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Behaviour of Organic Clay and Gyttja

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Results from Investigations in Swedish Gyttja-Bearing Soils Supplemented with Results from a Similar Finnish Investigation and Experience from Sulphide-Rich Soils (svartmocka)

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PREFACE

This report mainly deals with the results from a large laboratory investigation on the behaviour of fine-grained soils in the transition zone between mineral soils and organic soils, i.e. organic mineral soils and mineral organic soils such as organic clay and clayey gyttja. Some results from investigations on sulphide-bearing soils, which may or may not be organic but which by tradition have been grouped among the organic soils, have also been included.

The investigation was carried out at the Swedish Geotechnical Institute in 1982-1985 with support from the Swedish Council for Building Research and the Swedish Road Administration. Many of the results have been reported and implemented in Swedish practice as they have been obtained. Examples of reports where the results from this investigation have been reported or form a part of the experimental basis are:

- Evaluation of shear strength in cohesive soils with special reference to Swedish practice and experience.(Larsson, Bergdahl and Eriksson 1984. SGI Information No. 3. Shorter version also in ASTM Geotechnical Testing Journal Vol.10, No. 3, 1987.)

- Determination of organic content, carbonate content and sulphide content in soil.(Larsson, Nilsson and Rogbeck 1985. SGI Report No. 27)

- Automatic continuous consolidation testing in Sweden. (Larsson and Sällfors 1985. ASTM Special Technical Publication STP 892.)

- Consolidation of soft soils. (Larsson 1986. SGI Report No. 29)

- Dilatometer tests for determination of stratigraphy and properties in soil.(Larsson 1989, SGI Information No. 10.) (In Swedish)

The results have also been implemented in manuals and standards for laboratory testing in Sweden.

The purpose of this report is to present more of the basic data, to describe the gradual change in behaviour at the transition from a mineral to an organic soil and to present certain data and results that have not been reported elsewhere.

Prior to this project, a similar investigation was carried out at the Geotechnical Laboratory at the Technical Research Centre of Finland, (Slunga 1983). The results from this investigation are used to supplement the Swedish results and the author is indebted to his Finnish colleagues for the valuable information obtained at the start of the current project. Assistance has also been obtained from colleagues at Chalmers University of Technology who provided most of the test results from the Välen site.

Linköping in June 1990

Rolf Larsson

CONTENTS

1. SUMMARY AND CONCLUSIONS

An investigation has been carried out into the properties of finegrained soils containing organic matter and their variation with the composition of the soil. Also the applicability of current testing techniques to this type of soil has been investigated.

New testing techniques for determination of organic contents, carbonate contents and sulphide contents have been introduced, which enables a more relevant and accurate classification to be made.

Numerous CRS-oedometer test series with varying rates of deformation for the tests in the series have shown that the standard testing and interpretation procedure for inorganic clays may be used also for soils with organic content. The normally used standard rate of deformation, 0.0024 mm/min, is however a maximum rate for soils with organic contents which must not be exceeded and it is prudent to use a somewhat lower rate.

Provided that the standard rate is not exceeded, a unique stress-strain curve can be evaluated independent of the rate of strain in the test. Furthermore, this evaluated stress-strain curve coincides with the stress-strain curve obtained in standard incremental tests with doubled load increments and 24 hour duration for each increment. Also the evaluated preconsolidation pressures become the same when the appropriate evaluation methods for the two types of tests are used.

The rate effects as expressed by the coefficient of secondary consolidation increase in the transition zone between mineral and organic soils with increasing organic contents and the associated increase in water content. For even more organic soils, this trend becomes broken and in peat the rate effects seem to be related more to the degree of humification.

When using the results from the oedometer tests for settlement calculation, these rate effects have to be taken into account and a new calculation method which enables this has been introduced.

The permeability of organic fine-grained soils is lower than in corresponding inorganic soils. Combined with the high compressibility, this entails that the rate of consolidation as expressed by the coefficient of consolidation is in the order of one decade lower than corresponding values for inorganic soft clays. This fact and the high creep effects must be considered in all other strength and deformation testing of the soil.

A significant effect of the preferred horizontal orientation of the undecomposed plant remains can be observed on the permeability of the soil. The horizontal permeability in clayey gyttja is thus typically about 2 times the vertical permeability. In peats, where the permeability is usually much higher, this anisotropy depends on the degree of humification. It is high in undecomposed peat, but decreases to become insignificant when the soil is completely decomposed.

The coefficient of earth pressure at rest in the normally consolidated state, K_{onc}, has been evaluated from oedometer tests and triaxial tests and has been found to vary between 0.5 and 0.6 for all the investigated soils. The observed trend is that K_{onc} slowly decreases with increasing organic content. The organic contents in the investigated soils varied from 3-4% to about 30%.

Triaxial tests showed that in undrained tests the stress-paths within the yield surface were approximately vertical in a q-p'stress space. This means that initial pore pressure changes at an undrained stress change can be calculated in the same way as for inorganic clays from the assumption that within the yield surface the isotropic mean effective stress, p', remains constant.

The failure mode, however, differs. Failure in an active compression test on a soil with a significant content of organic fibres occurs when the increasing pore pressure has brought down the effective horizontal stresses close to zero. Failure then mainly occurs as vertical cracking of the sample. Correspondingly, failure in a passive extension test occurs when the effective vertical stress approaches zero and the sample is cut off in a horizontal plane.

Failure in undrained tests on organic soils occur at fairly large deformations. As the volume during the test is constant, this implies that there are large tensile strains in the minor stress direction. The cracking of the sample can therefore, as explained by Janbu (1979), partly be considered as tensile strain cracks, which are not necessarily associated with tensile stresses. An evaluation using Mohr-Coulomb criteria is not appropriate as the theory does not allow for this type of failure.

The results from oedometer tests indicate that a proper testing rate in undrained strength tests should be ten or more times slower than the standard rate of strain used for testing of inorganic clays. This procedure would be very time consuming. The combined results from the Swedish and the Finnish investigations show that the reduction factor to be applied to test results obtained at the normal rate of deformation in order to make them correspond to a one decade slower rate is 0.9 and the minimum correction factor to be used for extremely slow rates is around 0.85. These values seem to be fairly constant.

Direct simple shear tests are shown to be the type of test that gives the most relevant value of the undrained shear strength. Performing this type of test on soils with extremely low preconsolidation pressures is associated with special problems. In the relatively simple apparatus normally used in Sweden, the minimum vertical stress is about 20 kPa and direct normalization of the undrained shear strength against the preconsolidation pressure is not quite applicable for this type of soil. Several tests with varying preconsolidation pressures may therefore have to be performed in order to enable extrapolation of the results to the actual stress level in the ground.

All testing and especially normalization and comparisons are unusually difficult, as it is very small numbers that are compared and divided, and variations and inaccuracies make the relative scatter rather large. Even so, all the normalized results show that for a given preconsolidation pressure the undrained shear strength in all loading modes becomes higher when the organic content increases and the organic matter to a large extent consists of fibrous matter. This increase starts at an organic content around 2 % and continues non-linearly up to an organic content of about 20 %, where it evens out. The results generally confirm the limits for organic contents used in the Swedish classification system.

The deformations at failure in triaxial tests increase with increasing organic content and liquid limit. The deformations at failure are very similar in active and passive tests and no significant peaks are obtained in slow tests.

The sensitivities of soils with an organic content follow the general pattern in that they increase with increasing liquidity index. In comparison with normal values for inorganic soft clays with corresponding liquidity index, they are generally lower.

Comparisons between strength values obtained in field vane tests and fall cone tests and the results of the laboratory tests and, in those cases where such data are available, large scale field tests show that the recommended correction factors are generally applicable. The procedure described in SGI Information No. 3 to estimate if the results are normal and if the correction factors can be assumed to be applicable has to be followed, however. In some cases with obviously abnormal strength values, the deduced correction factors deviated strongly from the general values.

The results from investigations in "svartmocka" show that the real strength of the material is quite normal for soils with corresponding liquid limits and organic contents free from sulphides. The main difference is that both the field vane test and the fall cone test seem frequently to yield abnormally high strength values, which correspondingly have to be reduced much more than the general correction specifies. The reason for this is not known, but special care should be taken when these methods are used in "svartmocka".

2. INTRODUCTION

Fine-grained soils with organic content are common in Sweden. Even the glacial clays often contain minor amounts of organic material. This content gradually increases in the younger post-glacial clays as the biological activity increased during the period of their deposition. The upper layers in clay deposits in Sweden therefore often contain significant amounts of organic matter. In many areas, the content is so high that the soils are classified as organic clay or gyttja. In other areas where lakes were overgrown and peat bogs were formed, the peats often lie on top of deposits of gyttja and organic clay.

"Organic soils" or "soils with an organic content" has often been a concept with various meanings in geotechnical engineering and the rules for division into different groups have often been rather diffuse. The rules for division of the soils into organic soils, mineral organic soils, organic mineral soils and mineral soils have varied between different countries and different sciences. The previously commonly used method of measuring the organic content, the loss on ignition method, has also been rather approximate and contains numerous sources of errors. Soils rich in sulphides and so called "svartmocka", which mainly consists of sulphide-rich silt, have often generally been designated as organic soils. They may or may not contain significant amounts of organic matter, but the loss on ignition is always high because of the sulphides. In the same way, sediments rich in calcium carbonates have often incorrectly been designated as organic soils. There is therefore reason to be very cautious when using data and conclusions from older investigations on "organic" soils.

Better methods for analyses have been introduced and new and stricter rules for classification of organic soils and calcareous soils have been introduced. There are, however, at present no rules for classification of soils according to the content of sulphides.

The organic matter can occur in many forms, mainly ranging from organic substances in the pore water to aggregates formed of organic and mineral matter and pure organic particles in the soil matrix.

Minor amounts of organic compounds have been shown to affect the sensitivity of the soil and may be a contributing factor at the formation of quick clays, (Pusch 1973, Söderblom 1974). Organic colloids in the pore water decrease the permeability of the soil and an increasing amount of organic matter also affects the deformation properties.

It is uncertain whether the presence of aggregates of organic-mineral matter has any significant influence on the strength and deformation properties of the soil, but large amounts, as in the type of soil called "svartmocka", certainly seem to affect the results of some geotechnical testing methods such as the fall cone test, the field vane test and the determination of organic content by the loss on ignition method.

When the soil matrix starts to contain significant amounts of undecomposed remnants of plants there is a gradual change in behaviour. In this case, the soil contains increasing amounts of short elastic fibres with a tensile strength and a preferred horizontal orientation. The contact points in the soil matrix change from mineral-mineral to mineral-organic and organic-organic. This results in changes in shear strength values, shear strength anisotropy, mode of failure, elasticity and anisotropy of permeability.

Common features for almost all the organic soils are that they have very low effective densities and are located near the ground surface in areas with high ground water levels. Most of them have therefore consolidated only for very low effective stresses and are very compressible. This fact, coupled with a very low permeability and high rates of so called "secondary consolidation", makes the consolidation process very time consuming. The organic matter, if fibrous, seems to somewhat increase the shear strength of the soil, but because of the low preconsolidation and the slow consolidation process, both the strength and the gain in strength with time are low. Due to the highly elastic nature of the soil, the shear deformations also become large. Engineering in these soils is therefore an unusually difficult task.

The changes in behaviour of the soil when the organic content increases also affect the testing of the material and the interpretation of the results. The standard procedures and interpretation methods elaborated for inorganic soils are not necessarily appropriate for organic soils.

The present investigation has been made in order to highlight some of these aspects and to find suitable testing procedures for fine-grained soils with organic contents.

3. *PREVIOUS INVESTIGATIONS*

Previous investigations concerning the way in which the organic matter influences the geotechnical properties of fine-grained soils have been made by Pusch (1973) and by Slunga (1983). Pusch mainly investigated the effect of relatively small amounts of organic matter on the properties of clays. A comprehensive description was given of the origin of and the different processes leading to different kinds of organic matter and the current composition of glacial and postglacial clays. Electron-micrographs were used to illustrate various forms of organic matter in the soil. From this investigation it was concluded that even small amounts of organic matter may affect the sensitivity of the soil and that the organic content strongly affects the deformation properties.

The investigation reported by Slunga (1983) was mainly concerned with the shear strength of fine-grained organic soils and the applicability of the usual correction factors for results from field vane tests. Six different clays and gyttjas with organic contents ranging from <2 to about 14 % were investigated. The shear strength was measured by field vane tests in the field, and triaxial tests and direct shear tests in the laboratory were used as references. The triaxial tests were performed in large series with varying rates of deformation in order to determine the rate effects. The results indicated that the usual correction factors could be used also for this type of soil.

In this investigation, also different methods to determine the organic content in soil were studied. Among them was the colorimetric method, which has later become a standard method also in Sweden. A more detailed report with all the test results from this investigation has been given by the Geotechnical laboratory at the Technical Research Centre of Finland (1982).

At the Swedish Geotechnical Institute, investigations relating to the present study have been made by Inganäs (1978) concerning the permeability in organic soils and by Carlsten (1988) concerning the behaviour of peat.

The behaviour of "svartmocka" has been investigated by the Swedish Road Administration (unpublished). On a basis of 5 failures that had occurred, it was tentatively suggested that a correction factor of 0.5 should be applied to strength values obtained by field vane or fall cone tests in this type of soil. Further investigations by Schwab (1976) in three sites with "svartmocka" also indicated that lower correction factors should be used than those normally applied to other fine-grained soils, even if not quite as low as 0.5. The results from a test embankment on "svartmocka" reported by Holtz and Holm (1973), on the other hand, indicated that normal correction factors could be used for that case. The latter investigations show, together with data from Pusch (1973), that the organic content is grossly overestimated if it is determined from loss on ignition also when the usual correction for clay content is applied.

4. SCOPE OF THE PRESENT INVESTIGATION

The present investigation has been aimed at:

- finding simple and reliable methods to determine the organic content, the carbonate content and the sulphide content in soils, (reported by Larsson et al 1985)

- studying present techniques for consolidation testing in the laboratory with special reference to their applicability to organic clays and gyttjas and, if need be, to find more suitable test methods. (Partly reported by Larsson and Sällfors 1985)

- finding new ways to calculate settlements and deformations in soft soils with strongly time dependent stress strain relations, such as organic clays and gyttjas. (Reported by Larsson 1986)

- evaluating the applicability of different types of tests to determine the undrained shear strength and of existing correction factors for field vane tests and fall cone tests in organic clay and gyttja. (Partly reported by Larsson et al 1984)

Furthermore, the influence of the organic content on properties as sensitivity, permeability, coefficient of earth pressure at rest, anisotropy of permeability and undrained shear strength and on the shear strength normalized against the preconsolidation pressure has been studied.

Rate effects in triaxial testing have been studied together with the development of pore pressures in undrained tests and the influence of the organic content on the stress-deformation curves and the failure strains.

The structure and the composition of the investigated soils have also been studied in a scanning electron microscope at Linköping University and X-ray diffraction has then in some cases been used to determine the nature of various particles or aggregates in the soil.

Six different clays and gyttjas have been studied in detail.

The results from this investigation have been combined with the earlier reported investigations and certain data obtained in related research and consulting investigations at the Institute.

5. *ORGANIC SOILS IN SWEDEN WITH SPECIAL REFERENCE TO ORGANIC CLAY AND GYTTJA*

5.1 Origin

The organic matter in soils originates from living plants, animals and organisms, forming biogenic matter in contrast to mineral matter. Aquatic animals and organisms form the basis, or part of it, in many of the more or less biogenic sediments.

In more recent formations, much of the biogenic matter originates directly or indirectly from plants. Animal life on the continents plays a relatively small role from a geological point of view.

The various organic soils are formed at the decomposition of the dead organic substances originating from plants, animals and organisms. This process takes place in different ways, mainly by bacterial activity, and is intensified by a warm climate, suitable humidity and access to oxygen from the air. The processes are mainly as shown in FIG. 1.

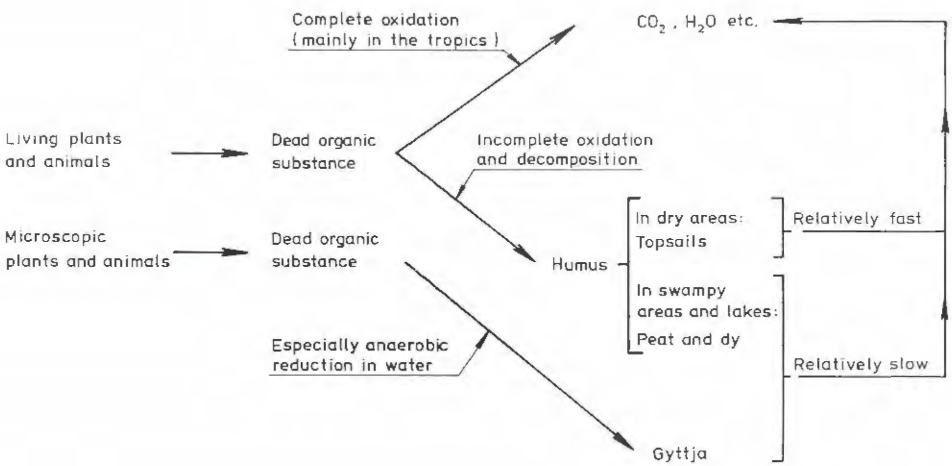


Fig. 1. Schematic process at decomposition of biogenic matter. After Hallden (1961).

Parts of the plant and animal remains cannot decay into humus or gyttja. Shells and the small but resistant SiO_2 skeletons of diatoms and needles of sponge animals are preserved in the soil.

Organic matter in soil is usually identified by its ability to be combusted.

Humus is a dark substance with colloidal structure. The destruction process from dead organic substances to humus, which is the solid product of the process, is called humification. The process takes place with aid of fungae, bacteria and other organisms.

The further decomposition processes are in dry areas so rapid that the thickness of the topsoils seldom exceeds a few tenths of a metre. In swampy areas, the processes are slower as lack of oxygen delays the oxidation. Access to air is prevented and the water in these areas may be almost totally devoid of free oxygen from dissolved air. In the upper layers, there is usually a certain amount of oxygen from precipitation, contact with the air, flowing water and fluctuating water levels. Some of this oxygen may penetrate also to deeper layers. Also in the absence of free oxygen, the decay proceeds in the form of fermentation and putrefaction. This is evidenced by the evolution of gaseous products, such as methane and sulphuretted hydrogen. Other sulphides and compounds devoid of oxygen are also produced. These anaerobic processes are much slower than the decay when there is access to free oxygen.

The formation of peat areas mainly occurs in humid parts of the temperate climate zones. Mires will form and peat accumulate wherever the conditions are favourable, irrespective of altitude or latitude, but are mainly located in those parts of the world where the climate is relatively cold and wet, as in Sweden.

Peat originates from plants and denotes the various stages in the humification process where the plant structure can still be discerned. Dy denotes the various stages in the humification process where the plant structure is completely destroyed. Peat is a sedentary soil which has been formed in place from the original material. Some types of dy are also formed in place, where they constitute the highest degree of humification of the peat. Other types of dy have been transported by water and precipitated in a colloidal form in environments with low contents of calcium.

Gyttja originates from remains of plants and animals rich in fats and proteins, in contrast to peat which is formed from remains of plants rich in carbohydrates. Dead microscopic aquatic animals are dissolved and decomposed with the aid of bacteria to a flocculent substance, in which mineral particles and less decomposed remains of plants and animals are embedded. Further decomposition occurs with the aid of organisms living in the substance, such as worms and larvae. Fermentation processes generating sulphuretted hydrogen and methane complete the formation of gyttja.

Gyttja formed in nutritious water is greenish in colour. In less nutritious environments, the gyttja becomes brown from mixing with brown-black dy. Gyttja has a more or less elastic consistency which is sometimes almost jellylike. Dy has a stickier consistency.

Gyttja formed in areas with calcareous soil often occurs as a transitional form between gyttja and marl called calcareous gyttja.

Depending on the content of mineral particles there are a number of various soils, such as clayey gyttja, organic clay etc. Due to the increasing biological activity at deglaciation, postglacial clays in general contain some organic matter.

5.2 Distribution of organic soils in Sweden

The areas with organic soils in Sweden are relatively large. Exact figures, however, cannot be given. In general, the peat areas alone cover a fifth of the total area of the country. The largest peat areas are located in the northern part of the country, but also in southern Sweden about 8 % of the area is covered by peat. The distribution is uneven, FIG. 2.

Estimating the distribution of the other types of organic soil is more difficult. A coarse method is to study existing maps of clay areas in Sweden and the composition of the topsoils in the cultivated areas, FIGS. 3 a and b.

The map of the clay areas includes glacial clays, postglacial clays and moraine clays. As can be seen from the map over the topsoils, a large portion of the areas along the northern coast denoted as clay areas in FIG. 3a should rather be called silt areas. The clay areas in southernmost Sweden contain moraine clays.

Regarding the distribution of glacial and postglacial clay, the general rule is that topographically lower lying parts mainly contain postglacial clays. This is especially pronounced on the Swedish west coast, where the postglacial clays are totally dominant in the main parts of the valleys. The glacial clay is here found below the postglacial clay and in narrow zones along the valley sides.

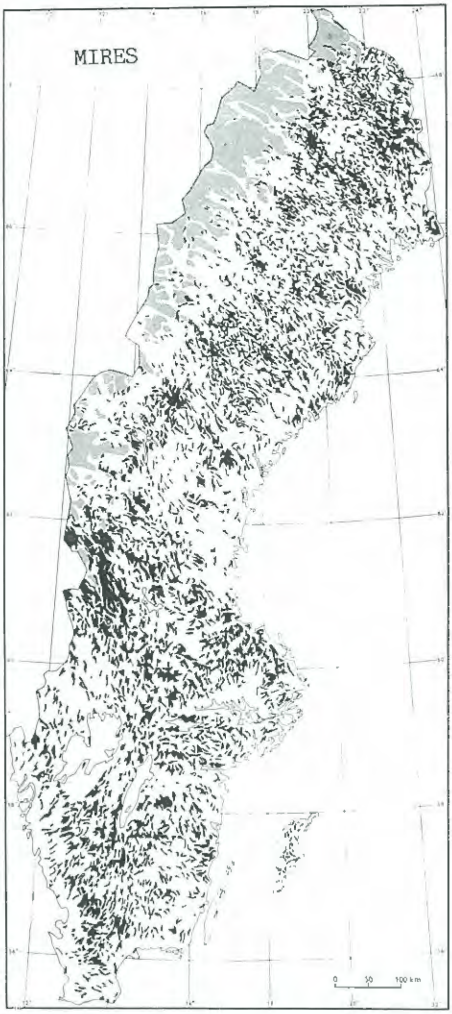
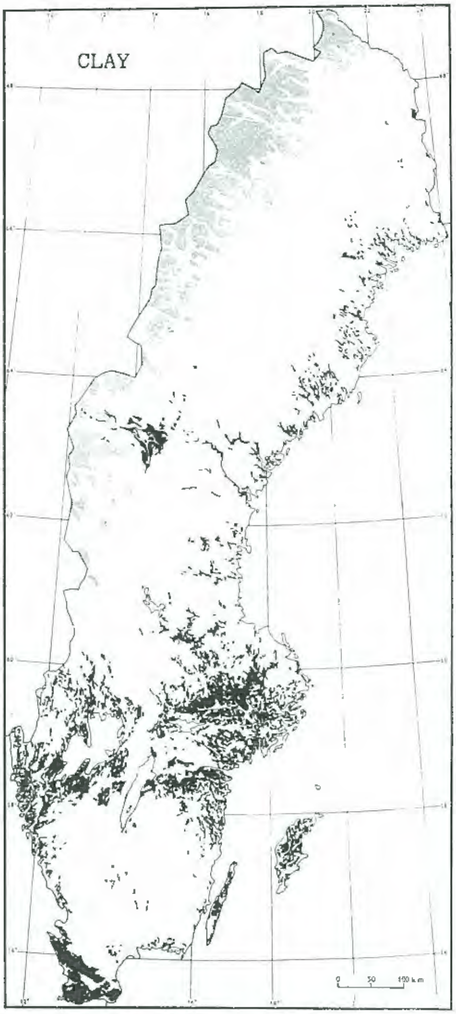


Fig. 2. Peat areas in Sweden. Grey areas represent bare rock, (Magnusson et al 1963).

