploading were measured with the hose settlement gauge as 0.08-0.10 m in the central parts. The settlements corresponding to the horizontal deformations measured three months after the load application were about 0.09 m.

The initial settlements calculated with empirical relations between undrained shear strength, plasticity and safety factor against undrained shear failure were 0.10 m, 0.17 m and 0.13 m for the three stages. No correction for overconsolidation ratio has been made in stage 1. Had a correction for overconsolidation similar to that proposed by Foott and Ladd (1981) been applied, the calculated initial settlements in stage 1 would have been over 0.3 m.

The " measured" and calculated initial deformation are summarized in Table 4.

Table 4. "Measured" and calculated initial deformations.

<table>
<thead>
<tr>
<th>Stage</th>
<th>&quot;Measured&quot; by hose settlement gauge</th>
<th>Calculated from horizontal movements</th>
<th>Calculated empirical</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-</td>
<td>&lt; 0.11</td>
<td>0.10</td>
</tr>
<tr>
<td>2</td>
<td>0.15-0.20</td>
<td>&lt; 0.18</td>
<td>0.17</td>
</tr>
<tr>
<td>3</td>
<td>0.08-0.10</td>
<td>&lt; 0.09</td>
<td>0.13</td>
</tr>
</tbody>
</table>

The values measured by hose settlement gauge include consolidation settlements occurring during the time for load application so that the initial deformations are somewhat smaller.
The total calculated initial settlements in the three stages amount to 0.40 m. The measurements indicate that the calculated values are in the right order of size, but the initial settlements seem in all three stages to have been somewhat smaller than calculated. The initial settlements are in the order of 20% of the total settlements.

---

**Fig. 72.** Horizontal distribution of total settlements. Antimony site, Białostowie. Embankments without drains.
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 remained even if the maximum has moved inwards during stage 2 and 3.

The same is valid for the embankment with vertical drains so that the distribution cannot be related to a greater rate of consolidation towards the toes due to horizontal water flow. The horizontal distribution of settlements is shown in Fig. 72.

8.3.2 Consolidation

The embankment has been built in stages with relatively short durations. Consequently, no full consolidation has been reached in any stage. The total "final" settlements disregarding creep settlements are calculated to be about 0.5 m, 1.3 m and 1.8 m for the three stages. At the end of stage 1, however, there were remaining excess pore pressures in the order of 10 kPa. At the end of stage 2 there were excess pore pressures in the order of 20 kPa and half a year after application of stage 3 the excess pore pressures were about 30 kPa. The total settlements on these occasions were 0.4 m, -1.1 m and 1.5 m respectively.

The consolidation settlements have been calculated with the CONMULT programme with and without creep effects. The division of soil into main layers follows the oedometer tests. The division and the key consolidation parameters are shown in Fig. 73.

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 to the results from the oedometer tests.

In the calculations, the upper one and a half metres of peat with vegetation, cracks and root threads have been considered as free draining. This assumption was made first after the piezometers at 2 m depth had
Fig. 73. Parameters used for calculation of consolidation at Antonig site, Bialoslawie.
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 homogenous amorphous peat had shown that the permeability of the homogenous 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 entered 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 metre of settlement. Here, this corresponds to a load reduction of 10-15% of the total applied load at the end of stage 1 and 2 and at the last observation in stage 3.

The measured and calculated (initial plus consolidation) total settlements are shown in Fig. 74.

The inclinometer readings showed that the horizontal movements practically stopped shortly after the loading phase and the initial "elastic" deformations are somewhat overpredicted. Therefore, no reduction has been made of the measured vertical deformations due to time-dependent lateral deformations during consolidations.

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 a rather low influence on the magnitude of the settlements. 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 including 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 settlement has been measured by special magnetic markers. The course of the settlements with time has been studied only from stage 2 and onwards. Fig. 75 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.

![Graph showing settlement increase over time](image)

**Fig. 75. Increase of settlements in stage 2 and 3, "Undrained" embankment at Antony site, Bialoslawie.**

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. 76.