The glacial clay is inorganic from a classification point of view, even if very small amounts of organic matter may occur also in these soils. The deposition of postglacial clay started at a time when large parts of the land were submerged under the sea. At the same time as the inland ice retreated, the temperature rose and thereby also the biological activity in the areas now free from ice. This brought an increase in the amount of plant and animal remains in the sediments that were deposited. Lakes and seas slowly became shallower by accumulation of a mixture of mineral and organic material on the bottom and by the ongoing land-heave bays were cut off from the sea and became lakes and marshes. Soils such as organic clays, clayey gyttjas and gyttjas were formed as the organic content increased. The postglacial clays, which are located to the topographically lower areas, thus to a large extent became overlain by soils with increasing organic contents. The thicknesses of the deposits vary from place to place. Deposits of postglacial clay of 10 to 20 metres are common, but the thicknesses may in extreme cases exceed 100 metres. The overlying deposits of soils designated as organic mineral or organic soils may be up to 10 metres thick.



a)

b)

Fig. 3a. Clay areas in Sweden. From Atlas över Sverige, bl 15-16. (Magnusson et al 1963).

Fig. 3b. Fine-grained topsoils in cultivated areas. (Magnusson et al 1963).

Gyttja and organic clay occur fairly frequently in the low lying and densely populated areas across central Sweden and along the west coast.

The different soil layers in a profile contain various amounts of calcium. Apart from the clay particles originating from lime-rich rock, also dissolved calcium was precipitated and even in the early stages various amounts of shell-bearing organisms were deposited. The remains of these may be microscopic and evenly distributed in the soil, but also wave-washed layers of shell soil occur. The carbonate contents in different layers thus vary depending on the environment at deposition. In the dry crust and other zones affected by climatic conditions, the carbonate contents have later often been strongly reduced due to weathering and leaching.

Relatively high carbonate contents are frequent in Sweden. In determination of organic content by the loss on ignition method, they constitute a large source of error that previously in many cases has not been observed. A soil profile from a valley on the Swedish west coast is shown in FIG. 4.

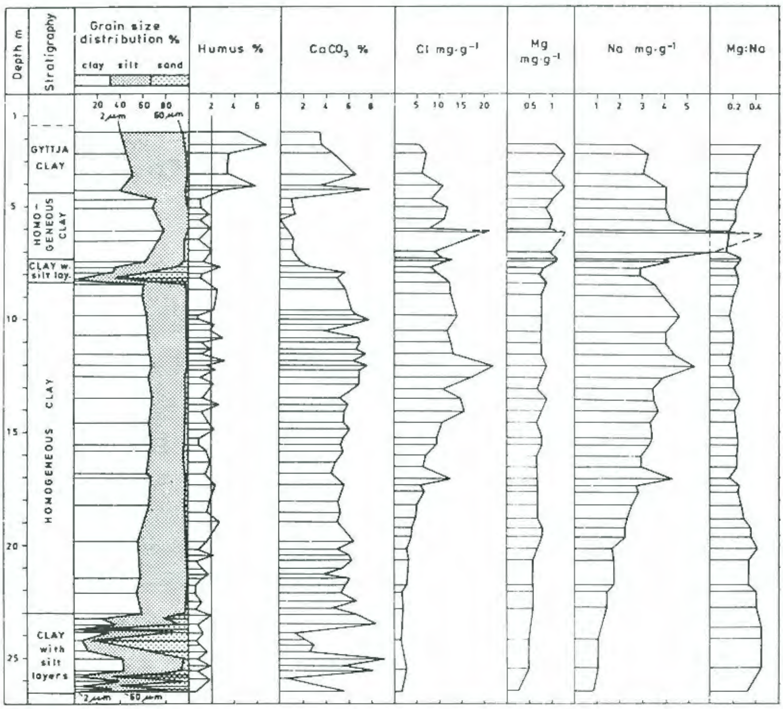


Fig. 4. Distribution of certain chemical and physical parameters in a soil profile from the Swedish west coast. From Cato 1981.

In some areas, the lakes were overgrown and became swamps and fens where different types of peat were formed. Such areas were also created in higher regions by topographical conditions leading to high ground water levels and marshes.

In clay sediments, there are also often layers containing remnants of decomposed organic material, which during processes occurring in a reducing environment have been transformed to ferrous sulphide, among other things. The ferrous sulphide, which in pure form is completely black, occurs as dark spots, patches, bands or completely colours the soil even at moderate contents.

In Finland and northern Sweden, a special type of mass transport took place. Because of the land heave and the narrow fjord-like creeks, the rivers started to erode through the earlier deposited deltaic sediments. The sediments were redeposited further and further away as the coast line was moved. The redeposition occurred in smooth water along the rivers and along the coast. Thick layers of silt and clay mixed with dead algae and remains of animals were formed. The deposition was made in a reducing environment. The decomposition of the organic matter in this environment created a special type of soil called "svartmocka", which covers large parts of the coastal areas of the Gulf of Bothnia but is confined to this region. It consists of silt and clay with a relatively high content of amorphous ferrous sulphide and the colour is black. The sulphide clay in general has a high silt content. The content of gyttja in "svartmocka" varies, but is usually relatively low.

A detailed description of the various kinds of organic substances and the processes forming them in clays with organic matter has been given by Pusch (1973). The formation of mires and peat bogs has been described by Hobbs (1986), among others.

5.3 Classification of organic soils

Classification of organic soil with respect to its colour, consistency and combustible organic substance has been made for geotechnical purposes in Sweden for many years, e.g. SGI Meddelande No. 5 (1959). The combustible organic substance was then measured as loss on ignition with some kind of correction.

The rules for classification now used are in accordance with the Swedish geotechnical classification system worked out by Karlsson and Hansbo (1984) in cooperation with the Laboratory Committee of the Swedish Geotechnical Society.

In this system, the soils are classified with respect to their mode of formation as well as their composition. Organic soils, calcareous soils, shell soils and sulphide soils are also distinguished.

Organic mineral soils and medium-organic soils are classified on a basis of the content and the type of the organic material and of the composition of the mineral material, TABLE 1.

Table 1. Guiding values for the classification of soils on a basis of organic content. After Karlsson and Hansbo (1981).

Soil group	Organic content in weight % of dry material (<2 mm)	Examples of designations
Low-organic soils	2-6	Gyttja-bearing clay Dy-bearing silt Humus-bearing sand
Medium-organic soils	6-20	Clayey gyttja Silty dy Humus-rich sand
High-organic soils	>20	Gyttja Dy Peat Humus-rich topsoil

The nature of the organic material is identified by its mode of formation, by inspection and by certain simple tests.

Gyttja is normally greenish in colour, but can be brown. It bleaches on drying, usually to a grey colour. In the wet state, gyttja has an elastic, rubbery consistency. It has a brittle rupture in bending and tension. It shrinks strongly on drying to form hard lumps with low density.

A special type of gyttja, "algae gyttja", almost exclusively originates from dead algae and the content of plant remains is small. It has a jelly-like consistency, is often brightly coloured and is very elastic. This type of gyttja is relatively rare and no such soil is included in the present investigation.

Clayey gyttja has in the damp state a green-grey colour. It differs from gyttja in the wet state in that it feels sticky, due to the clay content.

Gyttja-bearing clay has in the damp state a dull, slightly greenish, often dark colour, sometimes brown due to the presence of dy, sometimes black or with black patches due to ferrous sulphide. Gyttja-bearing clay is less elastic and less brittle than gyttja. On the soil surface it often cracks in a characteristic cubic pattern.

Gyttja-bearing silt and sand are seldom encountered.

Alkali extracts of gyttja-bearing soils are light yellow or light green in colour.

Calciferous gyttja can be distinguished from marl by lowering a sample into a beaker containing dilute hydrocloric acid. If the sample consists of calciferous gyttja it will retain its gyttja skeleton.

Dy consists of a dense, black or brown soil which, besides dy matter, also contains peat or gyttja matter and mineral particles. Pure dy is seldom seen. On drying, dy retains its dark colour. In contrast to gyttja, dy is relatively inelastic and has a mushy consistency. Like gyttja, it shrinks strongly on drying to form hard, very light lumps.

Sand or silt can often be mixed with dy, giving rise to intermediate forms such as sandy or silty dy or dy with sand or silt layers. Dy-bearing clay is uncommon.

Alkali extracts of dy have a dark colour.

The classification and division of peat is in practice based on ocular inspection of the structure and consistency and on the squeezing test according to von Post (1921). For many engineering purposes only a coarse division is made into three types, TABLE 2.

Table 2. Classification of peat on a basis of decomposition on the von Post scale. After Karlsson and Hansbo (1981).

Designation	Group	Description
Fibrous peat	H1-H4	Low degree of decomposition. Fibrous structure. Easily recognizable plant structure, primarily of white mosses.
Pseudo-fibrous peat	H5-H7	Intermediate degree of decomposition. Recognizable plant structure.
Amorphous peat	H8-H10	High degree of decomposition. No visible plant structure. Mushy consistency.

Fibrous peat is low-humified and has a distinct plant structure. It is brown to brownish-yellow in colour. If a sample is squeezed in the hand, it gives brown to colourless, cloudy to clear water, but without any peat matter. The material remaining in the hand has a fibrous structure.(Degree of decomposition on the von Post scale; H1-H4)

Pseudo-fibrous peat is moderately humified and has an indistinct to relatively distinct plant structure. It is usually brown. If a sample is squeezed in the hand, less than half of the peat mass passes between the fingers. The material remaining in the hand has a more or less mushy consistency, but has a distinct plant structure, (H5-H7).

Amorphous peat is highly humified. The plant structure is very indistinct or invisible. It is brown to brown-black in colour. If a sample is squeezed in the hand, more than half of the peat mass passes between the fingers without any free water running out. While squeezing, one can only feel a few more solid components such as root fibres, wood remnants, etc. These constitute any material remaining in the hand, (H8-H10).

Calcareous soils are classified on the basis of their content of calcium carbonates.

Presence of sulphides is always noted at classification of soils but the designation sulphide soil or "svartmocka" is used exclusively for the sulphide-rich soils in the coastal areas around the Gulf of Bothnia. No classification based on the content of sulphides has been made so far.

6. *SPECIAL PROBLEMS INVOLVED AT SAMPLING AND TESTING OF SOILS WITH ORGANIC MATTER*

The soils in Sweden that contain significant amounts of organic matter are in general more difficult to sample and to test than inorganic clays. An increasing amount of fibres makes it more difficult to produce clean cuts at sampling and further trimming of specimens. The organic matter also leads to a more elastic and time dependent behaviour of the soil. In sampling with piston samplers, great care has to be taken to avoid unnecessary inner clearances in the samplers, to provide sufficient waiting time for the soil to adhere to the walls of the internal liners or sample tubes and to withdraw the sampler carefully, thereby avoiding any unnecessary suction on the samples. The samples otherwise have a strong tendency to elongate because of the stress relief at sampling and thereby often become decreased in diameter so that they do not quite fill the sampling tubes.

This tendency to elongate continues with time after sampling. There is usually also some biological activity in the samples, which will increase if the temperature is raised. If the samples are exposed to free air the chemical environment changes, whereby oxidation and certain changes in properties may occur. It is therefore important that the samples be well sealed, stored at ground temperatures and tested within a short time after sampling.

The elasticity of the soil entails that criteria often used to estimate the quality of undisturbed samples in inorganic soils are not applicable to soils containing organic matter. The strain at failure in shear strength tests increases with the organic content and is not a measure of the quality of the samples. In oedometer tests, the strain upon reaching the preconsolidation pressure is often in the order of 4 - 5 %. Nevertheless, experience has shown that the preconsolidation pressure and all other consolidation parameters can be evaluated with a satisfactory degree of accuracy, (Sällfors 1975, Larsson 1986).

The soils containing organic matter often have very low preconsolidation pressures and are highly compressible. Due to their low preconsolidation pressures they also have low shear strengths. Testing in the preferred way with samples reconsolidated to in situ stresses in direct simple shear tests and triaxial tests therefore means testing at very low stress levels. This puts special demands on stress applications and measuring accuracy that often cannot be met with standard equipment. Sources of errors in the tests, such as friction and stresses in the membranes, also exert great influence and should preferably be eliminated. In the Swedish type of direct simple shear apparatus, which has a minimum vertical stress of 15-20 kPa, some samples cannot be tested at in situ stresses. One method often used is to run the tests on samples that are consolidated for higher stresses and then normalize the results against the preconsolidation pressure. In the case of soils with organic matter, a consolidation for higher stresses involves large deformations and significant changes in the soil structure. It has also been shown that the strength properties are not linearly related to the preconsolidation pressures in these soils, but several tests have to be performed in order to allow non-linear extrapolation of the results, (Bergdahl et al 1987).

When normalization is used, this puts high demands on accuracy in the determination of the preconsolidation pressure. This determination can be made with fairly good accuracy, but the relative scatter increases with decreasing preconsolidation pressures. Several tests may therefore be required to obtain a reliable average value. When the results of tests performed at in situ stresses are normalized, the scatter becomes unusually large as it consists of two relatively small numbers, which both have relatively high scatter, which are compared to each other and the scatter is magnified.

In routine testing of soils with organic matter, there are also special problems. The liquid limit of the soil can be determined either by the fall cone method or by percussion in the Casagrande device. In inorganic clays, the results from the two types of test are usually very similar, but in organic clays there is a clear trend that the latter limit becomes higher, (e.g. Karlsson 1981). One reason for this may be that the energy applied to the soil in the percussion test depends on its own density, which becomes lower when the organic matter increases. In Sweden, the fall cone method is the standard method and has been used exclusively in the present investigation. In some of the related investigations both methods have been used.

The grain size analyses by sedimentation tests are affected by the organic matter in the soil. Traditionally, this matter is removed by oxidation with hydrogen peroxide and washing. This is a very tedious procedure, which also may involve some unwanted changes in the soil. Parallel to the traditionally treated samples, sedimentation tests have also been performed on samples where only the salts in the pore water have been washed away. After correction for the weight of the organic matter, the results seem to be compatible, (Larsson 1985).

The organic content of the soil has until a few years ago mostly been estimated from loss on ignition for geotechnical purposes. This has been shown to be highly erroneous in soils containing carbonates or sulphides, which are both frequent in Swedish soils. In soils free from sulphides, the loss on ignition can be used if both the carbonate content and the clay content are determined and corresponding corrections are applied. All values of organic content in fine-grained soils determined by loss on ignition where these corrections have not been applied must be treated with great caution. For soils containing sulphides, the method cannot be used as there are no rules for correction for sulphide content.

Today, the loss on ignition method is used at the Swedish Geotechnical Institute only for soils with high organic contents, 20 % or more, and which are mainly free from carbonates. For all other soils, the simple colorimetric method with wet combustion and determination of intensity of colour in a photometer is used.

In Sweden, the undrained shear strength in fine-grained soils is normally determined by vane tests in the field or by fall cone tests in the laboratory. Both tests give similar values and the results have to be corrected before they are used in calculations. This correction is based on and increases with the liquid limit. Increasing organic contents generally involve increasing liquid limits and in organic soils correction factors down to 0.5 may have to be applied. This procedure in general works out fairly well in organic clay and gyttja, but in "svartmocka" the scatter is great. Schwab (1973) suggested that screw-plate tests could be used in this type of soil.

Lately, the dilatometer has been shown to give representative values of the shear strength in organic clay and gyttja,(Larsson and Eskilsson 1989). No similar research has been made in svartmocka.

Research on the combined cone penetration and pore pressure sounding test, (the piezocone or CPTU-test), is currently being carried out in both organic and inorganic soft soils.

7. *METHODS USED IN THE PRESENT INVESTIGATION*

7.1 Sampling and field testing

At each site, samples were taken at every metre of depth in 10 boreholes located in a square pattern with c/c 2m. In some cases, where the organic layers had a more limited thickness, samples were taken at every 0.75 metre of depth. The samples were taken with a Swedish standard piston sampler, which gives three samples 170 mm long and 50 mm in diameter at each sampling depth, (Kallstenius 1963). The samples were carefully brought into the laboratory, where they were stored in humid rooms at a temperature of 7^0 C. Of the three samples, only the lower two are used for qualified testing. The upper sample is used for visual inspection, determination of organic content, grain size analyses and other tests which do not require an undisturbed sample of the highest quality. Testing of the samples in the laboratory commenced as soon as they were brought in.

In the field, vane tests were performed at both sides of the sampling area. The tests were performed with the Geotech lightweight equipment and with the standard procedures for installation and rotation of the vane. These procedures specify that rotation of the vane shall start within 5 minutes of its installation and that failure shall be obtained within 1 to 3 minutes of rotation time. The vane size was 65 x 130 mm and all other dimensions were in accordance with the specifications. The tests were performed at the same depths as the samplings and the results from the different vane tests were very uniform. The ground water level at the time for the sampling was observed in open boreholes.

The procedure described above was used at the four new test fields. At the old test fields, the vane testing was omitted as such data were already available and sampling was made only to enable certain supplementary tests. In many of the old investigations and in some of the supplementary tests in other investigations, the heavier SGI-type of field vane equipment was used. The dimensions and procedures are basically the same as for the Geotech equipment and the results are normally compatible, except for varved and highly stratified soils where the disturbance at installation of the vane is unusually high. The results from both types of test are corrected in the same way on the basis of the liquid limit of the soil.

The relevance of the normally used correction is, as a first step, judged on the basis of the relation between the obtained uncorrected shear strength value (τ_{vane}), the preconsolidation pressure (σ'_c) and the liquid limit (w_L). The normal relation is $\tau_{vane} = \sigma'_c \cdot 0.45\ w_L$. If the uncorrected shear strength value is significantly higher than this relation yields, there is a great risk that the strength value will have to be reduced more than the general correction specifies. If the uncorrected shear strength value is significantly lower, supplementary tests will probably give higher values of the shear strength.

Later, in 1989, when the dilatometer tests were performed in the new test fields, the pore water pressures were also measured at different levels in the soil profiles.

7.2 Laboratory tests

7.2.1 Routine tests

In the laboratory, the usual properties bulk density, natural water content, liquid limit and plastic limit were determined. The Swedish routine also includes determination of undisturbed and remoulded shear strength by the fall cone test. The liquid limit was determined by the fall cone method, (Karlsson 1981).

7.2.2 Classification tests

The organic content was determined by a large number of both simple and more qualified methods. The more qualified methods and the simple colorimetric method gave compatible results, which have been used.

The carbonate contents were determined by a simple method devised by Moum (1967) which also was shown to give compatible results with more advanced methods.

In a few cases, the sulphide contents have been determined by a relatively simple method suggested by Svensson (1983) mainly based on Steinrath (1966).

The methods used for determination of organic content, carbonate content and sulphide content are described in detail by Larsson et al (1985).

The grain size distribution was determined in sedimentation tests and measurements with the suspended glass body method used at SGI, (Karlsson 1973).

7.2.3 Microscope studies

The general structure of the soils was studied on freeze-dried specimens. The dry specimens were broken to give horizontal and vertical fracture surfaces before they were mounted and coated for microscopy. The microscope studies were made at the University of Linköping using a scanning electron microscope of the Cambridge type equipped with facilities for x-ray diffraction. The usual procedure was to first scan the whole fracture surface and locate a representative area. This area was then successively magnified and photographed. In some cases individual particles and aggregates were x-rayed in order to establish their composition.

7.2.4 Oedometer tests

The oedometer tests were performed both as step-loaded tests and as tests with constant rate of deformation, CRS-tests. Series of tests were performed for each sampling level at each site.

The step-loaded tests were performed in a type of oedometer designed at SGI, which can also be modified for direct simple shear tests. The specimens are 50 mm in diameter and 20 mm high and are confined in a ceramic ring. The ring is greased with a silicone compound and the specimen is pushed directly from the sample tube into the ring by a special extruding jack. The ends of the specimen are cut by a thin stretched piano-wire and are carefully scraped off to be flush with the end planes of the ring. The ring is then placed in the oedometer in the loading frame, where filter-stones provide drainage at both ends, FIG. 5.

The series of step-loaded tests were performed in such a way that six specimens from the same sample tube were mounted at the same time in six oedometers in a row. The first load steps were then slightly varied in the different oedometers, e.g. 3.5, 4, 4.5, 5, 5.5 and 6 kPa. The load steps were applied for 24 hours and were then doubled. The 24-hour readings were plotted together in a single diagram and in this way an oedometer curve with six times as many points as usual was obtained, leaving very little room for subjective interpretation.

The preconsolidation pressure was evaluated according to the widely used Casagrande (1936) method, which is the usual procedure for such tests.

From the step-loaded tests also the coefficient of secondary consolidation, α_s, was evaluated as the slope ($d\varepsilon/d\log t$) of the time-settlement curve at the end of each load increment, FIG. 6.

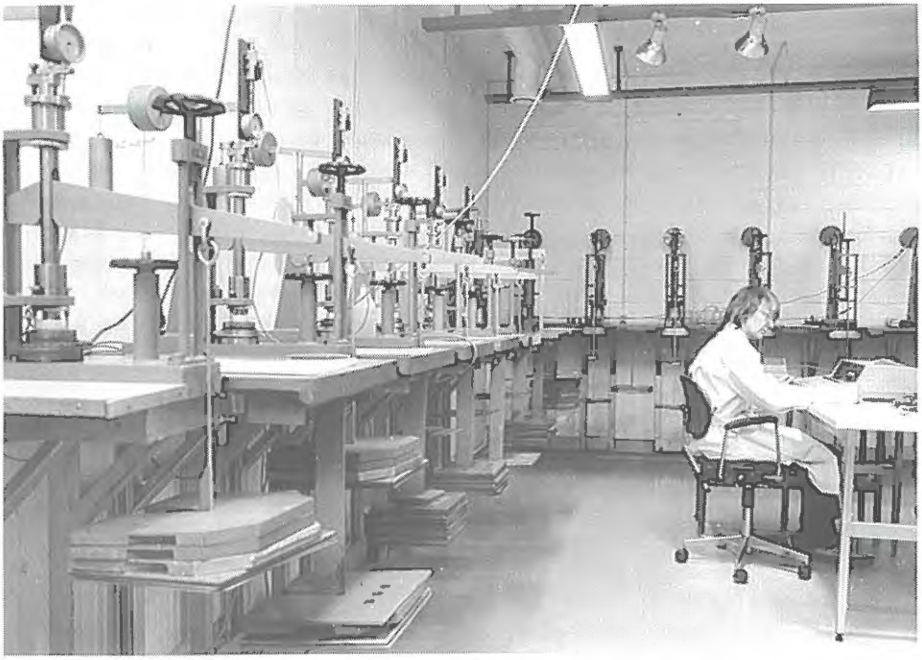
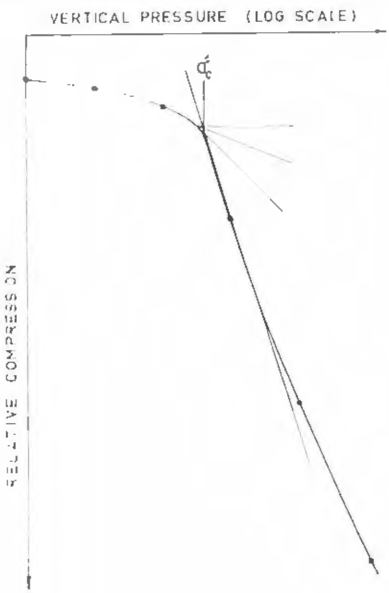


Fig. 5. SGI oedometers for step-loaded tests.



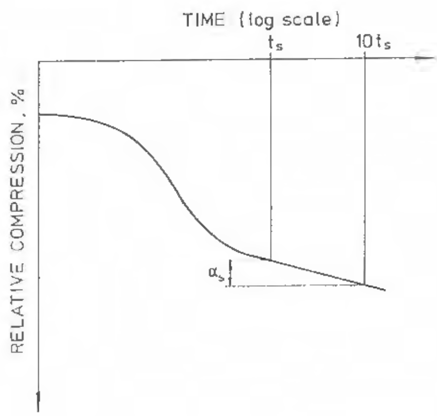


Fig.6. Evaluation of preconsolidation pressure and coefficient of secondary consolidation from step-loaded oedometer tests.

The CRS-tests, i.e. oedometer tests with constant rate of strain, are in reality performed with constant rate of deformation, but the notation "CRS-test" for this type of test is internationally used.

In the CRS-test, the specimen is mounted in the same way in a ring with identical dimensions as in the step-loaded test. It is then 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. 7.

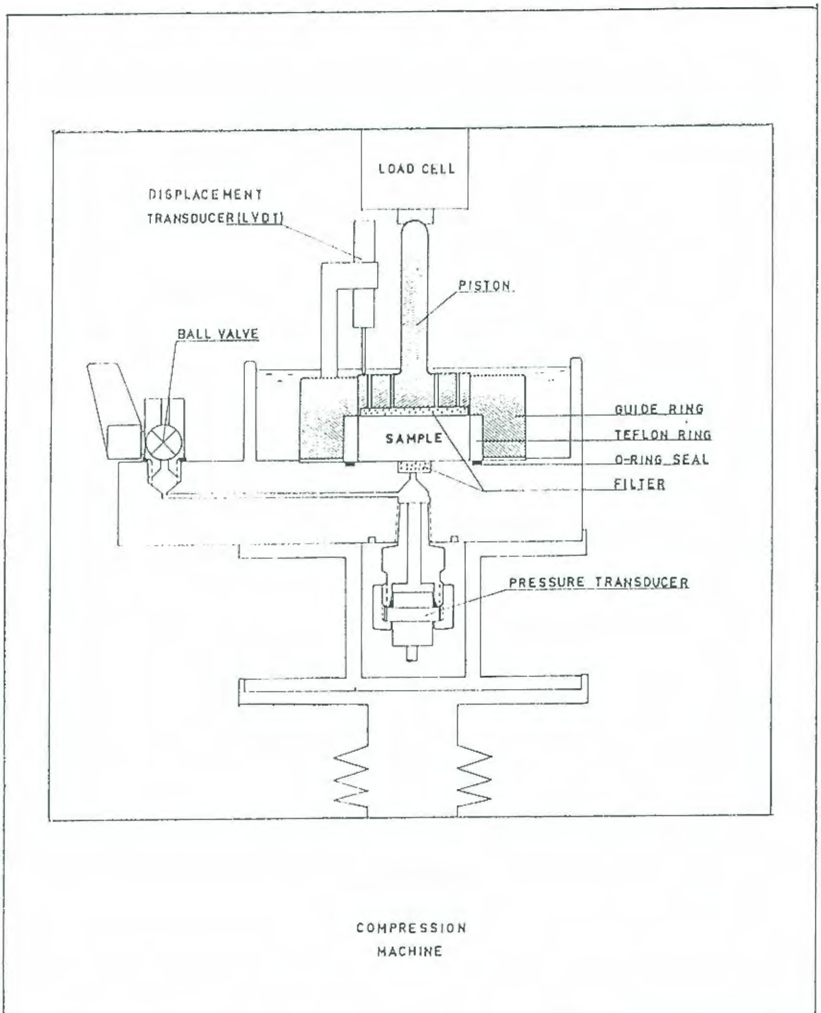
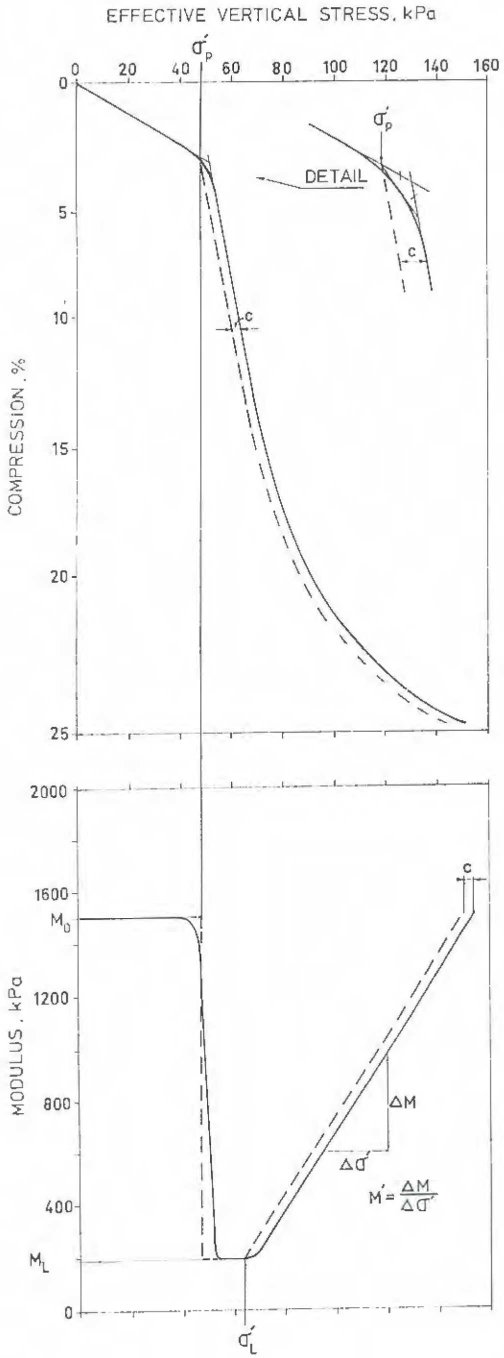


Fig. 7. Oedometer for CRS-test.

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 the variation of the compression modulus with effective stress can be evaluated, FIG. 8.



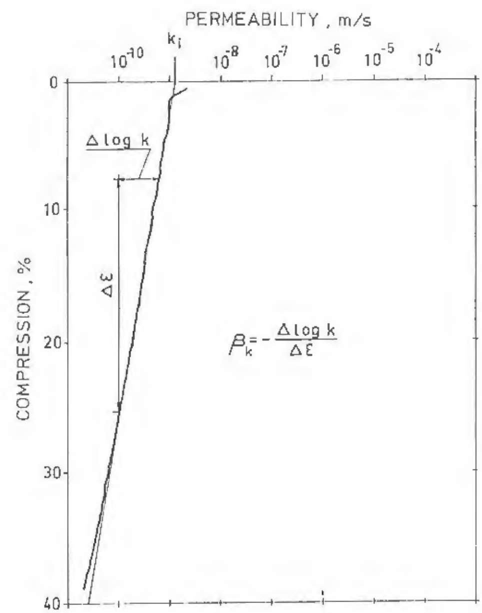


Fig. 8. Results from CRS-tests and evaluation of compression and permeability properties.

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, see FIG. 8.):

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 σ'_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. Both stresses and strains are plotted in linear scales.

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 σ'_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 plot is now modified. The initial constant modulus M_0 is extended to σ'_C. At σ'_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 σ'_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 σ'_0-σ'_C where $M = M_0$

2. The part in the stress interval σ'_C-σ'_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 the in situ initial modulus due to sample disturbance, swelling, and imperfect fit in the oedometer. In most cases M_0 is estimated from empirical relations such as $M_0 \approx 250\ \tau_{fu}$ or $M_0 \approx 50\ \sigma'_c$. To obtain a useful value of M_0 in the laboratory , the sample has to be unloaded when σ'_c is just exceeded to the "in situ" effective vertical stress σ'_0. It should then be allowed to swell before it is reloaded. M_0 is then evaluated from the reloading curve.

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

The CRS-tests were also run in series with several specimens from the same sample tube. In the test series, the rate of deformation was varied, usually by running two tests at the standard rate of 0.0024 mm/min, one at 0.0012 mm/min, one at 0.0006 mm/min and another test (or tests) faster than the standard rate. The program was varied somewhat depending on what other tests were run at the same sampling level.

Apart from CRS-tests, also tests with so-called "continuous loading", CL-tests, were performed. In this type of test the rate of deformation is electronically controlled by the recording computer in such a way that the relation between the applied vertical stress and the pore pressure at the undrained end of the specimen is kept constant, (Janbu et al 1981). The results from some of these tests have been reported by Larsson and Sällfors (1985) and are not treated in this report.

A smaller CRS-oedometer with a sample diameter of only 40 mm was also used. In this oedometer specimens were cut out and tested in a direction perpendicular to the vertical. The main reason for this was to measure the horizontal permeability of the soil, but it also provided an estimate of the preconsolidation pressure in the horizontal direction.

7.2.5 Direct simple shear tests

In Sweden, shear strength is often determined by direct simple shear tests in the SGI apparatus. The apparatus is a modified SGI oedometer with facilities for shearing the soil sample after consolidation. The sample has a diameter of 50 mm and the sample height after consolidation should normally be between 10 and 20 mm. Depending on the compression during consolidation, the sample is consolidated in slightly different ways.

The sample is placed on a fixed pedestal ø 50 mm, which has a filter-stone on top and an internal drainage channel. A top part, which at this stage is fixed so that it can only move vertically, is lowered onto the sample. This part also has a diameter of 50 mm, a filter-stone and a drainage channel. If the compression during consolidation is expected to be small, the sample is surrounded by a rubber membrane. Thin metal rings are fitted outside the membrane to keep the sample diameter constant during the test. There are small vertical clearances between the rings to prevent transmission of vertical forces by the rings. The rubber membrane is sealed against the pedestal and the top part by clamps and drainage is provided by the filter-stones and the drainage channels.

If the compression during consolidation is expected to be large, the sample is first consolidated inside a confining ring, where most of the compression is allowed to take place. The confining ring is then replaced by the rubber membrane and the metal rings before the final load is applied. This operation is carried out with the sample in place in the shear apparatus.

The apparatus is shown in FIG. 9.

The test is performed by first applying the vertical load to the sample and allowing it to consolidate.

After consolidation the sample is sheared. The bolts fixing the top part are removed and the ball bearing which transmits the vertical load allows the top part to move horizontally with a tolerable amount of friction. In shearing, the top part is moved horizontally at a constant speed while the bottom pedestal is fixed. The sample thereby undergoes a fairly uniform angular distortion. During the test, the horizontal shear stress, the horizontal displacement of the top part and the height of the sample are measured by electronic transducers and automatically recorded.

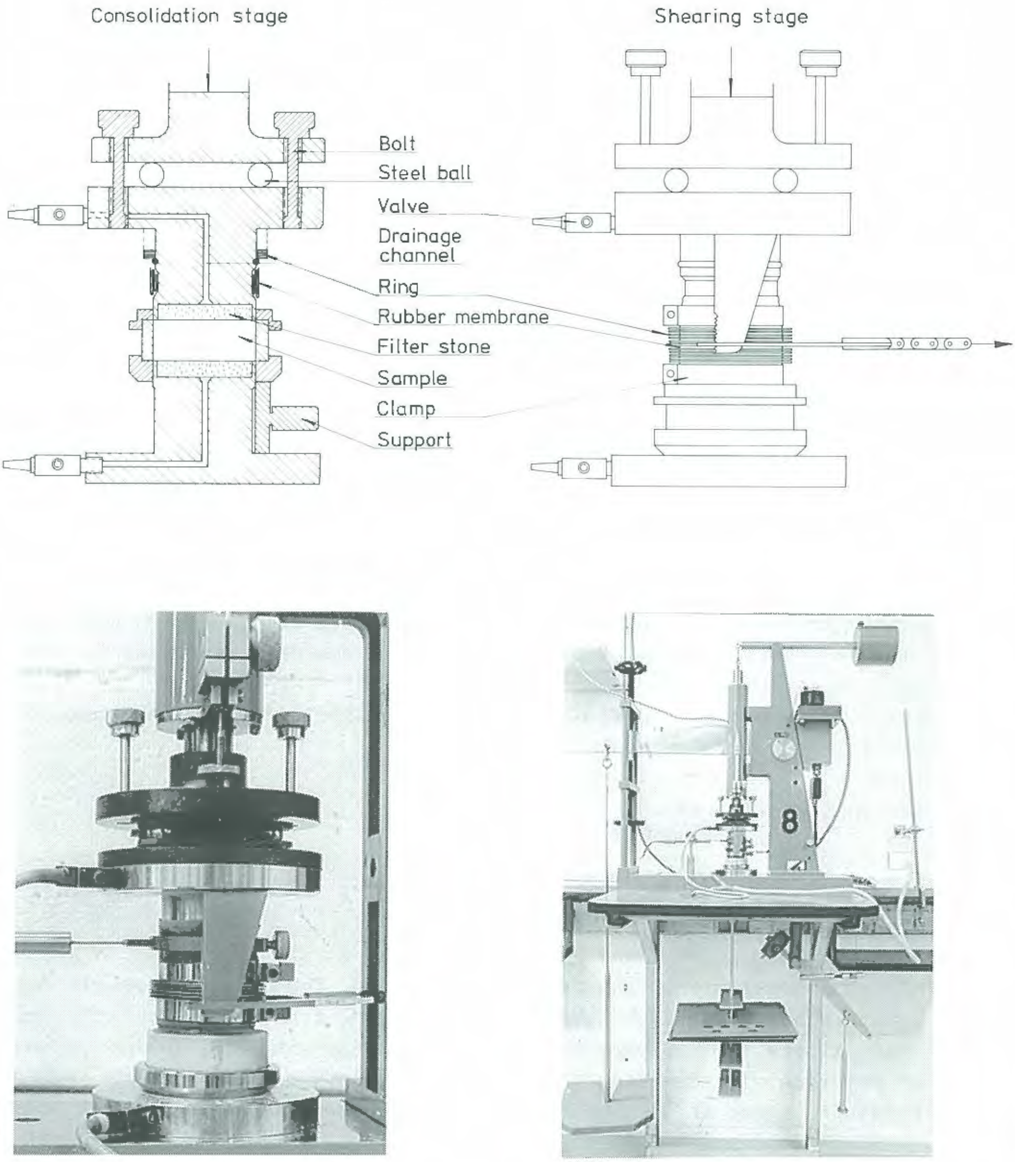


Fig. 9. The SGI direct simple shear apparatus.

The tests can be drained or undrained. In drained tests the vertical stress remains constant and the drainage channels remain open. In undrained tests the height of the sample is fixed during shear and the channels are closed. Consolidation is usually allowed for 24 hours and the rate of shear is such that an angular distortion of the sample of 0.15 radians is obtained in another 24 hours.

The weight of the top part and its accessories produces a vertical stress on the sample of about 15 kPa and the minimum vertical stress at which tests can normally be performed is around 20 kPa.

Direct simple shear tests have been made on all the soils at the SGI sites. In some cases the tests were performed at elevated preconsolidation pressures and the results have been extrapolated non-linearly to correspond to the actual preconsolidation pressures in the ground. Drained and undrained tests have been performed.

7.2.6 Triaxial tests

The triaxial tests were performed in the triaxial equipment at SGI and, in the case of Välen organic clay, in the equipment at Chalmers University. The two equipments are very similar and consist of modified Geonor triaxial apparatuses. The modifications consist of replacing the old pressure system with air pressure regulators and replacing the old measuring system with electronic transducers. The load cell measuring deviatoric stress is placed inside the triaxial cell to avoid friction errors, FIG. 10.

The triaxial cell is elongated to accommodate the load cell and soil samples with a length of 100 mm and a diameter of 50 mm. The load cells have a maximum working range of ± 200 N which corresponds to a difference between vertical and horizontal stress in the sample of about ± 100 kPa. The cells are thus specially designed for soils with shear strengths lower than about 40 kPa.

The top cap and the pedestal are made of polished perspex. To minimize end friction only the pedestal has a filter-stone which covers half the cross-sectional area. The samples are pushed out of the sample tubes with the special extrusion jack and cut to the right length with plane and parallel ends perpendicular to the length axis. No trimming of the diameter is performed as trimming is considered to introduce a new disturbance approximately equal to that at sampling and then leave an even smaller specimen to test. The sample is placed on the pedestal and the top cap is placed on top of the sample. Both pedestal and top cap have the same diameter, 50 mm.

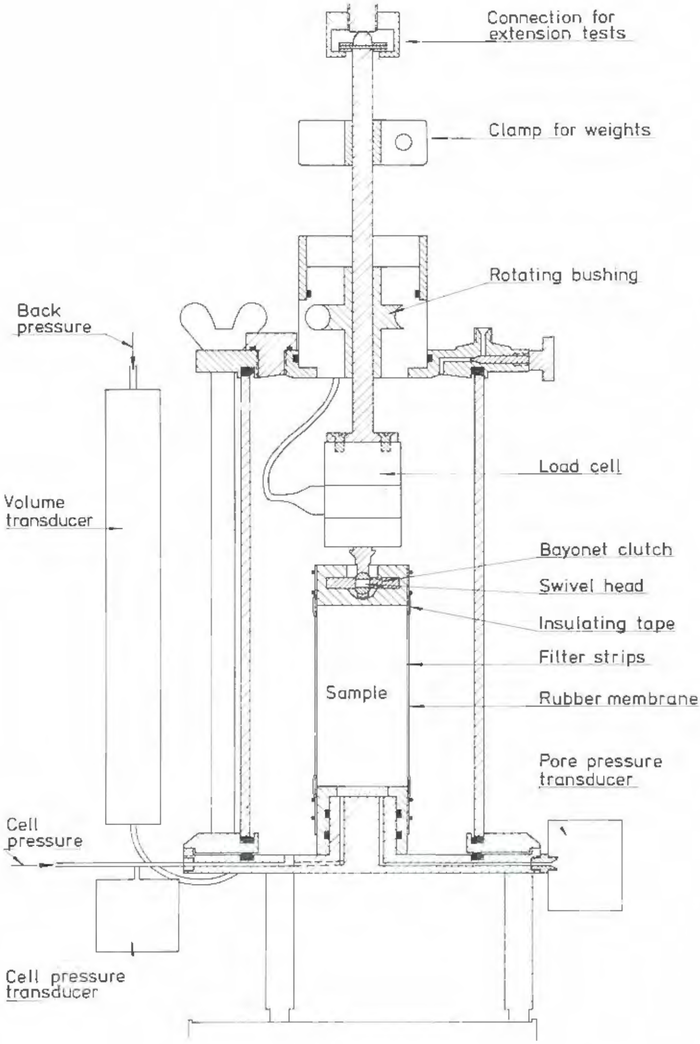


Fig. 10. Triaxial cell with internal load cell.

In extension tests a top cap which can be connected to the load cell by a bayonet clutch is used. In these tests a layer of insulating tape is applied at the joints between sample and top cap and sample and pedestal to prevent intrusion of the rubber membrane into the joints. Filter papers cut in spirals as recommended by Berre (1981) are used to provide drainage. A paper quality with low strength is used. Membranes of latex rubber are used around the samples and paraffin oil is used as cell fluid. The paraffin oil partly dissolves the rubber membrane which at the end of the reconsolidation phase has the shape of a crumpled loose-fitting bag unable to take up any significant stresses. The tests can be performed without a membrane as paraffin does not penetrate fine-grained soils. The role of the membrane is to prevent the paraffin oil entering and blocking the paper drains and, especially in passive extension tests, preventing the cell fluid from entering joints, cracks or fissures or more permeable layers. Paraffin oil or other type of non-conducting fluid also has to be used in the triaxial cell because of the submerged electronic load transducer.

Samples are normally reconsolidated to in situ stresses for 1 - 2 days before testing starts.

After determination of the preconsolidation pressure and estimation of the coefficient of earth pressure at rest by oedometer tests, samples from all the levels at each site were reconsolidated for estimated in situ stresses. Undrained triaxial tests were then performed, in most cases as both compression and extension tests with both the standard rate of deformation of 0.6 %/hour and a ten times lower rate. In some cases series of compression tests with a wider range of deformation rates were performed. Sometimes, also drained tests were performed with respectively slow increases in vertical or horizontal stress in order to verify the preconsolidation pressures obtained in the oedometer tests. In a few cases also undrained tests with increased preconsolidation stresses were performed.