The horizontal deformations have been measured by inclinometers. The inclinometer readings show that lateral movements of about the same magnitude have occurred throughout the soil profile. The horizontal extension of the movements has decreased as the areas with increased loads have become narrower, Fig. 77.
The maximum horizontal movements were 0.12 m at the end of stage 1 and 0.35 m at the end of stage 2. At the last reading in stage 3, the maximum horizontal movement amounted to 0.46 m. After large horizontal movements during uploading and shortly afterwards, the rate of horizontal deformations has rapidly decreased. Except for the early phases in each stage, the horizontal movements play a very small role in the course of consolidation. The maximal horizontal movements are shown versus the total settlements in Fig. 78.
Fig. 77. Horizontal movements. "Undrained" embankment, Antoniny site, Bialoslwicie.
The pore pressures under the embankment have been measured by piezometers type BAT. In this type of piezometer, the filter tip is placed at the end of an open pipe. The water pressure acts on a membrane above the filter. When a reading is taken, a pick up device containing a pressure transducer is lowered down to the membrane. A syringe needle leading to the transducer then penetrates the membrane and the reading is taken. There are always problems with piezometers having rigid connections to the ground when the differential settlements are large. In the present case they are exceptionally large. To prevent pushing of the piezometers, the piezometer pipes were encased for most of their length in plastic tubes but how well this has worked is uncertain. Some of the initial pore pressure responses during uploading were distinctly larger than the applied stress at the position of the piezometers. These elevated excess pore pressures disappeared rapidly, indicating that they were only local effects around the piezometers.
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. 79. This point was located approximately in the middle of the layers with low permeability.

![Graph showing measured and calculated excess pore pressures](image)

**Fig. 79.** Measured and calculated excess pore pressures at an original depth of 4.5 m below the ground surface. Centre of "undrained" embankment, Antoniny site, Bialosliwie.

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. However, there are obvious signs of pushing of the piezometer at the load application. This effect rapidly vanishes but there is at present some doubt about the exact location of the piezometer. The possible error does not affect the comparison very greatly for this piezometer, though.
The third piezometer in the profile was originally located only 1.3 m from the sand at the bottom. This piezometer has shown very large signs of pushing and until its present location is established no conclusion can be drawn from the readings.

At the end of stage 2, 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. 80.
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 middle part of the layers with low permeability, the effective stresses had not even reached the original preconsolidation pressures. 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.

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. 80.

The undrained shear strength was measured by field vane tests at the end of stage 1 and stage 2. The measured strength values were corrected according to the SGI recommendation (Larsson et al 1984). The corrected shear strengths are shown together with the original shear strength profile (see Fig. 68) in Fig. 81.

At the end of stage 1, there was an increase in the shear strength above 4 m original depth. Below this level, no increase in shear strength was measured. The shear strength increased further in stage 2 and in this stage there was also an increase in the lowest part of the profile. No increase in shear strength was measured in the part of the profile where the preconsolidation pressures were unchanged. The changes in undrained shear strength closely follow the changes in preconsolidation pressure.
In most of the profile, there seems to be an almost constant relation between corrected vane shear strength and preconsolidation pressure $\tau_f u / \sigma'_c = 0.37$. The only exception is the lowest metre of the profile where the corresponding relation is 0.28. Preconsolidation pressure here means original preconsolidation pressure, as well as measured and calculated quasi preconsolidation pressures including creep effects.

The relation between undrained shear strength and preconsolidation pressure according to direct simple shear tests on samples from 1.5 m and 5.8 m depth was 0.33.
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 in the right order of size but somewhat too large. According to the results no correction for overconsolidation should be applied to the calculated deformations based on vane shear strength.
9. DRAMMEN

The settlements of a large number of buildings in the town Drammen have been studied by the Norwegian Geotechnical Institute. This study is almost unique, as it concerns settlements in cases where the stresses are lower than or just around the preconsolidation pressure.

The geological history of the soil deposits in Drammen has been documented and the deformation properties of the soil have been thoroughly investigated. The undrained shear strengths were investigated by field vane tests and were found to be directly related to the preconsolidation pressures in normally consolidated as well as slightly overconsolidated soils. The investigations in Drammen led to the general model for development of creep settlements, quasi preconsolidation pressures and undrained shear strength due to "ageing" presented by Bjerrum in the seventh Rankine Lecture 1967. This model was further developed by Bjerrum in 1972. More detailed information on some of the buildings and slightly revised soil properties were presented by Foss in 1969.