8. INVESTIGATED SOILS AND MAIN TEST RESULTS

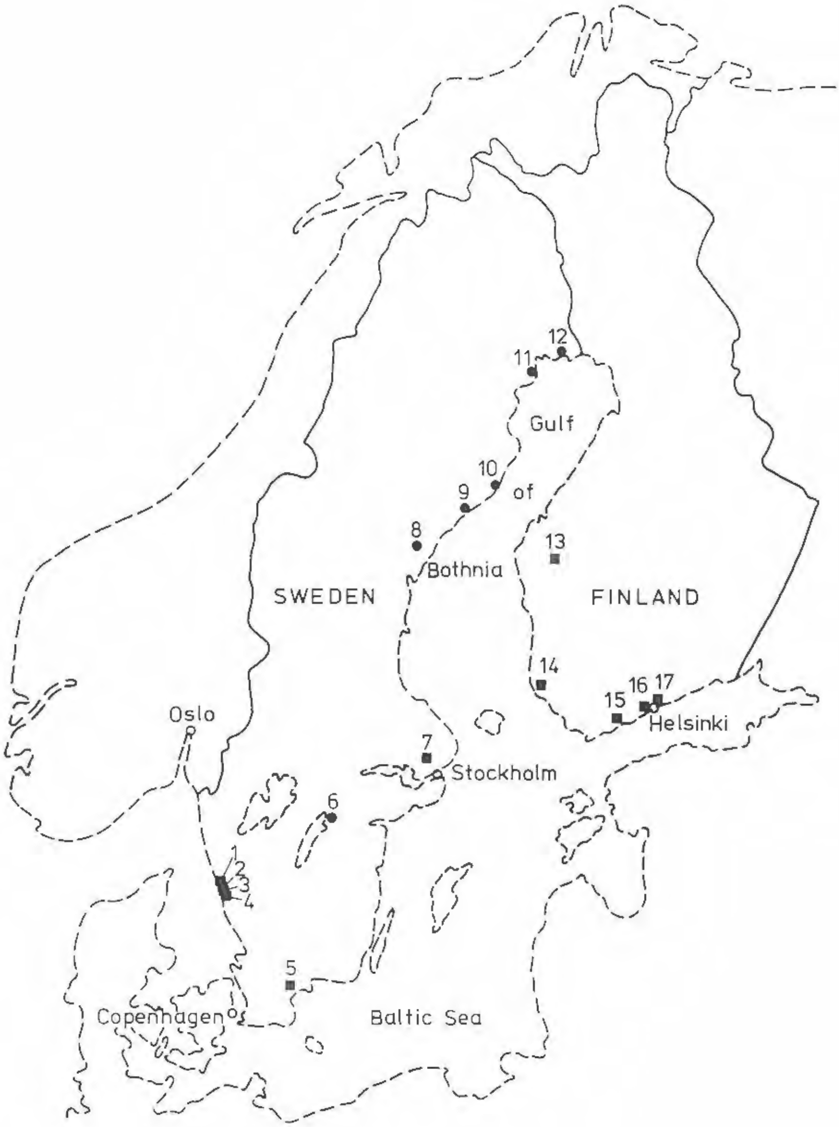
The sites in the present investigation were selected from information about the soil properties already available at the Institute. The prerequisite was that the layers of soil with organic matter should be fairly thick and very homogeneous. The sites were then selected to cover a range of soils with organic contents from slightly over 2% to over 20%. The selection was actually based on the consistency limits as the organic contents could only be roughly estimated from available data. The locations of the sites and of the other soils from which supplementary data have been used are shown in FIG 11.

8.1 Lilla Mellösa

The test field at Lilla Mellösa has been used by SGI since 1945. It is located north-east of Upplands-Väsby, about 40 kilometers north of Stockholm. The field has mainly been used for long-term observations of test fills constructed in 1945-1947, as well as for a number of different investigations concerning sampling and determination of strength and deformation properties, (Cadling and Odenstad 1950, Chang 1969 and 1981, Wiesel 1975, Larsson 1977, 1981 and 1986, Tavenas et al 1983, Carlsten and Eskilson 1984 and Larsson and Eskilson 1989).

The area is very flat and the soil consists of 14 metres of clay on top of a thin sand layer and rock. The soil profile is typical for a large number of similar profiles where the upper post-glacial soil layers are organic (gyttja) or slightly organic and the soil is coloured black or dark grey by a certain content of sulphides. A detailed description of the soil and its properties has been given by Chang (1969 and 1981). This description has been somewhat modified, mainly concerning the organic content and the deformation properties, as new and better methods to determine these properties have been introduced (Larsson 1986). The soil profile is shown in FIG 12.

At the top, there is a layer of organic topsoil. The dry crust is unusually thin and consists of organic soil. The desiccated crust is limited to 0.5 m and lies on 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 m 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 0.5% of the dry weight of the soil.



1 Välen	10 Umeå
2 Section 6/900	11 Luleå
3 Section 7/600	12 Kalix
4 Kungsbacka	13 Seinäjoki
5 Kristianstad	14 Lokalahti
6 Motala - Askersund	15 Siuntio
7 Lilla Mellösa	16 Rastaala
8 Kramfors	17 Vanhankau-punginlahti
9 Gideåbacka	

Fig. 11. Location of the test sites.

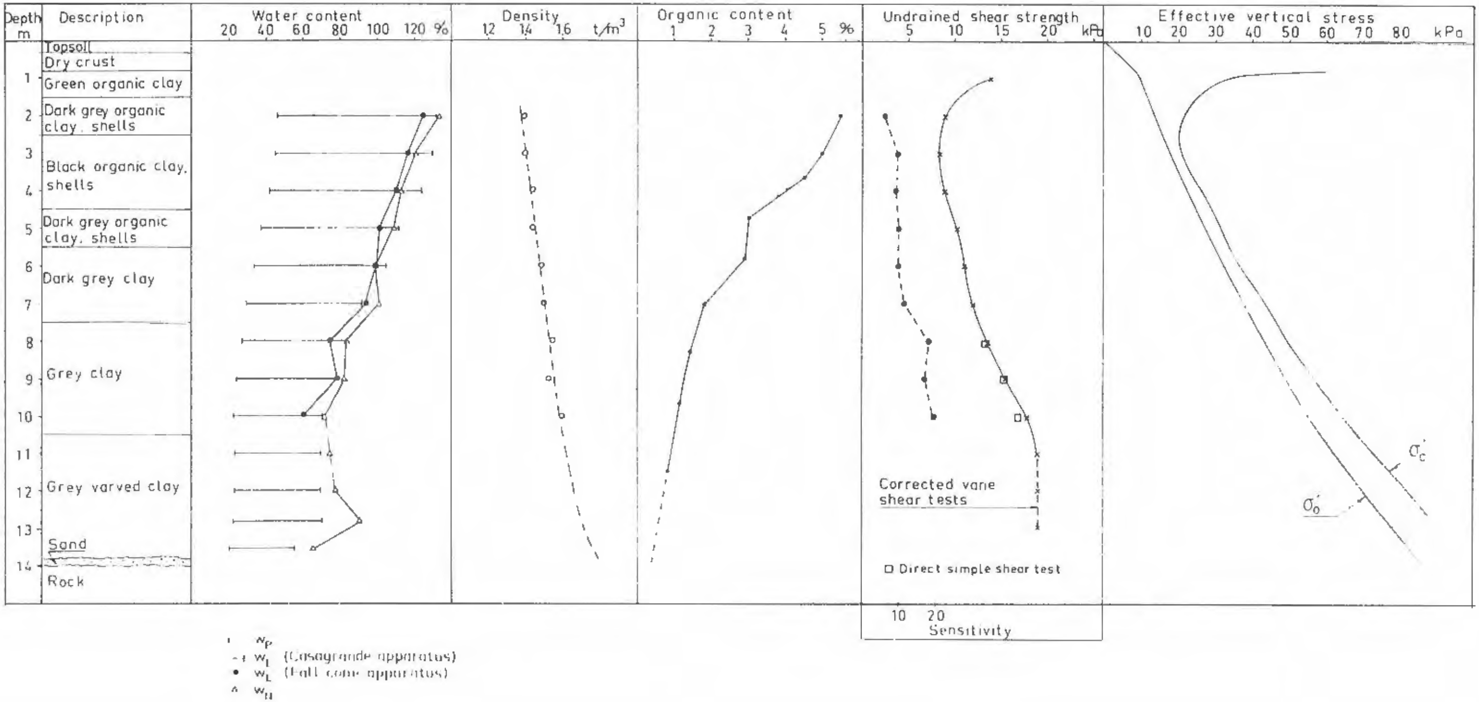


Fig. 12. Soil profile at Lilla Mellösa.

The natural water content is about equal to the liquid limit and decreases from a maximum of about 130% to about 70% in the bottom layers. The bulk density increases from about 1.3 t/m^3 to about 1.8 t/m^3 at the bottom. Below 10 m depth the clay becomes varved. The varves are at first diffuse, but become more and more pronounced with depth. The undrained shear strength as determined by corrected field vane tests has a minimum of 8 kPa at 3 m below the ground surface and then increases with depth. The sensitivity in the soil varies between 10 and 20. The shear strength of the soil in the grey inorganic clay between 8 and 10 metres depth has been studied in triaxial tests and direct simple shear tests in a special investigation (Larsson 1977).

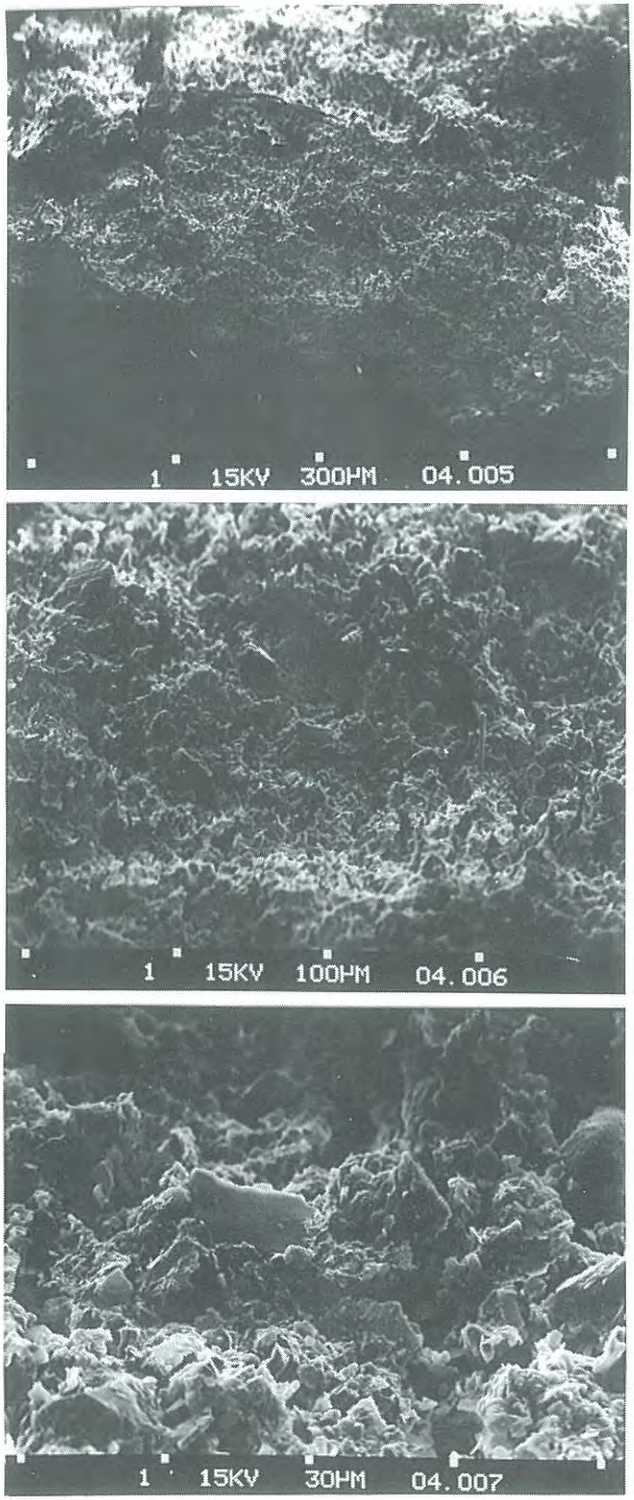
The pore pressure in the original ground has been found to be close to hydrostatic for a groundwater level 0.8 m below the ground surface. The permeability of the soil has been studied in both field and laboratory tests (Tavenas et al 1983, Carlsson and Eskilson 1984, Larsson 1986).

Overconsolidation because of the various effects in the dry crust and directly below it occurs down to abut 2.5 m depth. Below this depth the overconsolidation ratio is almost constant 1.2.

In 1982 new samples were taken with a block-sampler in a joint research with Laval University in Quebec (La Rochelle et al 1981). From the block-samples taken between 3.6 and 4.0 m depth specimens were trimmed and then tested in CRS-tests, direct simple shear tests and triaxial tests. Specimens were also freeze dried and examined in the electron microscope, FIG. 13.

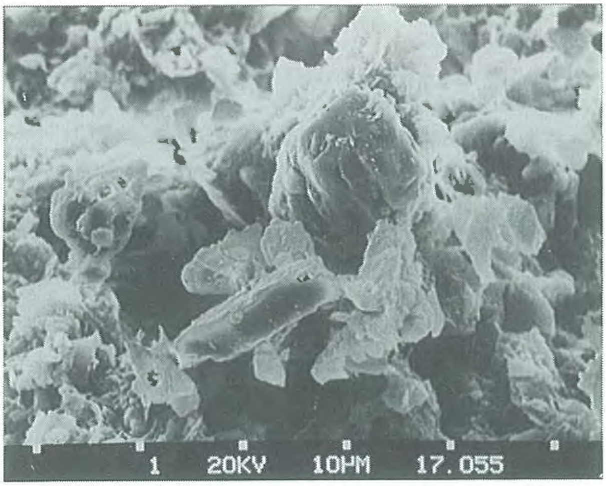
The magnifications are in the range 100 to 1000 x and a scale is given at the bottom of the micrographs. The organic content of the soil at 3.75 m depth is between 3 and 4%. Occasional organic fibres and some diatoms can be observed, but otherwise no significant difference in structure compared to the "inorganic" soil between 8 and 10 m depth can be seen. The content of ferrous sulphide is about 0.5% by weight. It shows up as peaks for presence of iron and sulphur in x-ray diffraction but cannot be seen directly in the micrographs.

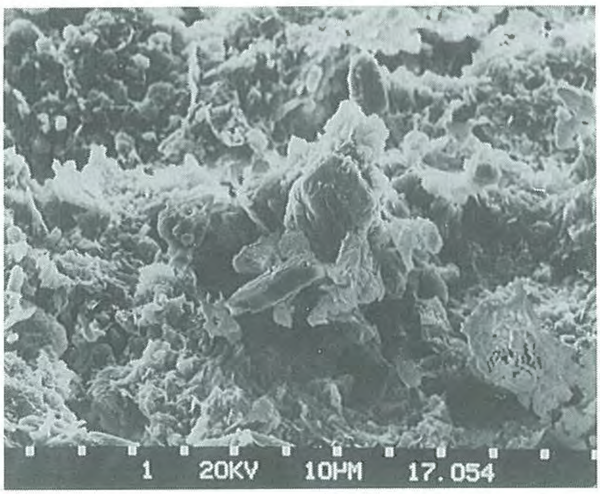
The initial permeability of the soil is only about $6 \cdot 10^{-10}$ m/s, which together with a low compression modulus gives minimum values of the coefficient of consolidation around $4 \cdot 10^{-9}$ m^2/s. This is in the order of one fifth of the normal values for soft inorganic clays.



a)

Fig. 13. Electron micrographs of Lilla Mellösa clay at a) 3.75 and b) 9 metres depth.





b)

8.2 Välen

Välen is a test field in the southern part of the city of Gothenburg which has been used by Chalmers University in a number of studies of the geotechnical properties of organic clay. The field is located on the north side of the small stream "Stora ån" just above the point where it discharges its waters into the inner part of the Askim bay called Välen. In different projects in this field the preconsolidation pressure has been studied using full scale load tests (Sällfors 1975), the effect of various parameters in the field vane test has been investigated (Torstensson 1977) and full scale plate load tests have been performed to investigate the bearing capacity.

The soft soils at the site have a thickness of about 11 metres. The conditions in the area are fairly uniform, but some properties such as undrained shear strength and preconsolidation pressure vary strongly with depth. It is therefore difficult to correlate some of the investigations that have been carried out in different parts of the field. This is especially valid for the field vane tests which were comprehensive at two levels, but for which a continuous profile is missing. In this case they have mainly been replaced by results from fall-cone tests, which according to local experience give very similar results in this type of soil (Hansbo 1957, Sällfors 1975). A typical soil profile for the site is shown in FIG. 14.

The dry crust in the area is about 1 metre thick, but the effects of closeness to the ground surface and some root threads extend for another metre. The dry crust is followed by green-grey gyttja-bearing clay with an organic content of 5-6%. This layer extends to a depth of about 5.5 metres. From this level the content of shells rapidly increases and in large parts of the area there is a thin band classified as shell-soil. The lower layers of soft soil consist of silty clay.

The natural water content in the organic clay between 2.5 and 5.5 metres depth is about equal to the liquid limit and varies between 110 and 130%. The bulk density in the layer is about 1.4 t/m^3. In the layers lower down, the water content decreases and the density increases to become about 60 % and 1.7 t/m^3 respectively in the silty clay.

The pore pressure profile at the site is complex. The ground water level in the dry crust seems to vary seasonally from close to the ground surface to the bottom of the crust. In the deeper layers the pore water pressure is generally higher and corresponds more to a hydrostatic head close to the ground surface. Some measurements indicate a connection between the pore pressure in the band of shell soil and the water level in the stream, while measurements in other parts of the field do not show such effects.

Välen

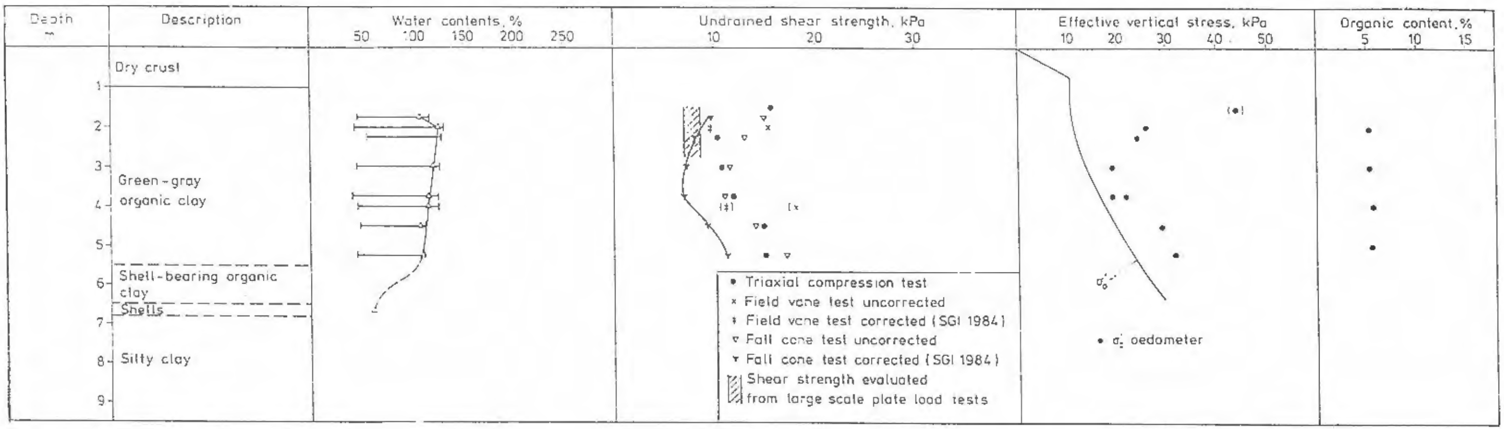


Fig. 14. Soil profile at the Välen test site.

The results from oedometer tests and the large scale field loading tests show that the soil is overconsolidated at the top because of dry crust effects. The overconsolidation ratio has a minimum of about 1.2 at a depth of 3.5 m and then increases further down. Exact values of the overconsolidation ratio further down, however, cannot be given because of the complex pore pressure distribution.

In the organic clay the preconsolidation pressure has been estimated by series of CRS-tests with different rates of strain and by large field loading tests. The results formed part of the basis for the method of interpretation of the preconsolidation pressure from this type of test suggested by Sällfors in 1975, which has been used in Sweden since then.

The undrained shear strength has been determined in fall-cone tests, series of large scale plate loading tests in the field and series of triaxial compression tests with different rates of deformation in the laboratory. The profile of undrained shear strength mirrors the preconsolidation profile and there is an almost perfect fit between the uncorrected strength values from the fall cone test and the Hansbo relation $\tau = \sigma'_c \cdot 0.45 \cdot w_L$. The sensitivity of the organic clay is 8 to 9.

Specimens of this organic clay have previously been examined in the scanning electron microscope (Larsson 1977), FIG. 15.

The micrographs show that in this soil there is an abundance of undecomposed organic fibrous material which clearly can be expected to affect the properties.

The initial permeability is around $1 \cdot 10^{-9}$ m/s and the minimum coefficient of consolidation is about $1 \cdot 10^{-8}$ m^2/s or about half of the normal values for inorganic clays.

8.3 Section 6/900

6/900 was a section in a planned location of the new "Särö road", i.e. road 158 between the cities of Gothenburg and Kungsbacka via Särö. The site is located in the valley of the stream "Stockaån" near the village Kyrkobyn. The new road was finally located away from this place and this new test site is still a cultivated area. Comprehensive sampling and field vane tests were made at the site for the present investigation and in a later investigation the pore water pressures were measured.

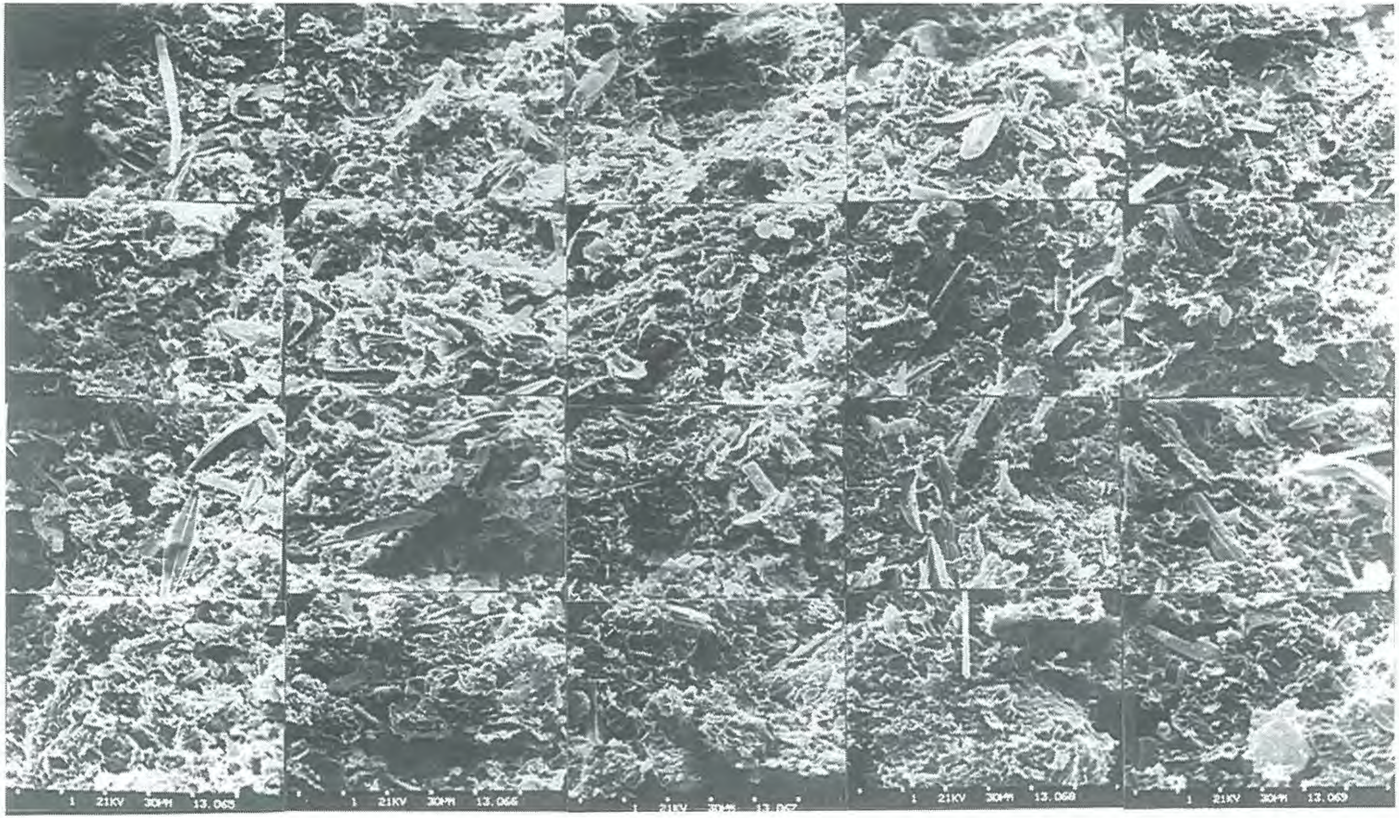


Fig. 15. *Serial micrograph of Välen organic clay.*

The ground in the valley is very flat and is intersected by the stream "Stockaån". The bottom of the stream lies about 2 m below the surrounding ground. The soft soil layers at the site are about 12 metres thick.

The upper part of the soil profile consists of a dry crust about half a metre thick, followed by another metre of stiff to medium stiff organic clay with root threads. Below this follows green-grey clayey gyttja with infusions of coarser plant remnants and shells down to a depth of 7.5 metres. In the depth interval between 5 and 7.5 m thin seams with more organic material can be observed. In the next metre of depth there is a gradual transition to a clay very rich in shells. The soft soil in the rest of the profile down to firm bottom consists of interchanging layers of silty clay rich in shells and clayey shell soils. FIG 16.

The organic content in the clayey gyttja is between 10 and 11%. The natural water content is slightly lower than the liquid limit, which vary between 150 to 175% and 180 to 205% respectively. The natural water content in the lower layers is only between 30 to 50%. The density in the clayey gyttja is 1.20 to 1.25 t/m^3 and varies between 1.5 and 2.0 t/m^3 in the lower layers.

The pore water pressures in the soil below 3-4 metres depth correspond to a water head 0.5 m below the ground surface, while the free ground water level in the uppermost soil varies seasonally from close to the ground surface to about 1.0 m below.

The undrained shear strength in the clayey gyttja has been determined in series of triaxial compression and extension tests, direct simple shear tests and fall cone tests in the laboratory and by field vane tests in the field. The sensitivity according to results from fall cone tests varies between 5 and 8.

The consolidation parameters have been determined in series of CRS-tests with different rates of deformation, in series of standard incremental tests and in tests with other loading procedures (Larsson & Sällfors 1985). Overconsolidation effects in and just below the dry crust extend to about 2.5 m depth. Further down, the overconsolidation ratio in the clayey gyttja is almost constant at 1.2. The horizontal stress has not been measured directly but the horizontal preconsolidation pressures have been estimated from CRS-tests on horizontally oriented specimens and from drained triaxial tests with increasing horizontal stresses. The results indicate a coefficient of earth pressure at rest for normally consolidated conditions K_{onc} around 0.57, which for the actual overconsolidation ratio in situ of 1.2 would give a K_0 of around 0.65 (see e.g. Larsson 1977).

Vallda - Kyrkbyn, Section 6/900

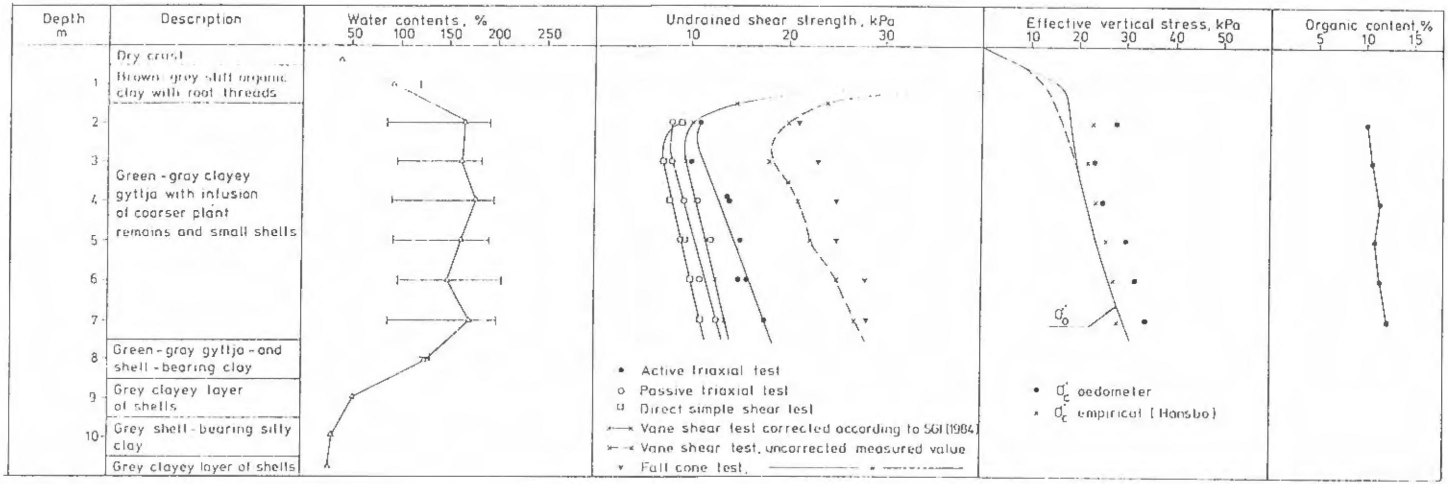


Fig. 16. Soil profile at section 6/900.

The initial permeability in the vertical direction is $2\text{-}5 \cdot 10^{-10}$ m/s. Between 2 and 4 metres depth the horizontal permeability is about 2 times higher and between 5-7 metres depth, where thin seams of more organic material can be observed, the horizontal permeability is about 7 times the vertical. The coefficient of consolidation at vertical water flow has a minimum of $2\text{-}4 \cdot 10^{-9}$ m^2/s, which is 5 to 10 times lower than corresponding values in soft inorganic clays.

The consolidation parameters evaluated from the oedometer tests are shown in FIG 17. The parameters are average values from all the CRS tests run at the standard rate of deformation and slower.

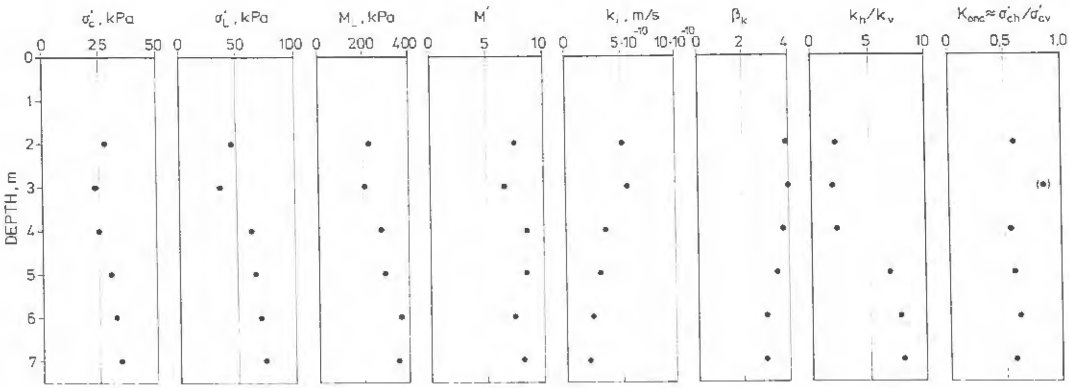
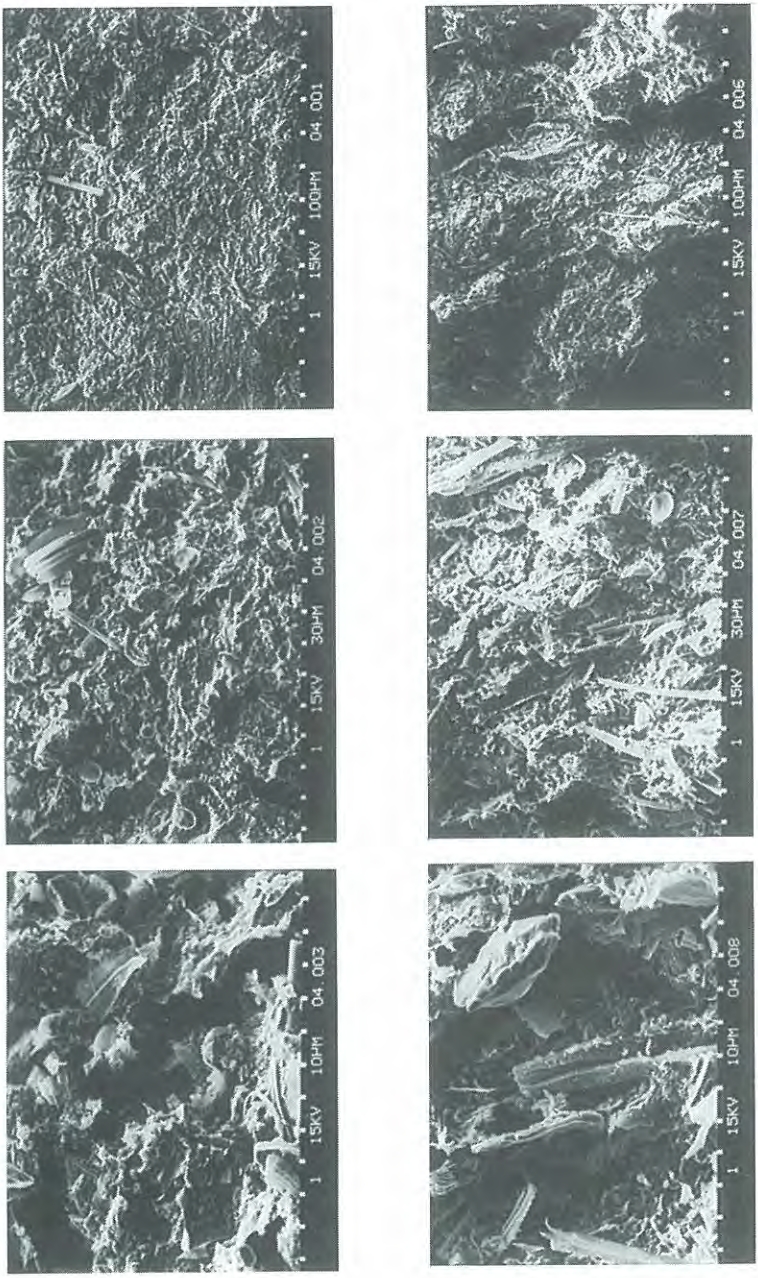


Fig. 17. Consolidation parameters evaluated from CRS-oedometer tests at section 6/900.

Freeze-dried samples were broken and the fracture surfaces were examined in the electron microscope. In order to obtain a good image of the surfaces in a scanning electron microscope, the surfaces of the specimens should be fairly smooth and plane. This was easily achieved when the specimens were broken to give horizontal fracture surfaces but was much more difficult for vertical fractures (see e.g. FIG. 27). For clays, there are special peeling techniques to obtain smooth surfaces, (Wong and Tovey 1975), but these cannot be applied to fibrous materials and to cut the surfaces would destroy the natural structure. The preferred horizontal orientation of the fibres in the soil was therefore often more obvious for the naked eye at preparation of the specimens than was later illustrated in the micrographs.

The micrographs were taken in series starting with a low magnification and then successively magnifying the central part of the area in order to show the texture of the soil both in detail and as a mass. Two such series using a specimen from a depth of 4 m are shown in Fig. 18. One series from a horizontal and one series from a vertical fracture



a) b)

Fig.18. Electron micrographs of specimens from 4 m depth in section 6/900; a) horizontal fracture surface b) vertical fracture surface.
The scales are shown at the bottom of the micrographs where the real distances between the white markers are given in µm.

surface are shown. The abundance of undecomposed fibrous organic material in the soil is evident from the pictures and in this case also the preferred horizontal direction can be seen by comparison of the two fracture surfaces. That both the fibres and their orientation can be expected to affect the soil properties is fairly obvious.

8.4 Section 7/600

7/600 is another section along the planned "Särö-road". It is located in the same valley close to the stream "Stockaån" about 700 metres further upstream from section 6/900. Topography, stratification and the properties of the soil are similar to those at the previous site, but the organic content in the clayey gyttja is somewhat lower (9-10%) and so are the natural water content and the liquid limit, FIG. 19.

The same type of investigations as at the previous site have also been made here. The natural water content and the liquid limit in the clayey gyttja vary between 135 to 155% and 155 to 170% respectively. The density is 1.30-1.35 t/m^3 and the sensitivity is between 7 and 9.

Pore pressure measurements and observations of the free ground water level give the same pore pressure distribution as at section 6/900. The overconsolidation effects in and just below the dry crust extend to 4 m depth and below this level the overconsolidation ratio remains constant 1.2 for the clayey gyttja. The coefficient of earth pressure at rest in the normally consolidated state, K_{onc}, is estimated to be around 0.55.

The initial permeability in the clayey gyttja is $2\text{-}6 \cdot 10^{-10}$ m/s and the minimum values of the coefficient of consolidation, c_v, are also here 5-10 times lower than corresponding normal values for soft inorganic clays. At this site thin seams of more organic material were observed in the soil between 2 and 4 metres below the ground surface. In this zone the horizontal permeability was found to be 2-3 times higher than the vertical. For the rest of the layer with clayey gyttja the corresponding relation was 1-2 times. The evaluated consolidation parameters from CRS-tests are shown in FIG. 20.

Freeze-dried samples have been investigated in the same way as the previous site in the microscope, FIG. 21. The same abundance of undecomposed fibrous material is found even if the preferred orientaion is not as obvious. In this case the magnifications have been brought further to also show the finer silt and clay particles that in the lower magnifications are more or less concealed by the larger organic particles.

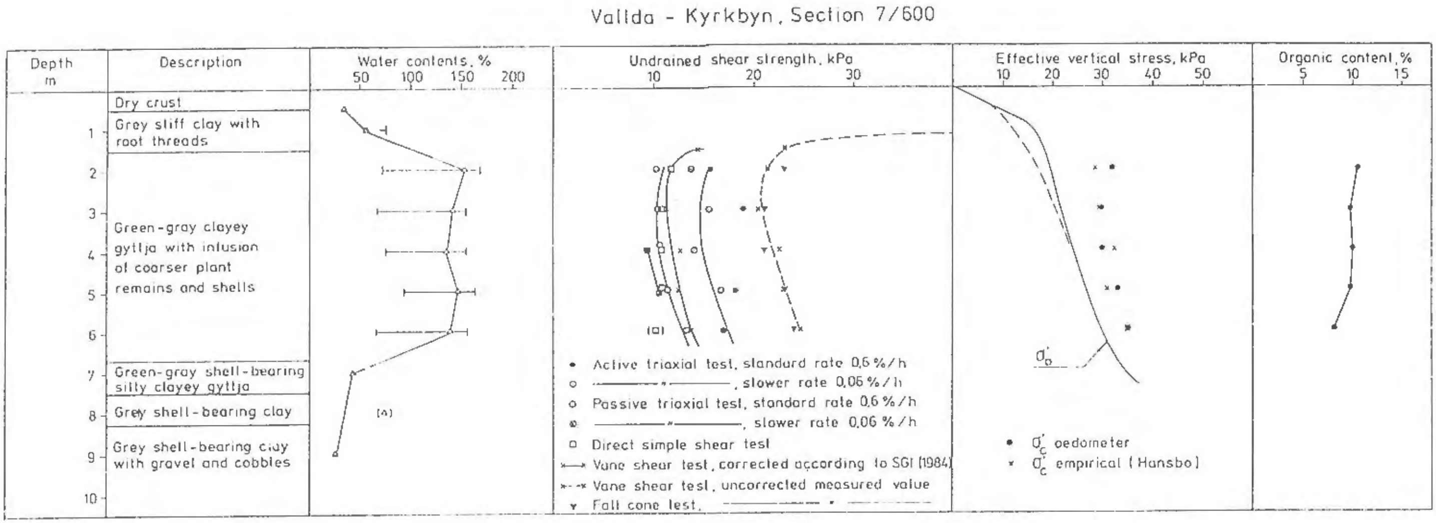


Fig. 19. Soil profile at section 7/600.

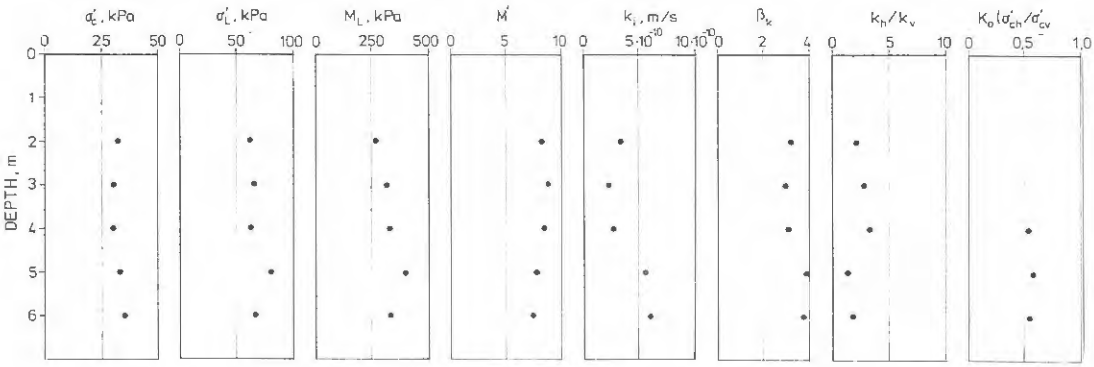
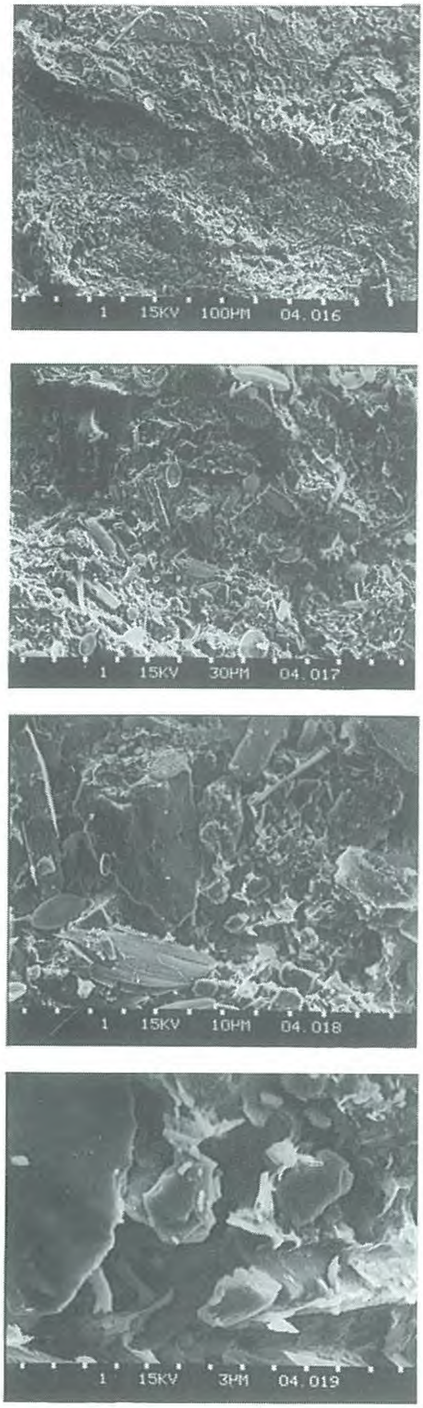
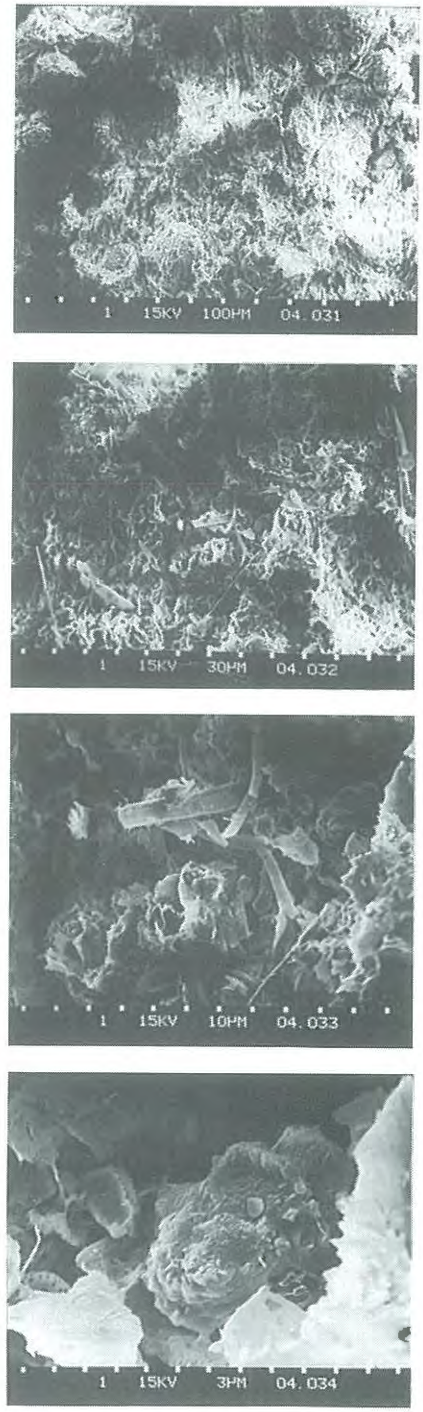


Fig. 20. Consolidation parameters evaluated from CRS-oedometer tests at section 7/600.

Fig. 21. Electron micrographs of specimens from 5 m depth at section 7/600; a) horizontal fracture surface, b) vertical fracture surface.


1 15KV 100PM 04.016
1 15KV 30PM 04.017
1 15KV 10PM 04.018
1 15KV 3PM 04.019

a)


1 15KV 100PM 04.031
1 15KV 30PM 04.032
1 15KV 10PM 04.033
1 15KV 3PM 04.034

b)

8.5 Kungsbacka

This new test site is located in a green open area in the city of Kungsbacka. The area is situated at the south side of the small river "Söderå" where this is crossed by the main railway along the Swedish west coast. On the other side of the site is the heavily used road "Gymnasiegatan" and the site was chosen from the results of a preliminary investigation for a tunnel for this road under the railway. The tunnel has not been built and the level crossing still remains, regulated by gates and signals.