The results from a laboratory study on the course of consolidation in oedometer specimens with different heights were presented by Berre and Iversen (1972). This investigation showed that the course of consolidation considerably deviated from the Terzaghi consolidation theory and that different stress-strain paths were followed in different parts of the specimens due to creep effects. A numerical method of calculating the settlements including creep effects in a single uniformly loaded layer was presented by Garlanger (1972). The method was based on Bjerrum's model of 1967.

The observed buildings are located in an area of Drammen where the soil conditions are described as fairly uniform. The general soil conditions consist mainly of five different layers of soil. The upper layer is a 2.5 m thick deposit of fine sand which is underlain by a silty clay having a thickness of about 2.5 m. Below the silty clay, there is a 5.5 m thick deposit of high plastic postglacial clay. This layer has a natural water content of 50-55% and liquid limit of 50-65%. The main interest in the investigations has been focussed on this layer as it is the most compressible layer and the layer exhibiting the largest creep effects. The overconsolidation ratio in this layer was 1.6 according to Bjerrum (1967). Below the high plastic clay, there
is a layer described by Bjerrum as a thin layer of sand and by Foss (1969) as a transition zone containing numerous shells, small pebbles and pockets or thin layers of sand.

The transition zone is about half a metre thick and below that there is a low plastic clay layer about 20-25 m thick. Soil layers of glacial origin are located at a depth of about 35 m. The soil profile given by Foss (1969) is shown in Fig. 82.

The settlements of six buildings, of which one had a lighter and a heavier part, were studied in this area. About sixty per cent of the settlements are stated to have occurred in the high plastic clay layer. The origin of this figure is unclear as the presented measurements are made on points on the buildings. The settlements became far larger than predicted for most of the buildings and using the relation between initial stress, preconsolidation pressure and applied load Bjerrum (1967) concluded that only about 50% of the overconsolidation effect (the preconsolidation pressure minus the overburden pressure) could be utilized without causing undesirably large creep settlements. With an overconsolidation ratio of 1.6, this would correspond to the mobilization of about 80% of the preconsolidation pressure.
A closer study of the data presented by Bjerrum shows that the overconsolidation ratios under the different buildings varied between 1.25 and 1.6. The preconsolidation pressures have also later been revised by Foss (1969) who showed that the overconsolidation ratio of 1.6 in three of the cases should rather be 1.5. In Fig. 83, the total settlements after 10 years for each building have been plotted versus the average mobilized degree of the preconsolidation pressure in the high plastic layer. Data have been taken from Bjerrum (1967) and have been modified according to Foss (1969). The result is still that it seems as though 80% of the preconsolidation pressure can be utilized without any larger creep deformations.

Fig. 83. 10 year settlements of buildings in Drammen versus stress ratio.
There are considerable differences between the separate buildings also in other aspects than overconsolidation ratio. The thicknesses of the soil layers vary at the different locations. The depth of the excavations varies and so do the geometries of the bases of the buildings. This affects not only the load distribution in the high plastic layer, but also the stress situation in the low plastic clay. This is not unimportant because, although this clay is less compressible, it has a lower overconsolidation ratio (OCR = 1.15) and considerable thickness. Because of the thickness of the layer and the low overconsolidation ratio, the creep effects in this layer should not be disregarded either, even if the creep parameters are numerically small.

A further aspect of importance is whether the transition layer between the high plastic and the low plastic clay layers should be considered as draining or not. According to Foss (1969) this zone is not believed to provide any drainage. This belief is substantiated by some field observation of pore pressure dissipation within the area. On the other hand, Bjerrum (1967) reports that the pore pressure dissipation was relatively rapid and that piezometers installed at two locations after 10 and 30 years respectively showed that all excess pore pressures had disappeared. These observations, coupled with the observed settlements at the locations and the permeabilities reported by Berre and Iversen (1972), are not quite compatible with the assumption of no drainage in the transition layer.

Furthermore, two of the buildings are situated so near each other and surrounding buildings that some interference can be expected (Foss 1969). The same is valid for the two parts of the same building with different load intensities.

The interpretation of the settlements of the buildings in Drammen is thus complex and it would be presumptuous to make a claim of accuracy for a calculation based on the published data.

However, the observations in Drammen are virtually unique and the general soil profile and type of loading serve very well to illustrate some aspects of the course of consolidation at loads in this stress region.
Some fictive calculations have therefore been made with a soil profile as given for Konnerudsgate 16 (Fig. 82) and a geometry for the loading as for the actual building there.

The compression parameters have been converted from the $C_{c}/1+e_0$ values given to average modulus within the stress interval of interest. Permeability and coefficient of secondary consolidation for the high plastic clay have been taken from Berre and Iversen (1972). Corresponding values for the rest of the profile have been estimated from empirical relations. The division of the soil profile into main layers and their consolidation parameters are shown in Fig. 84.

![Fig. 84. Consolidation parameters used in calculation of settlements in Drammen.](image)

Prior to construction of the buildings, excavations were made and the weights of the buildings were thus partly compensated. In the fictive calculations, an excavation as at Konnundsgate 16 has been made, so that the upper sand layer is reduced in thickness. The applied loads are loads in excess of the weight of the excavated masses.

The base dimensions of the actual building were 58 metres by 13 metres and these have been used in the calculation. The time for construction was about seven months.
The consolidation process has been calculated with the CONMULT programme for various applied loads corresponding to certain average stress levels in the high plastic clay layer. As can be seen in Fig. 82, the stress level varies considerably in different parts of the profile. An average relative stress level \((\sigma_0 + \Delta\sigma) / \sigma_C\) of 1.0 in the high plastic clay in reality entails a) that the preconsolidation pressure is exceeded in the silty clay, the upper part of the high plastic clay and the upper part of the low plastic clay b) that the stress is just at the preconsolidation pressure in the middle of the high plastic clay, in the transition layer and in part of the low plastic clay and c) that the stresses are lower than the preconsolidation pressure in the lower parts of both the high plastic and low plastic clay layers.

The consolidation process has been calculated with and without creep effects and for the cases of full drainage as well as no drainage in the transition layer.

The settlements calculated without creep effects are shown in Fig. 85. The time-settlement curves are quite different depending on whether drainage in the transition layer is assumed or not. The stress levels 1.0 and 1.1 correspond to the cases where piezometers were installed 10 years and 30 years after construction. In both cases, excess pore pressures of up to about 5 kPa are calculated in the case of no drainage, even when no creep effects are assumed.

The settlements calculated with creep effects are shown in Fig. 86. The settlements are in all cases considerably larger than the settlements calculated without creep effects. This is in relative terms valid also for the stress level 0.8 where the calculated settlements including creep after 20 years amount to 0.06-0.07 m, compared to 0.04 m without creep. The practical significance of such a difference would be small, though. There is also a considerable difference in the calculated time-settlement curves, depending on whether the transition layer is assumed to provide drainage or not. After 30 years, almost all differences in total settlements due to the drainage conditions in this layer had evened out in the case of no creep. When creep effects are considered, very large differences remain and the curves calculated for different drainage condition have at this time barely started to converge. According to these calculations, the drainage
conditions are very important also for the development of creep deformations. While the concept of stress-strain relations coupled only to time for sustained load may be very useful in geological aspects involving several thousands of years, the drainage conditions have to be considered when the behaviour of constructions in normal time aspects are in question.

The calculated excess pore pressures in the case of no drainage were about 6 kPa in the high plastic clay at the times when the piezometers were installed. No excess pore pressures were calculated at the same times if drainage was assumed in the transition layer. In this context, it should be observed that the actual settlements on the two occasions were about two times the settlements calculated with creep and assuming no drainage in the transition layer.
Fig. 86. Calculated total settlements including creep versus time.
A comparison between the total settlements calculated with and without creep effects is made in Fig. 87. The settlements have been calculated after 27 years and full drainage is assumed in the transition layer. The creep effects increase the calculated settlements from just under two times at a stress level of 0.8 to nearly three times at a load factor of 1.1. This difference increases with increasing time. The same difference would have been obtained in the case of no drainage in the transition layer, but after a longer period of time.