The soil conditions resemble those in section 7/600 but in Kungsbacka a one metre thick fill has been acting on the soil for a long time and the ground water level is lower. These conditions have created higher preconsolidation pressures and undrained shear strengths than those usually found in this type of soil.

The layers of soft soils at the site are more than 20 metres thick. On top of the fill with gravel and stones there is a thin layer of top soil and a lawn. Under the fill lies the old dry crust, which is also about a metre thick. The following layer of main interest consists of green-grey clayey gyttja with infusions of shells. It is about 4.5 metres thick and extends to a depth of 6.5 m. Below this level there is a gradual transition to inorganic clay and from a depth of 8.5 m below the ground surface the soil is classified as grey clay with shells. FIG. 22.

The natural water content and the liquid limit in the clayey gyttja vary between 120 to 150% and 140 to 165% respectively. The organic content is 9-10%. At 8.5 metres depth the natural water content and the liquid limit have decreased to 90% and 80% respectively. The bulk density is around 1,30 t/m^3 in the clayey gyttja and increases to 1.5 - 1.6 t/m^3 in the clay further down.

The pore pressure was measured at 4 and 7 metres below the ground surface in June 1989 and the free ground water level in the crust was studied in an open hole down to 2 metres. The free ground water level was then found to be located about 1.9 m below the ground surface. The pore pressure profile was not hydrostatic, but the pore pressures measured at 4 and 7 metres depth showed water heads at the lower bound of the fill and at the ground surface respectively. The ground water conditions in the upper layers were probably affected by the dry conditions during that summer, but it is difficult to estimate "normal conditions". This is partly because of a possible effect of an existing tunnel for pedestrians in the vicinity and partly because it can be assumed that the soft soil layers still undergo consolidation for the applied fill and that excess pore pressures still exist.

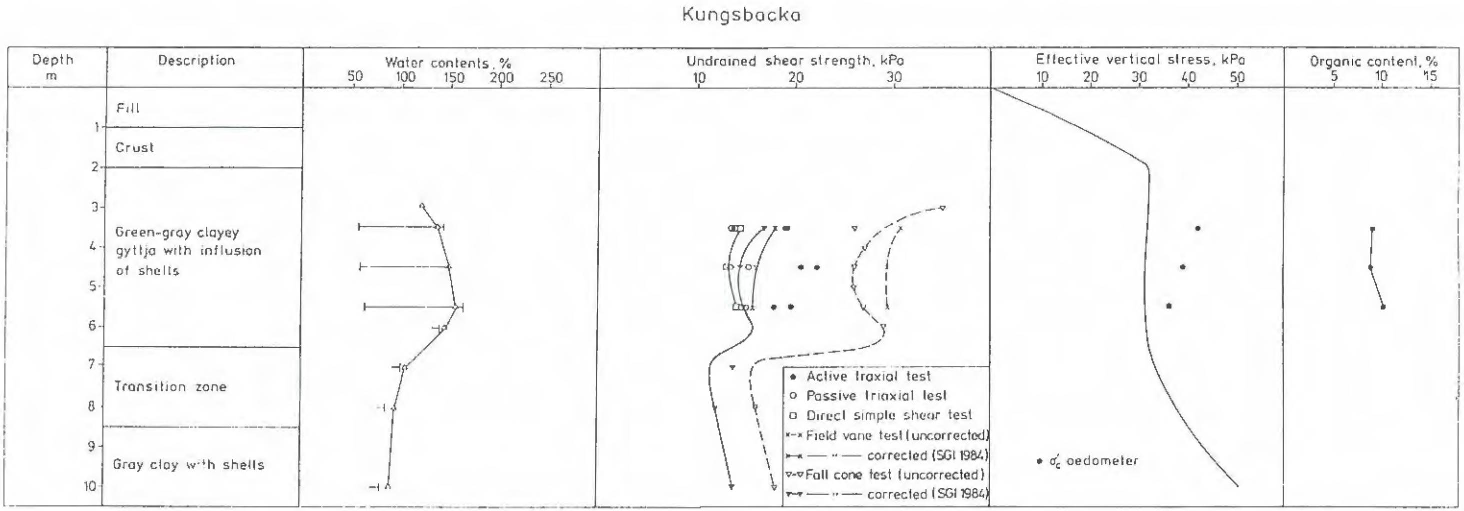


Fig. 22. Soil profile at the test site in Kungsbacka.

The same type of testing as in the other new test fields was performed. The tests showed that the clayey gyttja is slightly overconsolidated and that it has a considerably higher strength than the underlying almost normally consolidated inorganic clay. The sensitivity in the clayey gyttja is between 13 and 15 according to the fall cone tests. The coefficient of earth pressure at rest in the normally consolidated state K_{onc} estimated from oedometer tests and triaxial tests is around 0.57.

The vertical permeability of the clayey gyttja is $3\text{-}7 \cdot 10^{-10}$ m/s. The minimum coefficient of consolidation c_v amounts to about $5 \cdot 10^{-9}$ m^2/s, which is about 4 times lower than normal corresponding values in soft inorganic soils. The horizontal permeability was found to be 1.4-2 times the vertical.

The consolidation parameters evaluated from the CRS-tests on the clayey gyttja are shown in FIG. 23.

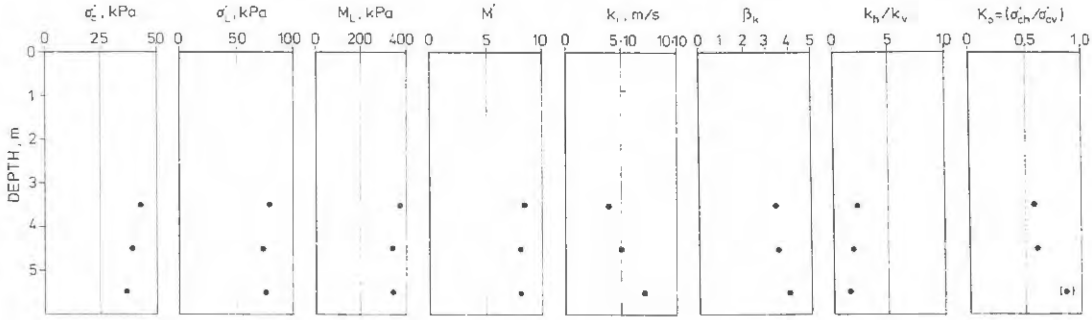


Fig. 23. Consolidation parameters evaluated from CRS-oedometer tests for the soil at the test site in Kungsbacka.

Scanning electron micrographs were taken on vertical and horizontal fracture surfaces in freeze-dried specimens. The micrographs show an abundance of undecomposed fibrous organic material with a preferred horizontal orientation. In the larger magnifications also some of the silt and clay particles become visible. The inorganic material still constitutes most of the solid material in the soil, but it is quite obvious that the organic matter may be assumed to have a strong effect on the properties, FIG. 24.

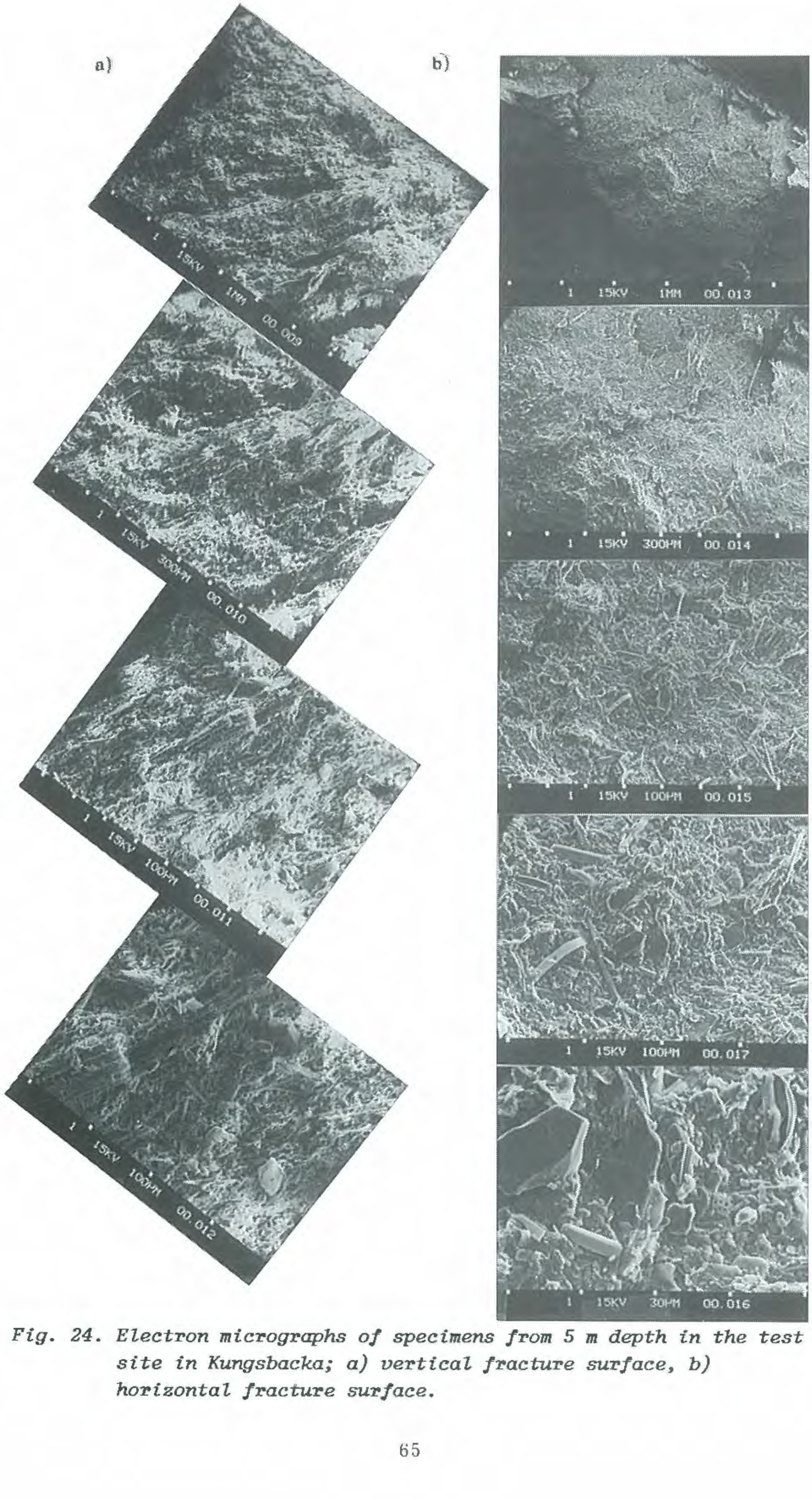


Fig. 24. Electron micrographs of specimens from 5 m depth in the test site in Kungsbacka; a) vertical fracture surface, b) horizontal fracture surface.

8.6 Kristianstad

The new test site in the city of Kristianstad in southern Sweden is located at the "Hammarsjö field" in the southern part of the city. It is near the point where the small river "Helgeån" discharges its waters into a lake, "Hammarsjön". Comprehensive investigations were carried out in this area when the new motorway on E66 bypassing Kristianstad was planned. Because of the very compressible soils and the very low undrained shear strengths the road was constructed with vertical drains and very wide pressure berms had to be laid out to ensure stability. This site was selected because it has a fairly homogeneous layer with high organic content and a thickness of about 4 metres. The organic content is above the limit for classification of the soil as "mineral organic". This organic soil is also interesting because it is a dy-bearing gyttja. Because it is brown-black and contains a large proportion of highly decomposed dy matter, it has sometimes been mistaken for amorphous peat.

The area is very flat. The test site is located outside the zone affected by the pressure berms at the border between the "road area" with vegetation consisting of grass and small bushes and the cultivated area outside. The soil profile consists of about half a metre of topsoil and dry crust on top of 4 m of brown-black dy-bearing gyttja. Between 4.5 and 5.5 metres depth there is a rather abrupt transition to grey clay. The exact depth of this transition varies somewhat in plane. The soft clay continues to more than 8 metres depth. Below this level there are no available results from investigations, FIG. 25.

The natural water content and the liquid limit in the gyttja varies between 200 to 400% and 260 to 410% respectively. The organic content varies between 27 and 38%. In the clay further down, the water content and the liquid limit are about 70 and 60% respectively. The bulk density is between 1.08 and 1.18 t/m^3 in the gyttja and in the clay it is between 1.55 and 1.70 t/m^3.

The pore pressure was measured at 3 m depth and the ground water level was observed in a 1.5 m deep open hole in July 1989. The free ground water level during this extremely dry period was located 1.4 m below the ground level. The pore pressure measurement showed that the pore pressure profile was not hydrostatic but the water head at 7 m depth was 0.7 m below the ground surface. Normal ground water conditions in this low lying area relatively close to a large lake may be assumed to be approximately hydrostatic for a ground water level around 0.5 m below the ground surface.

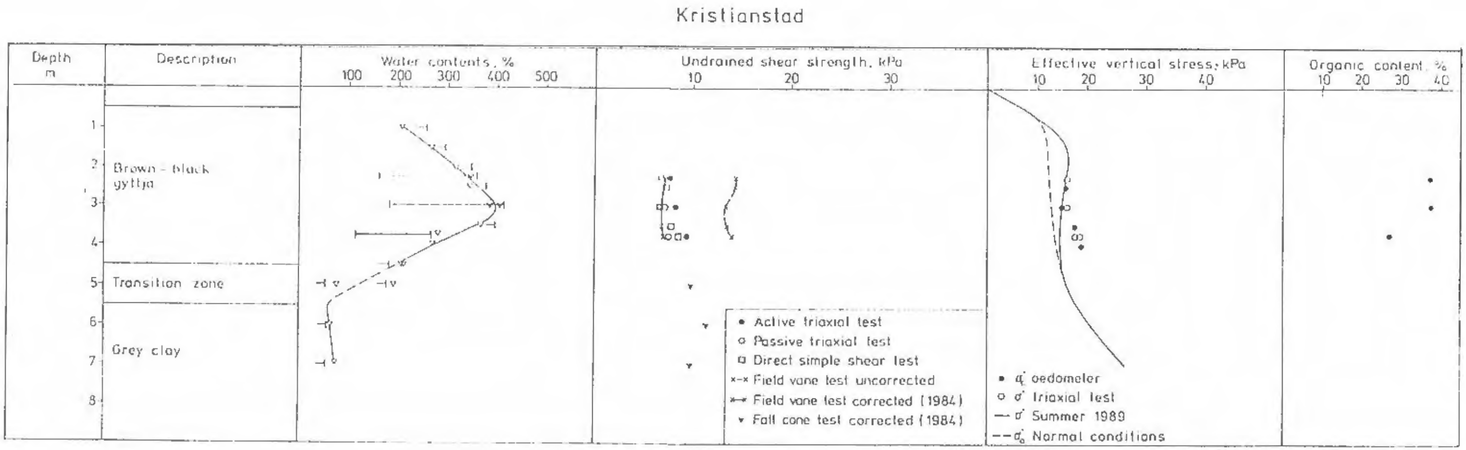


Fig. 25. Soil profile at the test site in Kristianstad.

The same testing programme as for the other new test fields was performed for the gyttja, even if the very low effective in situ stresses and shear strengths created special problems in the laboratory tests. The results, however, were in good agreement, both with each other and in comparison with the field tests and the in situ conditions.

The results from the oedometer tests and the triaxial tests show that the gyttja in its upper part is normally consolidated for the extreme conditions in July 1989 and that the entire layer of gyttja may be assumed to have an overconsolidation ratio of 1.3 in more normal conditions.

The permeability of the soil is $1\text{-}3 \cdot 10^{-9}$ m/s and the minimum coefficient of consolidation c_v is about $5 \cdot 10^{-9}$ or about a fourth of the corresponding normal value for soft inorganic clays. The horizontal permeability is about 1.5 times the vertical.

The coefficient of earth pressure at rest in the normally consolidated state estimated by oedometer tests had an average of 0.52. The scatter was larger than usual because of the small values of the compared preconsolidation pressures in the vertical and horizontal directions. The sensitivity in the gyttja varies between 6 and 9.

The consolidation parameters for the gyttja determined in the CRS tests are shown in Fig. 26.

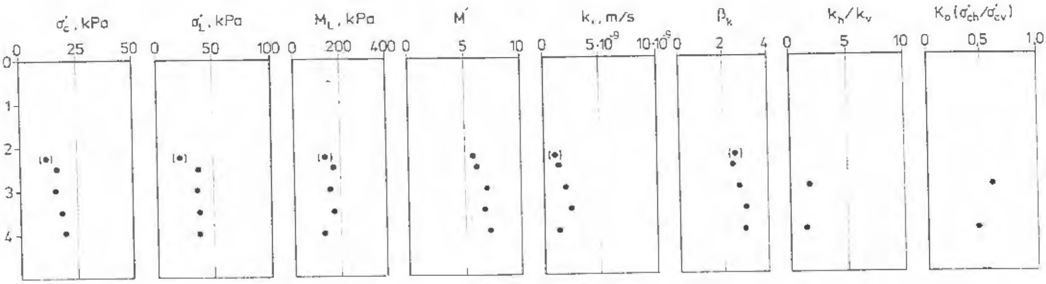
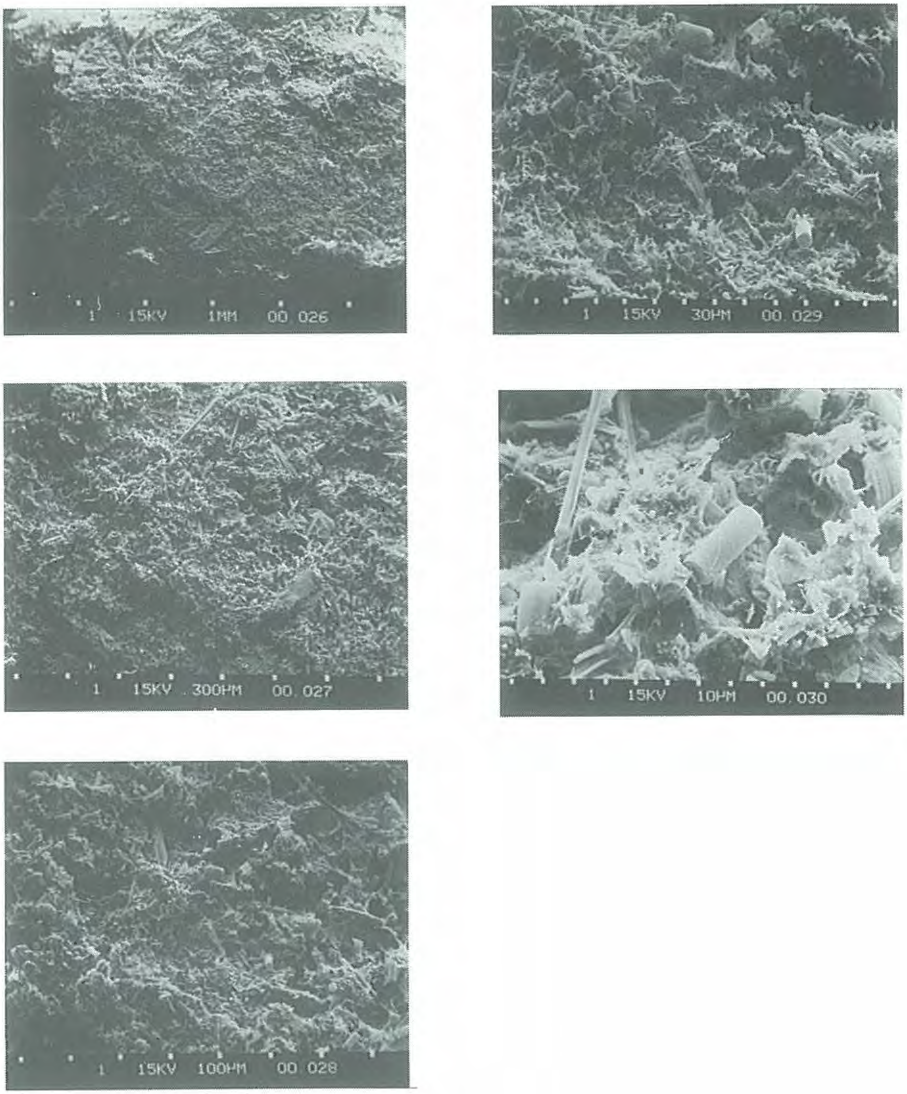


Fig. 26. Consolidation parameters evaluated from CRS-oedometer tests at the test site in Kristianstad.

Specimens of the soil have been freeze-dried to be examined in the microscope. On normal drying this type of soil shrinks to fractions of its original volume and forms small, hard lumps. At freeze-drying the shrinkage is very moderate if any. When the samples are broken to give clear fracture surfaces for examination the vertical fracture surfaces become very uneven because of a preferred horizontal orientation of the fibres in the soil, FIG. 27.

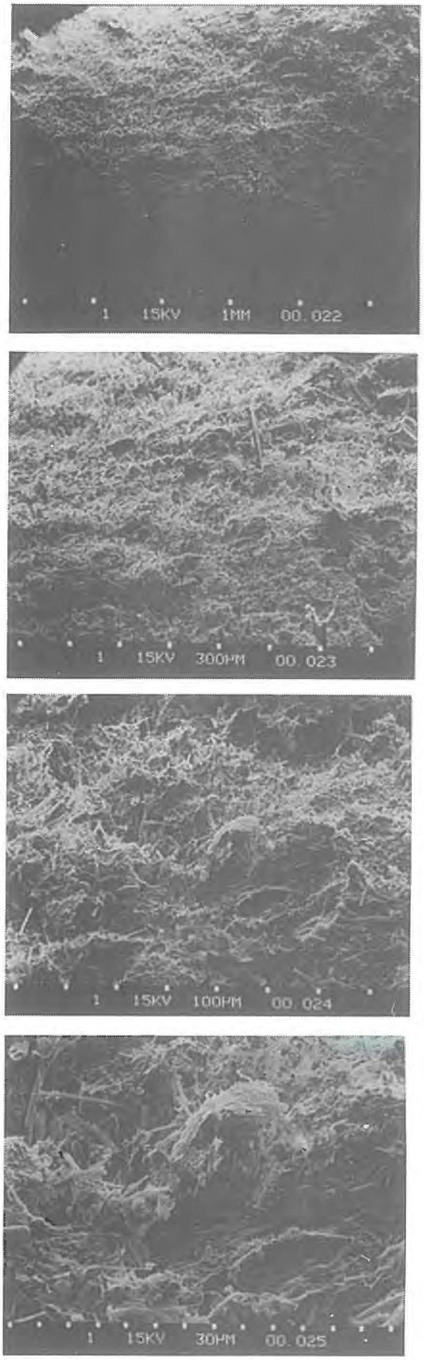
Fig. 27. Photograph of a freeze-dried sample broken to give a vertical fracture surface. The original diameter of the sample was 50 mm.