The shape of the stress level - total settlement curve and the effect of creep even at low stress levels is explained by the uneven stress levels in the profile already mentioned. The calculated settlements in the main layers above and under the transition zone are shown in Fig. 88. In the low plastic clay, the overconsolidation ratio was 1.15
to start with. This means that the real stress level here was over 0.8 and creep should occur at any load increase. Already at an average stress level of 0.9 in the high plastic clay, the preconsolidation pressure is exceeded in part of the low plastic clay layer. No significant creep deformations are calculated in the high plastic clay at stress levels lower than 0.8, but from this level they increase with increasing stresses. The preconsolidation pressure is exceeded
in the upper part of the high plastic clay layer at an average stress level of 0.98. Due to the uneven stress levels the total settlement - average stress level curve becomes more evenly rounded than if the stress levels had been more equal in all parts of the profile. The creep deformations at low stress levels all occur in the low plastic clay and are due to the low overconsolidation ratio in this layer. The settlements including creep effects are largest in the low plastic clay layer until the preconsolidation pressure is reached in the overlying soil. At higher stress levels, the deformations in the high plastic clay rapidly increase with increasing stresses and at a stress level of 1.1 they approach 60% of the total settlements.

The settlements calculated with creep and full drainage in the sandy transition zone are compared to the measured settlements in Drammen reported by Bjerrum (1967) in Fig. 89.

The stress levels for the measured values have been estimated from the figures given by Bjerrum (1967) and Foss (1969). No direct comparison should be made, due to the approximative nature of the stress levels and the fictitious nature of the calculations. It can only be observed that a certain similarity exists between the time-settlement curves thus calculated and the measured curves.

According to the calculations, a very large part of the creep deformations at stress-levels below the preconsolidation pressure in the upper layers occur in the low plastic clay. The coefficients of secondary consolidation assumed in these layers are very low and only amount to 15-25% of the values in the high plastic clay. The creep deformations still become relatively large due to the thickness of the low plastic clay layer and the low overconsolidation ratio. The settlements in this layer at lower stress levels become relatively large also because the preconsolidation pressure is exceeded in parts of the layer already at low stress increases.

The original presentation of the settlement records from Drammen when the behaviour was related solely to the stress level in the high plastic clay layer may thus be slightly misleading. The concept that creep deformations start at a stress level of about 80% of the preconsolidation pressure and thereafter increase with increasing stress level is not affected by the calculations, however.
The calculations have shown that also in the stress region below or just around the preconsolidation pressure the permeability and drainage conditions play dominant roles for the development of creep deformations.

Fig. 89. Measured and calculated time-settlement curves for buildings in Drammen.
10. DISCUSSION AND CONCLUSIONS

10.1 Creep effects

The tests and measurements for the different loading cases have shown that the behaviour of the soil in the field largely follows the pattern postulated by Bjerrum (1967, 1972). Creep deformations occur within the process of pore pressure dissipation and afterwards and the compressibility is thus time-dependent. Quasi preconsolidation pressures develop due to the creep effects and the increase in shear strength follows this increase in quasi preconsolidation pressure rather than the applied vertical stress. The concept of relating the creep effects to time for sustained load, however, is too coarse, except for very long geological time aspects concerning many thousands of years. In the normal engineering time aspect, the creep processes are to a large extent governed by the permeability and drainage conditions of the soil. This condition was mentioned both by Bjerrum (1972) and by Berre and Iversen (1972) and also mathematically treated by Garlanger (1972) but may not be apparent from the usual presentation of Bjerrum's model.

The calculations of the soil behaviour with the soil model and the multilayer programme described have in all aspects qualitatively predicted actual field behaviour. The calculations have, however, been performed at or close to the centre of the loaded areas only and the calculation method is applicable to full-scale field cases alone.

The correlation between the oedometer tests and field behaviour is made with consideration to the large differences in the rates of compression occurring in the oedometer as compared to the actual field cases. It is also based on empirical observations of the pore pressure development at full-scale loading in the field.

The stress-strain relation has been found to be highly time/strain rate dependent also in the small-scale oedometer tests with their relatively high rate of compression. In fact, the shape of the time settlement curves from incremental tests and the results from continuous oedometer tests with different rates of strain indicate that the stress-strain relation is continuously time-dependent. What is classically called "primary consolidation" may thus solely be, or is at least partly, the effect of the hydrodynamic delay of a continuously time-dependent compression process. A continuous time-dependent
stress-strain relation normalized against the strain rate has been suggested by Leroueil et al (1985). A similar function would be obtained if the model described here was extended to rates faster than the reference rates. The interpretation of the separate processes occurring within the oedometer tests is somewhat speculative and highly debated, though. The introduction of the creep processes occurring at fast rates of compression may thus at present involve more problems and uncertainties than increase in accuracy in the prediction of soil performance in situ. The omission of these processes may affect the predictions in the very earliest stages of the consolidation process. The presented model is thus just a practical way to adapt the results from small-scale oedometer tests to field conditions, and is especially suited to predicting the long-term behaviour of the soil.

10.2 Relevance of the oedometer case

The calculations have been made at or close to the centre of the loaded areas. If points further out from the centre are considered, a number of aspects would have to be accounted for. The closer the boundary for the loaded area is approached, the more the loading condition deviates from the oedometer case. The shear stresses increase and the principal stresses rotate. This affects the development of excess pore pressure, which has to be accounted for. For points where the principal stress-axes remain unchanged, this can be done using the normally determined yield surface. For points where the principal stresses rotate, an hypothesis such as that described by Larsson and Sällfors (1981) and exemplified by Larsson (1983 and 1984) must be used. There is so far no extensive verification from field measurements for such hypotheses, though. The deviation from the oedometer case entails horizontal deformations, as well as a change in compressibility (Larsson 1981). The closer to the boundary for the loaded area the point of consideration is located, the greater the effect of horizontal water flow becomes. Accurate predictions of the consolidation process close to the boundaries of the loaded area would thus require a finite element program which could take into account creep as well as horizontal water flow and a soil model which accounts for anisotropy and the change in soil compressibility and pore pressure response occurring when the shear stresses increase and the principal stresses rotate. Such a program should, of course, also account for the initial
deformations and for the fact that the uploading phase is normally partly drained.

10.3 Relevance of small-scale oedometer tests

The calculations for the test fills and embankments have been based on a large number of tests and the soil profiles have been divided into large numbers of layers with relevant soil parameters. Possible errors in test results and the effect of unrepresentative samples have in this way been evened out. The quality of the calculations can be expected to be related to the extent of the soil investigation and routine investigations are usually not as comprehensive as in the presented cases. The problem that small specimens are not always representative for the soil mass is thus usually larger than appears in the presented results.

10.4 Assumptions on drainage conditions

The calculations are performed with the measured properties and with normal assumptions about drainage conditions, except for the stage-loaded embankment. In the stage loaded case, the original predictions gave a much slower rate of deformation than the actual field rate. A new assumption about the drainage conditions was made after the results from the first stage had been obtained, and the agreement between the predictions and the field performance improved considerably for this and the following stages.

In the same way, the predictions of long-term behaviour of actual field cases may be improved by a follow-up of field behaviour, including pore pressure measurements of the early stages of the consolidation process, mainly in order to establish the drainage conditions. Modifications of the initial assumptions for drainage in the other cases have been omitted. Such modifications could not have been made beforehand and would thus be misleading as to the ability to correctly predict the course of consolidation in advance. Neither have any modifications of the measured values or the empirical correlations been made. There are so many factors involved in the calculations that almost perfect fits to any behaviour could afterwards be obtained with
the introduction of a number of so called "reasonable assumptions". Such calculations are of very limited value, though.

10.5 Sources of errors

10.5.1 Sample disturbance

Discrepancies between predicted and observed settlements are often attributed to sample disturbance and errors in the tests. Sample disturbance is often a problem in sensitive and brittle soils and especially in clays with infusions or layers of silt or coarser material. This problem has not been very pronounced in the actual cases and the quality of the samples has in most cases been checked and found to be good. There are, however, more and probably greater sources of error in the calculations.

10.5.2 Applied load

The intensity of the applied load is not always certain. In fills the density of the material is somewhat uncertain. The degree of saturation may vary with time and also periodically. The fill is seasonally covered by snow and will with time often be covered by some kind of vegetation. Road embankments are subjected to adjustments and repavings. For buildings, the estimation of the live load is often made from a general pattern and is not exact. The decrease in load due to settlements is somewhat uncertain and may vary with the ground water level in the upper layer.