The uneven fracture surfaces make it difficult to study the texture of the soil in the vertical plane. This is illustrated in the micrographs in FIG. 28, where hardly a single plane area can be found to study in the vertical fracture surface. In the horizontal plane, on the other hand, the soil splits on fairly smooth surfaces where the characteristics of the texture can be studied. In this dy-bearing gyttja there is also an abundance of organic material and fibres, but the organic material in general can be seen to be more decayed and the fibres more naked than in the previously shown gyttjas. The magnifications are rather low, but in the largest magnifications the presence of silt particles can also be seen, FIG. 28.

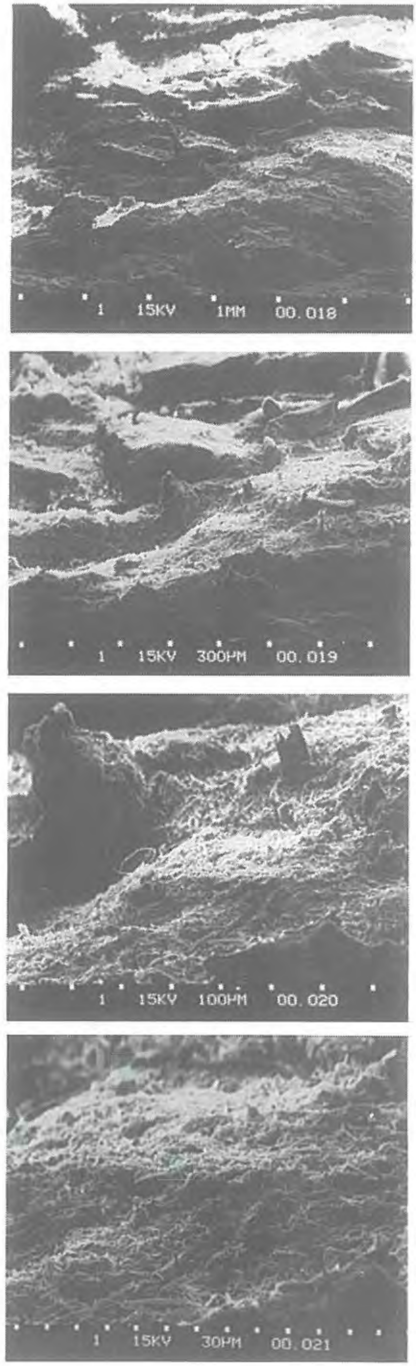


a)

Fig. 28. Electron micrographs of specimens from 3 m depth at the test site in Kristianstad; a and b) horizontal fracture surfaces, c) vertical fracture surface.


1 15KV 1MM 00.022
1 15KV 300UM 00.023
1 15KV 100UM 00.024
1 15KV 30UM 00.025

b)


1 15KV 1MM 00.018
1 15KV 300UM 00.019
1 15KV 100UM 00.020
1 15KV 30UM 00.021

c)

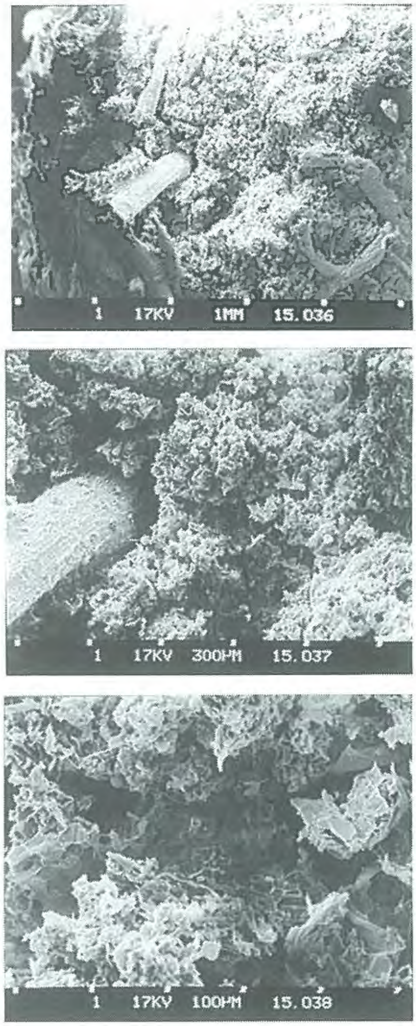


Fig. 29. Electron micrographs of an amorphous peat.

For comparison, a corresponding series of micrographs from an amorphous peat is shown in FIG. 29.

The amorphous peat was sampled from 2 metres depth along road 50 between the cities of Motala and Askersund. It has a density of 1.02 t/m^3 and a natural water content of 760%. In the pictures it can be seen that the soil consists of some coarser and more intact fragments of wood surrounded by a decomposed amorphous mass of organic origin.

9. SHORT PRESENTATION OF SOILS IN RELATED INVESTIGATIONS USED AS SUPPLEMENTS

9.1 Organic soils in Finland

During the years 1979-1982 the Geotechnical laboratory at the Technical Research Centre of Finland conducted investigations on the influence of organic content on the shear strength of fine grained soil. The main purpose of the investigation was to improve the knowledge of the applicability of the usual correction methods for strength values obtained in field vane tests.

Five different sites with organic clays and clayey organic soils were selected. The soil layers that were studied had organic contents varying from less than two to 13-15%. All of them had been deposited in brackish water during the Littorina stage of the Baltic Sea.

Comprehensive series of field vane tests and triaxial compression test with different rates of deformation were performed in addition to direct shear tests, routine tests, oedometer tests and analyses for geological description of the soils. The organic contents have been determined in accurate analyses. The results have been presented by Slunga (1983) and a more detailed report was given by the Geotechnical laboratory of the Technical Research Centre of Finland in 1982.

The soils in the Finnish investigation may be assumed to be very similar to the soils in the present Swedish investigation. The layers studied in detail had the following properties; TABLE 3.

These soils were also similar to the investigated Swedish soils in that they had preconsolidation pressures mainly in the range 10 to 40 kPa and undrained shear strengths in the range 4 to 16 kPa.

One of the major conclusions from the investigation was that the undrained shear strength in these soils could be estimated by field vane tests corrected by the normally used methods, among them Helenelund's (1977) method. This method is very similar to the method now used in Sweden (Larsson et al, 1984).

Table 3. Properties of the soils studied in the Finnish investigation 1979-1982. (From Slunga 1983)

Site	Organic content %	Water content %	Liquid limit %	Plasticity index %	Classification
Raastala depth 5.25 m	<2	120	73	40	clay
Raastala depth 2.75 m	5-6	140	109	70	organic clay
Seinäjoki	3-4	61-76	64-76	40	organic clay
Siuntio	8-9	155	153	103	clayey organic soil
Vanhankaupunginlahti	7-8	130	130	100	clayey organic soil
Lokalahti	13-15	234	246	162	clayey organic soil

The test results gave detailed information on the rate effects on the evaluated undrained shear strengths. Also rate effects on the stress-strain behaviour in shear tests and the influence of the organic content on the shear strains at failure can be studied from the results, FIG 30.

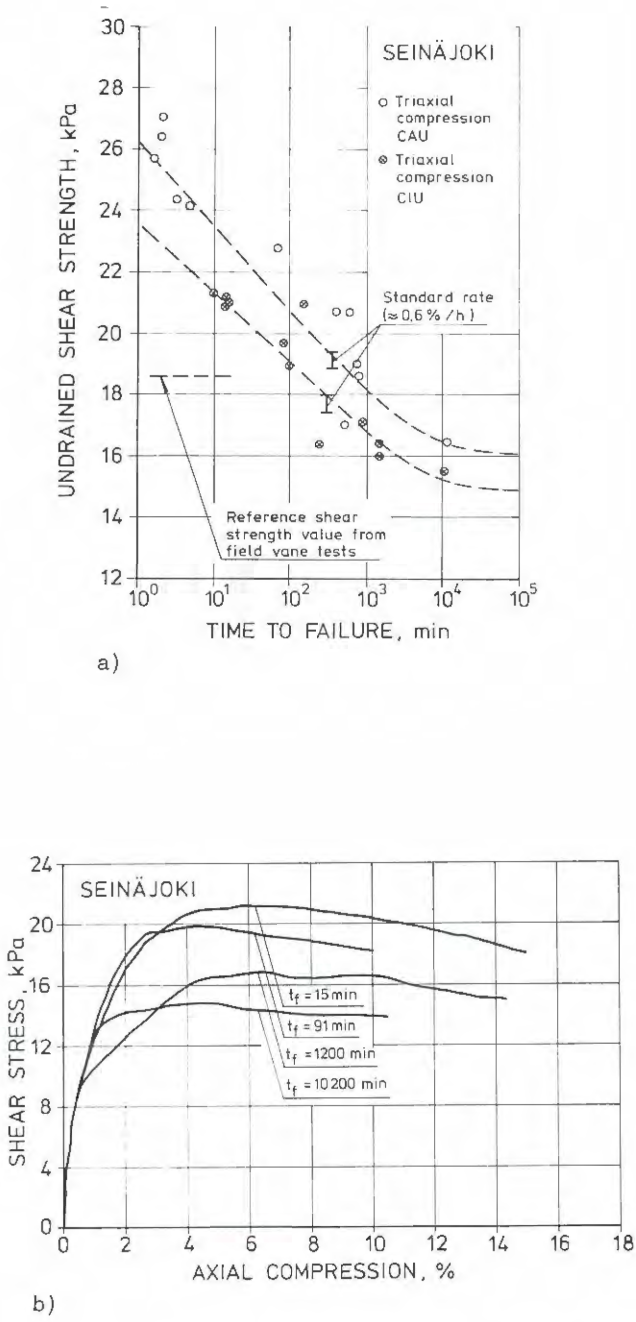


Fig. 30. Rate effects in undrained triaxial compression tests on a) evaluated undrained shear strength and b) the stress-strain relations. (Data from Slunga 1983)

9.2 Sulphide soils in Sweden

The investigations by Schwab (1976) mainly concerned the bearing capacity, shear strength and deformation behaviour of soft organic sulphide soils. Three different sites with "svartmocka" along the Gulf of Bothnia were selected. When more careful determinations of the organic contents were made, it turned out that in only one of the sites should the soil be classified as organic, (organic silty sulphide clay). Also in this site the organic content was less than half of what had previously been estimated from loss on ignition.

At the three sites sampling, field vane tests, screw plate tests and large scale plate loading tests were carried out. In the laboratory routine tests and undrained triaxial and direct simple shear tests were performed.

At two of the sites also test embankments were built to failure; one in Schwab's project and one conducted by SGI on commission by the Swedish Road Administration (Holtz & Holm, 1973).

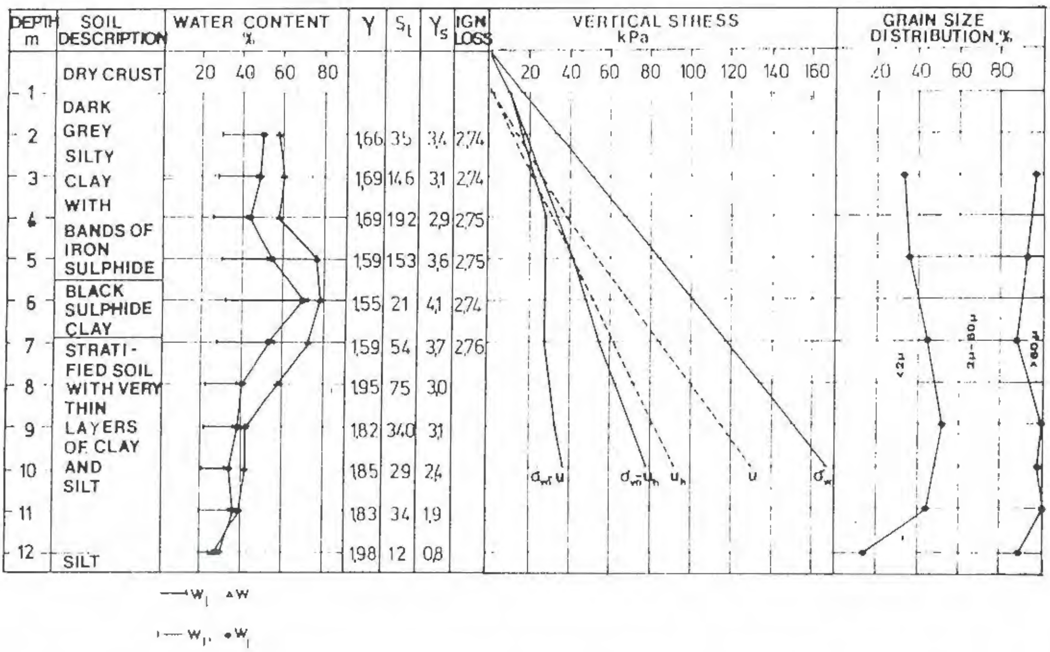
The three sites were located at Kramfors, Umeå and Kalix.

- <u>Kramfors</u>

The test site was located at Dynäs, about 8 km north of the city of Kramfors near the coast of the Gulf of Bothnia. The site is very level with an elevation of 6 metres above sea level. The area emerged from the sea because of the land heave about 800 years ago. The current rate of land heave is about 8 mm/year.

The soil in the profile consist of 12 metres of soft fine grained soils on top of 2 metres of silt on bedrock, FIG. 31.

The dry crust is about 1 m thick. Below this there is dark grey silty clay with black bands richer in sulphides down to a depth of 5.5 metres. Between 5.5 and 7 metres depth there is a layer of black sulphide clay and below this the soil becomes varved with interchanging silt and clay seams. From 10 metres depth the soil mainly consists of silt with thin clay layers.



Depth m	γ	S_t	$γ_s$	IGN LOSS
2	1,66	35	3,4	2,74
3	1,69	14,6	3,1	2,74
4	1,69	192	2,9	2,75
5	1,59	153	3,6	2,75
6	1,55	21	4,1	2,74
7	1,59	54	3,7	2,76
8	1,95	75	3,0	
9	1,82	34,0	3,1	
10	1,85	29	2,4	
11	1,83	34	1,9	
12	1,98	12	0,8	

Fig. 31. Soil profile in the test site at Kramfors. (From Schwab 1976)

The natural water content is about 50% in the silty clay and increases to about 75% in the sulphide clay. In the varved clay and silt soil it gradually decreases with depth. In this soil profile the liquid limit as determined both by fall cone and by the Casagrande apparatus is about 10% lower than the water content. The plasticity index in the silty clay is only about 20% and the liquidity index as high as 1.5. The sensitivity is also very high in this soil but it is more moderate in the sulphide clay where the plasticity index is higher. The bulk density varies from about 1.7 t/m^3 in the silty clay to 1.55 in the sulphide clay and 1.9 in the varved soil further down.

The loss on ignition for the soil between 2 and 7 metres depth had averages around 2.8% with values up to 4% in the sulphide clay. This closely corresponds to the empirical reduction that should be applied to such values in fine grained soil with the clay contents found (Larsson et al 1985). The soil may therefore be assumed to be inorganic.

The free ground water level in the dry crust was observed at a depth of 0.8 metres. The pore pressure profile was not hydrostatic but an artesian water head of 6 metres above the ground surface was observed in the silty layer just above the bedrock.

The undrained shear strength as determined by routine tests was in the order of 15 kPa. The soil is reported to be slightly overconsolidated down to a depth of 5 to 6 metres but no data are given.

The results from the determinations of the shear strengths in Kramfors are shown in FIG. 32.

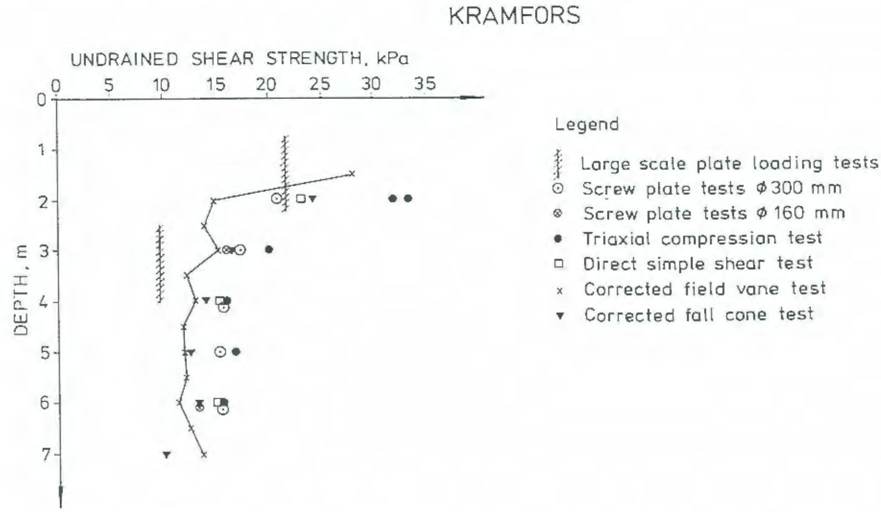


Fig. 32. Undrained shear strength in the soil profile at Kramfors as determined by various test methods. (Data from Schwab 1976)

The comparisons betweeen the large scale field tests and the small scale field and laboratory tests are somewhat uncertain. The large plate load tests at 2.5 metres depth showed a relatively low shear strength but Schwab stresses the difficulty in excavating and performing these tests in such a way that this highly sensitive (quick) silty clay remains undisturbed.

The stability calculations for the test embankment yielded safety factors that deviated strongly depending on whether the most critical or the very roughly assumed failure surface was considered and whether end effects were accounted for or not.

The screw-plate tests, which are intermediate in size, yielded results that were close to the direct simple shear test results and somewhat higher than the corrected field vane and fall cone test results. From the correlations at this site, the correction factor to be applied to the field vane test may be assumed to be between 0.8 and 1.2, as compared to about 0.95 that would normally be used in soils with corresponding liquid limits.

• Umeå

The test site is located at Rödäng about 3 km northwest of the city of Umeå near the Gulf of Bothnia. The area is level and the elevation is about 9.5 m above sea level. The current rate of land heave in this area is about 9 mm/year and the test area emerged from the sea around 1000 years ago.

The sediments at the site are about 14 metres thick. The desiccated dry crust is about 1.5 m thick and the soil below consists of black silt with thin layers of fine sand. Below 10-11 metres depth the sediments become coarser and there is a sand layer on top of the bedrock, FIG. 33.

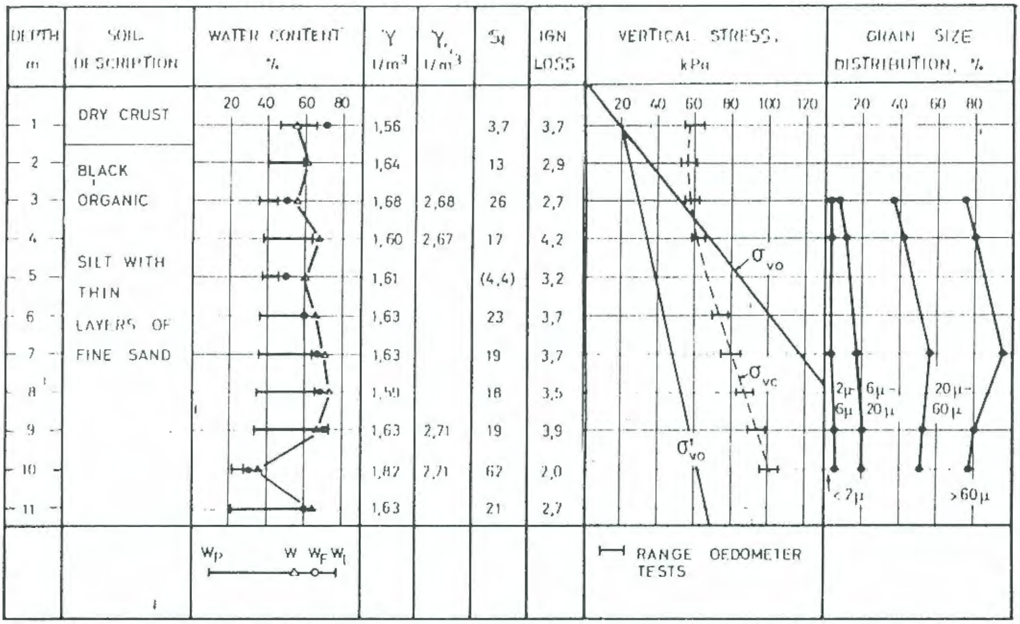


Fig. 33. Soil profile at the test site at Umeå. (From Schwab 1976).

The natural water content in the black silt is 60-70% and the liquid limit as determined by the fall cone method and by the Casagrande method is somewhat lower. The sensitivity of the soil as determined by fall cone tests is 13 to 26 and the bulk density is around 1.6 t/m^3.

The clay content in the soil is only 3 to 5%. The ignition loss varies between 2 and 4% with an average of 3.3%. After correction this would correspond to an organic content of just over 2%. The organic content was also determined by a more accurate wet combustion method and was then found to be about 2% for the whole profile. The soil in the profile is thus on the border to becoming the prefix "organic" to the main classification which is "silt".

The free ground water level was found to vary between 0.5 and 1.5 m below the ground level. The pore pressure profile was not hydrostatic but the pore pressure in the bottom sand layer was found to be slightly artesian.

According to the oedometer tests, the soil has an overconsolidation of about 1.7 for the entire profile below the dry crust. It has never been subjected to any load other than its own weight, natural variations in ground water levels and snow loads.

The undrained shear strengths obtained in the routine fall cone tests and the field vane tests were fairly uniform around 22 kPa. The shear strength values obtained from both types of tests, however, were considerably higher than what is normal for soils with corresponding preconsolidation pressures and consistency limits.

The results from the triaxial tests and the direct simple shear tests were considerably lower, as were the results from the screw-plate tests and the large scale plate loading tests, FIG. 34.

The results from Umeå thus showed that in this soil the usual correction factors for the results from field vane and the fall cone tests were not sufficient, but the strength values had to be further reduced.

Results from triaxial test with varying rates of deformation also showed that the evaluated undrained shear strength was about 85% of the original value if the standard rate of deformation of 0.6%/hour was lowered by 1 to 2 decades.