The description of the sequence for load application is also often a very coarse approximation and the exact procedure is often complex and dependent on factors arising during construction.

10.5.3 Stress distribution

The stress distribution with depth is another source of error. Different theories for stress distribution give slightly different results. Furthermore, the load itself is described as rigid or totally without stiffness, neither of which is usually quite true. The varying stiffnesses in the different layers should ideally be accounted for,
as well as the limited depth to firm bottom. Even a smaller error in the stress distribution might involve the difference between exceeding the preconsolidation pressure in a highly compressible layer or not. Another problem is how to account for the load distribution occurring in the lower parts of a stage constructed embankment when new stages are applied.

10.5.4 Ground water conditions

Calculation of the course of consolidation is also usually made with the assumption of a constant ground water regime and permeabilities that change only with compression. Long-term measurements of ground water levels and pore pressures have shown that fluctuations are a rule rather than an exception. These fluctuations have been shown to affect not only the crust and draining layers but also penetrate far into the clay layers (e.g. Berntsson 1983). Changes of temperature in the ground create volume changes and pore pressure changes in the affected soil masses. The affected zone is usually limited to a few metres in the upper part of the profile. In this part, however, the permeability of the soil is also affected and if freezing should occur in the upper drainage paths the effect might be very pronounced.

Seasonal variations in settlement rate were observed at the Skå Edeby test field and have also been found in a number of other settlement observations in Sweden.

Moreover, the determination of the pore pressure profile is often very coarse. To determine the pore pressure is as important as to determine the preconsolidation pressure and this can be done very accurately. The pore pressure profile has in many cases been shown to deviate strongly from the hydrostatic water pressure. Still, many settlement calculations are carried out with the pore pressures estimated from an observation of the free water level in a borehole or the crust and with the assumption of hydrostatic water pressures further down in the profile. In many cases in Sweden, it has been possible to express the preconsolidation profile in the way that "the soil is normally consolidated for a ground water level so many metres below the ground surface" and this ground water regime has then often been assumed to exist without further measurements. This situation must be improved and this will hopefully come automatically with the wider use of piezocones in soil investigations.
10.5.5 Drainage conditions

Another large uncertainty is the drainage condition. This may be a minor problem in the thick homogenous clay deposits with only a thin crust, where also horizontal water flow can be relatively easily accounted for. Most soil profiles are more complicated, though. The fissured crusts may be thick and root threads and channels may penetrate several metres down in the clay. To estimate the drainage conditions in this part of the profile and how they will change with compression is very difficult. Further down, any amount of infusions or layers may occur and to estimate their extension and discharge capacity may be difficult and expensive. Even in the homogenous clay profiles, there is usually a transition zone in the bottom part with a gradual increase in the frequency and thickness of coarser layers and to estimate a level where free drainage should be assumed may not be so easy.

10.6 Sensitivity of calculations to changes in input data

Some fictive calculations have been performed to obtain an idea of the sensitivity of the results to changes in input data. It has then been found that the results are very sensitive for the assumptions on the drainage conditions. This has been demonstrated here by the calculations for the special cases in Drammen, but is valid generally.

On the other hand, the calculated courses of consolidation are normally not very sensitive for errors in the creep parameters. In the model used, the creep parameters are very important, but the extra settlement due to creep is still only a part of the total settlement. The consolidation process is also partly self-regulating in the way that creep effects larger than those required to keep the pore pressure at a certain level will create a decrease in the effective stresses which in turn will slow down the creep process. This is illustrated in Fig. 90, where the results of fictive calculations of settlements in a thick clay deposit are shown. The coefficient of secondary consolidation has been varied from zero to 1.5, 2.3 and 3.1%/log cycle of time.

As can be seen from the figure, there is a large difference whether creep is assumed or not, but the curves with different assumptions about the creep parameters are very close. It is first in the later stages of the process, when the excess pore pressure is no longer kept
up, that the curves start to deviate and differences in the creep parameters start to show. The effect of these differences becomes largest close to the drainage boundaries. For most calculations, the accuracy of the creep parameters obtained from the empirical relations would be quite sufficient.

Fig. 90. Results of calculations with varying creep parameters.
The settlements including creep effects become larger and faster than if creep is disregarded. How this agrees with previous experience from observed and calculated settlements is very difficult to evaluate. Real long-term observations are scarce and of those existing, very few include observations of pore pressures, settlement distribution or horizontal displacements. Much of the Swedish "experience" emanates from cases where the soil has simply been assumed to be normally consolidated.

Some attempts have been made to summarize old "experience" but it has not been possible to draw any conclusions. The conditions in the different cases have varied too much, the pore pressure situations have been uncertain, calculations of load and stress distribution have usually been made in some simplified way and there has been much room for subjective interpretations of the investigations and test results. Also in other aspects, there has been a wide scope for assumptions. Assumptions of high permeabilities in the horizontal direction and large shear deformations have, for instance, often been made without any supporting measurements. On the other hand, there has hardly been any questioning of the validity of Terzaghi's consolidation theory.

Bjerrum (1973) found that the agreement between old predictions of settlements in Norway and the corresponding observations became relatively good, provided that the settlements were large enough. This was in a region where a typical overconsolidation ratio of about 1.5 has been found. In the old predictions, the overconsolidation effects had been disregarded and the settlements had been made with assumptions of normally consolidated soils. Two calculations of the course of consolidation in a thick clay layer are shown in Fig. 91. One calculation is made for a normally consolidated profile without creep effects and where the applied load causes approximately a doubling of the vertical stresses. In the other calculation, the soil exhibits normal creep effects and has an overconsolidation corresponding to half of the extra load. All other parameters are identical.

As appears from the calculations, it is quite possible to obtain almost identical time - total settlement curves if both overconsolidation and creep effects are disregarded or if they are accounted for. This type of self-cancelling error was pointed out by Tavenas and Leroueil
in 1981. Similar results would be obtained in more normally consolidated soils if both load distribution with depth and creep were disregarded. Observed agreement between predictions and total settlement may thus be seriously questioned unless all aspects of the behaviour are measured and accounted for.

Several field observations of settlements have been made during recent years in connection with road constructions in Sweden. The investigations have been more extensive than older investigations usually were and modern test techniques with more objective evaluation have
been used. No "final" settlements have been measured but the general observation is that the settlements that have occurred during the times for observation have been faster and larger than what was predicted from calculations where creep effects were disregarded.

10.8 How come that not all clays are overconsolidated due to creep effects?

Another question that arises is why not all soils are overconsolidated to some degree. In Sweden, many soft clays are referred to as normally consolidated. One reason for the apparently normally consolidated state is to be found in the properties of these soils. They are normally relatively high plastic with high compressibility and low permeability, so that the large creep deformations required to create quasi preconsolidation pressures require a long time to develop. The reason is also to be sought in stress history. Owing to the ongoing isostatic land heave in this region, there is in many areas a very slow but steady increase in effective stress in the soil as the ground water level is lowered. Because of the changes in ground water conditions there is also a slow leaching of salts from the soil and this and other chemical changes may break down some of the preconsolidation effects.

It should also be considered that there are no totally normally consolidated profiles. The dry crust and a considerable layer below this are usually overconsolidated. It is also common that the bottom parts of the profiles show overconsolidation effects when the lower boundary provides drainage. It is thus usually the centre parts of the clay profiles with long drainage paths that are "normally" consolidated. The term "normally consolidated" means that the preconsolidation pressure corresponds to an effective stress at a certain ground water condition. This ground water condition is usually a ground water level at some depth below the ground surface and a hydrostatic water pressure below that level. The ground water conditions vary, however, and the pore pressures are seasonally much higher than in the normally consolidated state. Pore pressures as low as at the normally consolidated state may not even have been actually measured in all cases. The "normally consolidated soil" may thus for most or part of the time be slightly overconsolidated. An increase of the pore pressure to the hydrostatic pressure from a ground water level in the ground surface would, in the cases of Skå Edeby and Lilla Mellösa, increase the overconsolidation ratio to 1.5 at the top of
the "normally" consolidated part of the profile. This effect would gradually decrease downwards. Apart from putting the relevance of the overconsolidation ratio in question, the fluctuation in ground water level also has the effect of periodically halting the creep process and generally slowing it down.

That there are periodical variations in ground level and a very slow long-term settlement also in natural ground in the soft clay areas in Sweden was found already in 1918 by Virgin.
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