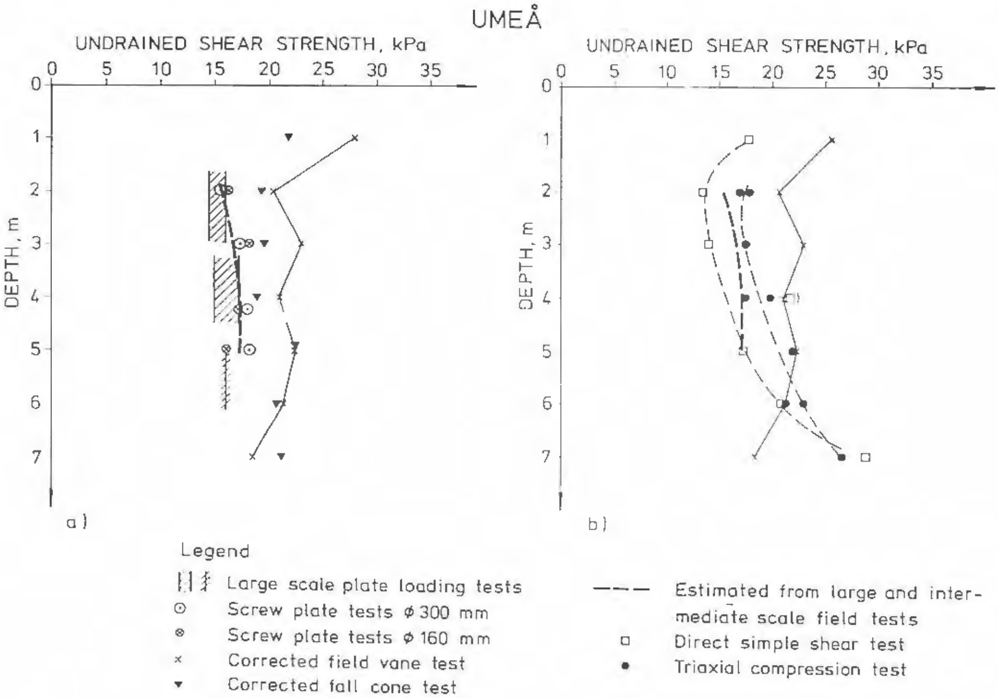


Fig. 34. Undrained shear strength in the soil profile at Umeå as determined by a) field tests and routine tests and b) field and laboratory tests.

- Kalix

The test site is located north of the Kalix River about 6 km northwest of the community of Kalix. The area slopes slightly towards the south and the river. The area lies close to the northern end of the Gulf of Bothnia and emerged from the sea about 300 years ago. The rate of land heave in this area is 9 mm/year and the elevation of the ground surface in the test site is between 2.5 and 3.0 m above sea level.

The soil consists of about 18 metres of black organic silty clay on top of fine sand and till. The depth to bedrock is 24 metres.

The soil investigations carried out by Schwab (1976) are concentrated to the upper 6-8 metres of the deposit. The investigations made by Holtz and Holm (1973) about 50 metres away extend to 13 metres depth but are not as extensive, FIG. 35.

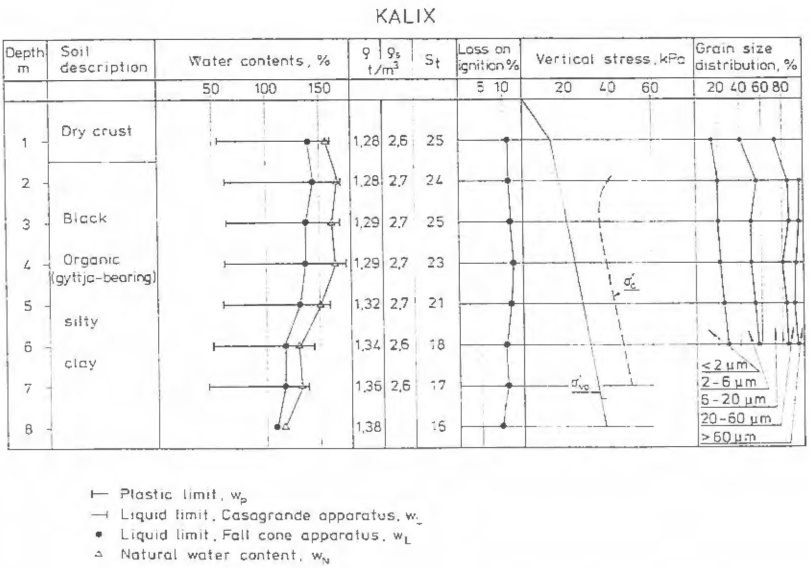


Fig. 35. Soil profile at the test site at Kalix. (From Schwab 1976 and Holtz and Holm 1973)

The dry crust is about 1.5 m thick and the ground water level varies between 0.5 and 1.5 metres below the ground surface. The pore pressures further down are mainly hydrostatic for a ground water level 0.9 metres below the ground surface.

The natural water content in the clay is about 160% down to 4 metres depth and then decreases with depth and is about 100% at 12 metres. The liquid limit as determined by the fall cone method is about 20% lower in the upper part of the profile and then gradually approaches the water content and becomes equal to it at about 12 metres depth. The liquid limit as determined by the Casagrande apparatus is somewhat higher than the natural water content for all of the investigated parts of the profile. The bulk density gradually increases from 1.3 t/m^3 just below the crust to 1.4 t/m^3 at 12 metres depth and the sensitivity as determined by fall cone tests is between 10 and 25.

The clay content in the soil is 20% or slightly more, which means that the soil is close to the border of being classified as a clayey silt instead of a silty clay. The loss on ignition for the soil in the upper part of the profile was 11-12% (9-10% according to Holtz and Holm 1973). After correction for the clay content this would normally indicate an organic content of about 9%. Determinations with more accurate wet combustion methods on samples from the same site by Pusch (1973) showed an organic content of 4.1%. This is only half of the previously estimated value and changes the accurate classification from a clayey gyttja to an organic clay. Also other investigations on "svartmocka" in this area have shown that the organic content is generally much lower than was previously believed (Jerbo 1970, Nystrand 1970 and 1980).

The undrained shear strength determined by fall cone and field vane tests has a minimum of about 8 kPa at 2-3 metres depth and then increases with depth by about 1 kPa/m.

The soil is slightly overconsolidated, although it has only been subjected to the stresses from its own load, natural variations in ground water level and snow loads. The overconsolidation ratio slowly decreases with depth in the investigated upper part and is about 1.6 at 7 metres depth.

The results from the other shear strength tests showed a fairly good agreement between corrected field vane tests and direct simple shear tests. When compared to the larger field tests, the screw plate tests gave higher strengths, while two of the large plate load tests gave strengths compatible with the corrected field vane tests and the direct simple shear tests. The third plate load test performed at a depth of 3.5 m gave somewhat lower strength values, but Schwab reports problems with tilting of the plate at an early stage of this test, FIG. 36.

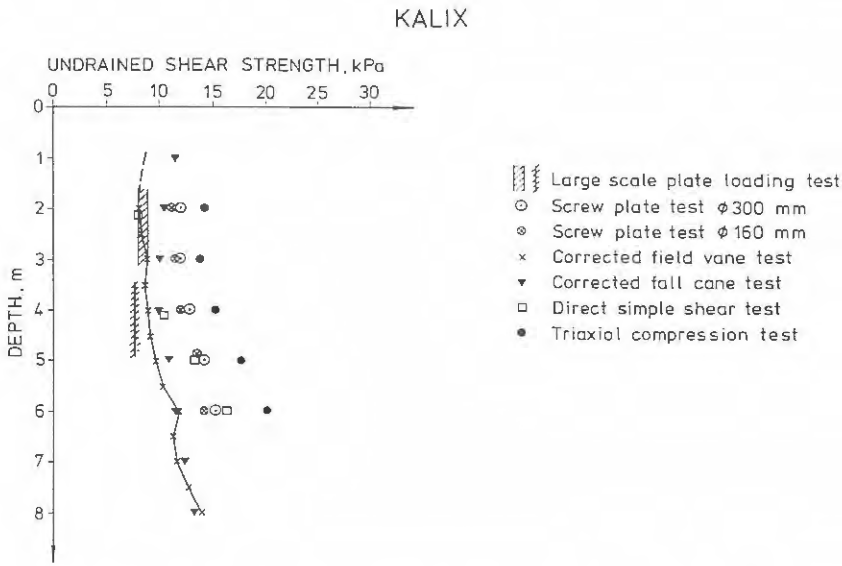


Fig. 36. Undrained shear strength in the soil profile at Kalix as determined by different methods. (Data from Schwab 1976 and Holtz and Holm 1973)

The results from stability analyses of the test embankment built to failure (Holtz and Holm 1973) indicate that the shear strength is only slightly higher than the corrected vane shear strength. The combined results from Kalix indicate that the normally used correction factor for field vane tests can be used in this type of soil and that also the results from direct simple shear tests are directly applicable in stability calculations.

- Gideåbacka

Gideåbacka is a place situated close to the Gulf of Bothnia about 20 km northeast of the city of Örnsköldsvik. In 1987 a landslide occurred here along the river "Gideå älv" involving about 200 000 m^3 of soil. The Institute was then commissioned by the community to investigate the slide area and adjacent parts of the river bank (Hermansson 1990).

The soil in the area typically consists of about 2 metres of fine sand with some organic matter on top of about 10 metres of mainly black organic silt or clayey organic silt. In a layer between 8 and 10 metres depth the clay content is slightly higher than 20%, which changes the classification to a silty organic clay. The liquid limit is just above the water content, which varies between 50 and 60% in the silt and 60 and 70% in the clay. In some layers the sulphide content is lower and the soil there is grey or has the typical green-grey colour of gyttja-bearing soil. The organic content of the soil is about 4% in the organic silt and 5 to 6% in the silty clay. The sulphide content is about 2%. An average profile is shown in FIG. 37.

The bulk density in the soil is around 1.6 t/m^3. It is slightly lower in the clay and slightly higher in the silt further down. The sensitivity varies between 10 and 30.

In the field investigation field vane tests, pore pressure measurements, piezocone soundings and sampling were carried out. In the laboratory routine tests, oedometer tests and direct simple shear tests were performed.

The pore water pressures in the soil roughly correspond to the hydrostatic pressure at a ground water level 1 metre below the ground surface, except for the coarser bottom layers where the pore pressures are about 20 kPa lower and seem to be more closely connected to the water level in the river.

The undrained shear strength values obtained in the field vane tests were high as were the results from the fall cone tests in the more clayey layers where measurements were also made with this type of test.

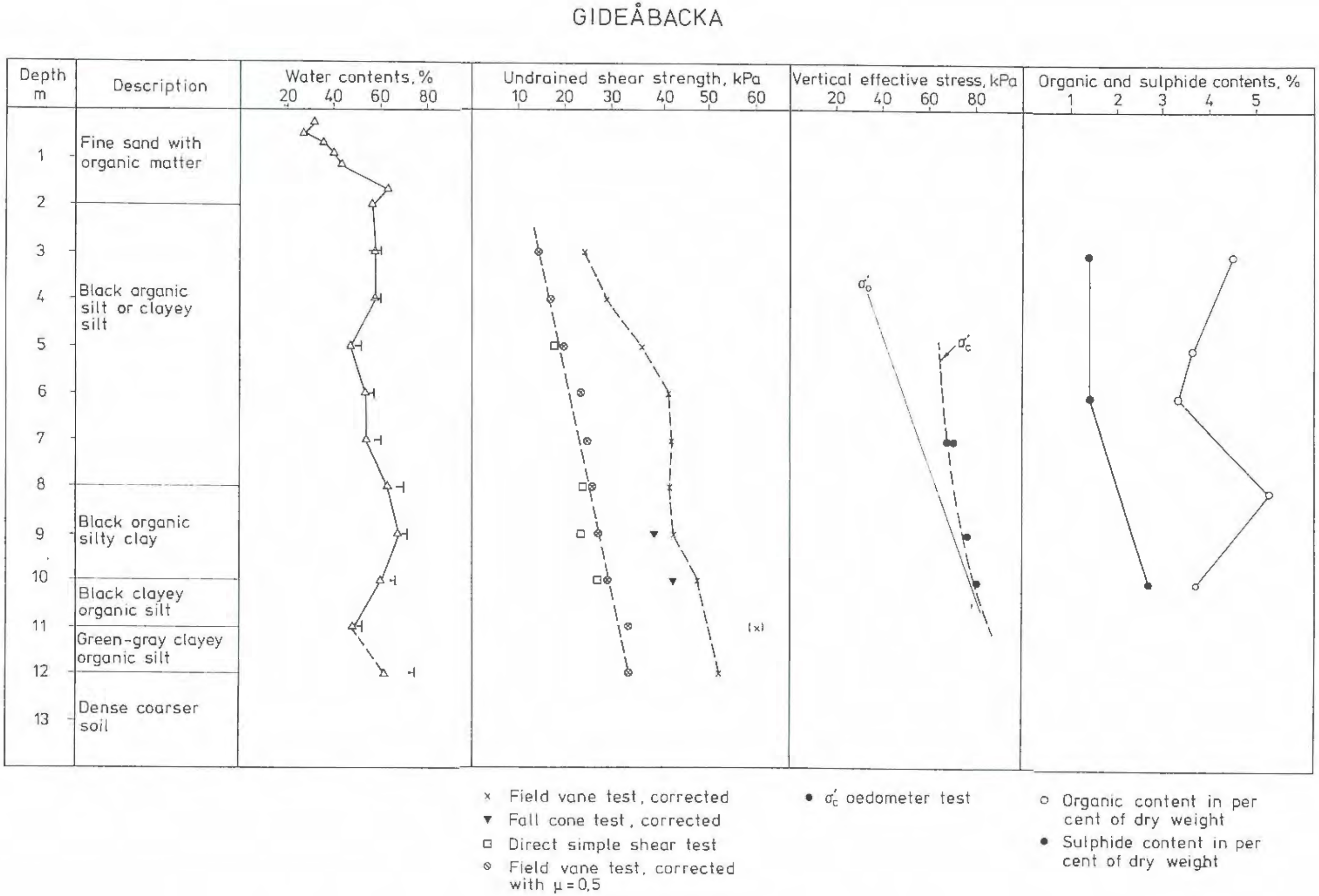


Fig. 37. Typical soil profile at Gideåbacka. (Data from Hermansson 1990)

In the oedometer tests, the preconsolidation pressures were evaluated with confidence only in the more clayey layers, but weaker indications obtained in the silt samples support the trend in FIG. 37. In the usual check of the results against Hansbo's relation, the strength values from the field vane tests and the fall cone tests were found to be at least twice as high as is normal with regard to the liquid limits and the preconsolidation pressures.

The results of undrained direct simple shear tests gave considerably lower shear strengths than the corrected field vane tests. So did also the results from the piezocone tests empirically evaluated as $\tau_{fu} \approx (q_T-\sigma_{vo})/17$. In order to be compatible with the results from the direct simple shear tests, the results from the field vane tests and the fall cone test had to be corrected by a factor of 0.5.

Comprehensive stability analyses later showed that shear strengths of the same order as those obtained in the direct simple shear tests should be used in the calculations.

- <u>Luleå</u>

A couple of samples of "svartmocka" from Luleå were examined in a special laboratory investigation at SGI.

The soil was a black silty clay with a clay content of about 27%, a natural water content of 85% and a liquid limit of 76%. The bulk density of the soil was about 1.6 t/m^3.

Fall cone tests and oedometer tests were made and the undrained shear strength values were almost twice the normal values for soils with corresponding liquid limits and preconsolidation pressures.

The loss on ignition was about 6% of the dry weight, which after normal correction with respect to the clay content would correspond to an organic content of 4%. More accurate determinations showed the organic content to be only about 1.5%.

Specimens of the soil were freeze-dried and fracture surfaces were studied in the microscope, FIG. 38.

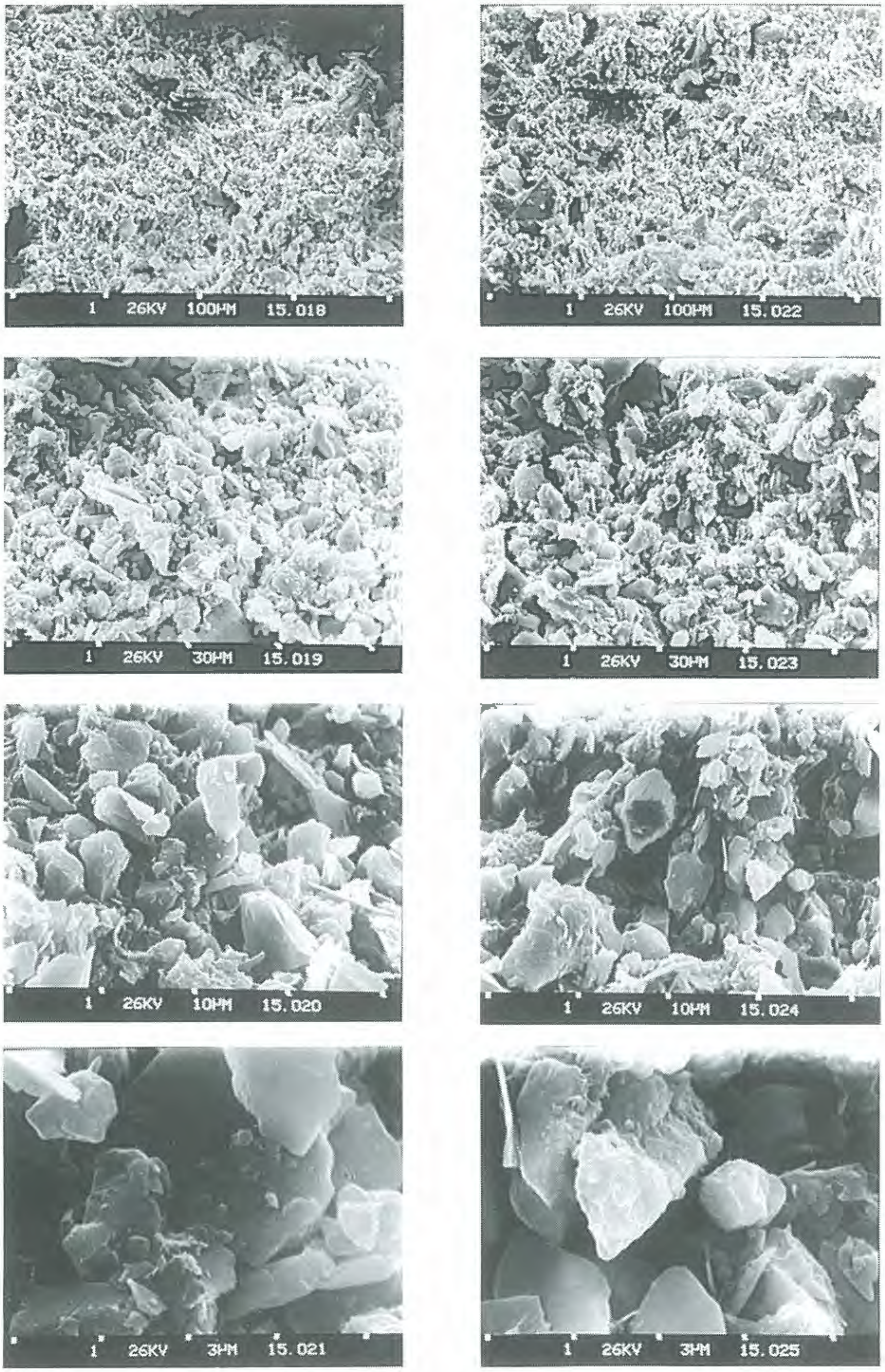


Fig. 38. Electron micrographs of "svartmocka" from Luleå.

No visible fibres or other organic substances were observed. Neither was there any significant difference to other very silty clays or any preferred orientation of the particles. Some diatoms could be seen, as well as some almost spherical aggregates, such as the one in the foreground in FIG. 39.

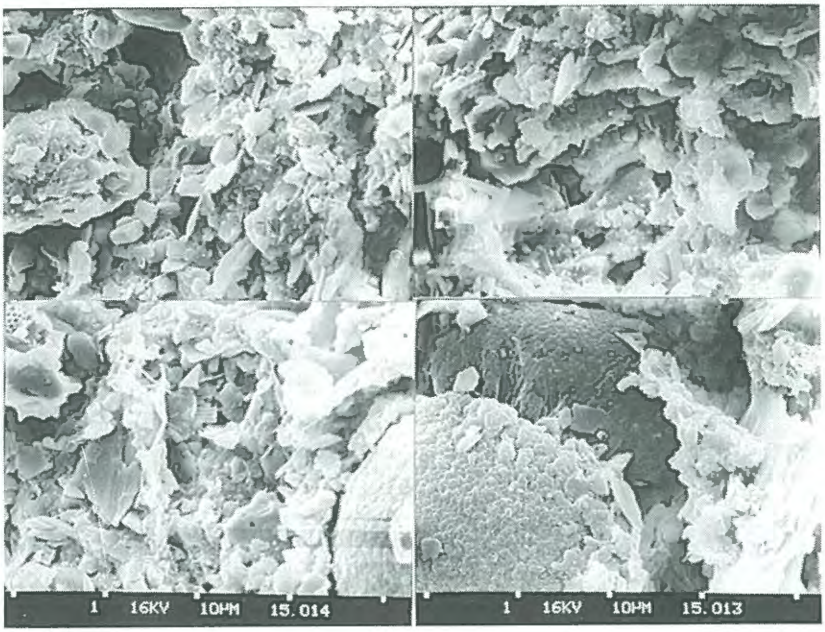


Fig. 39. Micrograph showing "svartmocka" from Luleå with a large spherical aggregate in the foreground.

X-ray diffraction showed that this aggregate contained large amounts of sulphur and iron. Similar spherical aggregates of presumed origin from organic matter or induced by organic activity in post-glacial clay have been shown by Pusch (1973). They have also been found in a number of other sulphide-bearing clays.

10. STUDIES OF THE RESULTS OF OEDOMETER TESTS

10.1 Rate effects in CRS-oedometer tests and comparisons with standard incremental tests

The stress-strain relations obtained in oedometer tests are dependent on how the tests are performed. In the incremental test the results depend on the number of increments that are used and the duration in time for the different load steps. In tests with constant rate of deformation the obtained stress-strain curves are dependent on the rate of deformation. In 1975 Sällfors showed that, when using the suggested method, the preconsolidation pressure evaluated from CRS-tests was practically independent of the testing rate, provided that this was kept below a critical rate.

In the present investigation CRS-tests with different rates of strain as well as series of standard incremental tests were performed at each level in the investigated layers in the new test fields. Some of the results from these tests were combined with Sällfors' data from 1975 and supplementary tests on Bäckebol clay by Larsson and Sällfors (1985). This investigation showed that, provided that the rate of deformation in CRS-tests was kept at or below the standard rate, $4 \cdot 10^{-5}$ mm/s, all the evaluated parameters σ'_c, M_L, σ'_L, M', k_i and β_k are insensitive to the rate of deformation when the suggested evaluation method is used. It was also shown that in organic soils the standard rate of deformation is very close to the critical rate and it is therefore prudent to use a somewhat lower rate of deformation for these soils. Furthermore, it was shown that the results from CRS-tests evaluated in this way are practically identical to the results from standard incremental tests with doubled load increments and 24 hour duration for each load step, FIGS. 40 and 41. The applicability for both types of results in settlement calculations has later been treated by Larsson (1986).

Only some of the results were available and were used by Larsson and Sällfors in 1985. The rest of the results later confirmed the conclusions, however.

Some of the test results are shown in Appendices A and B. There is some scatter in the results, but it must be remembered that in these very soft materials a scatter of ±10% only amounts to ±1-3 kPa. The very small scatter in Fig. 40 is due to using the average of several tests.

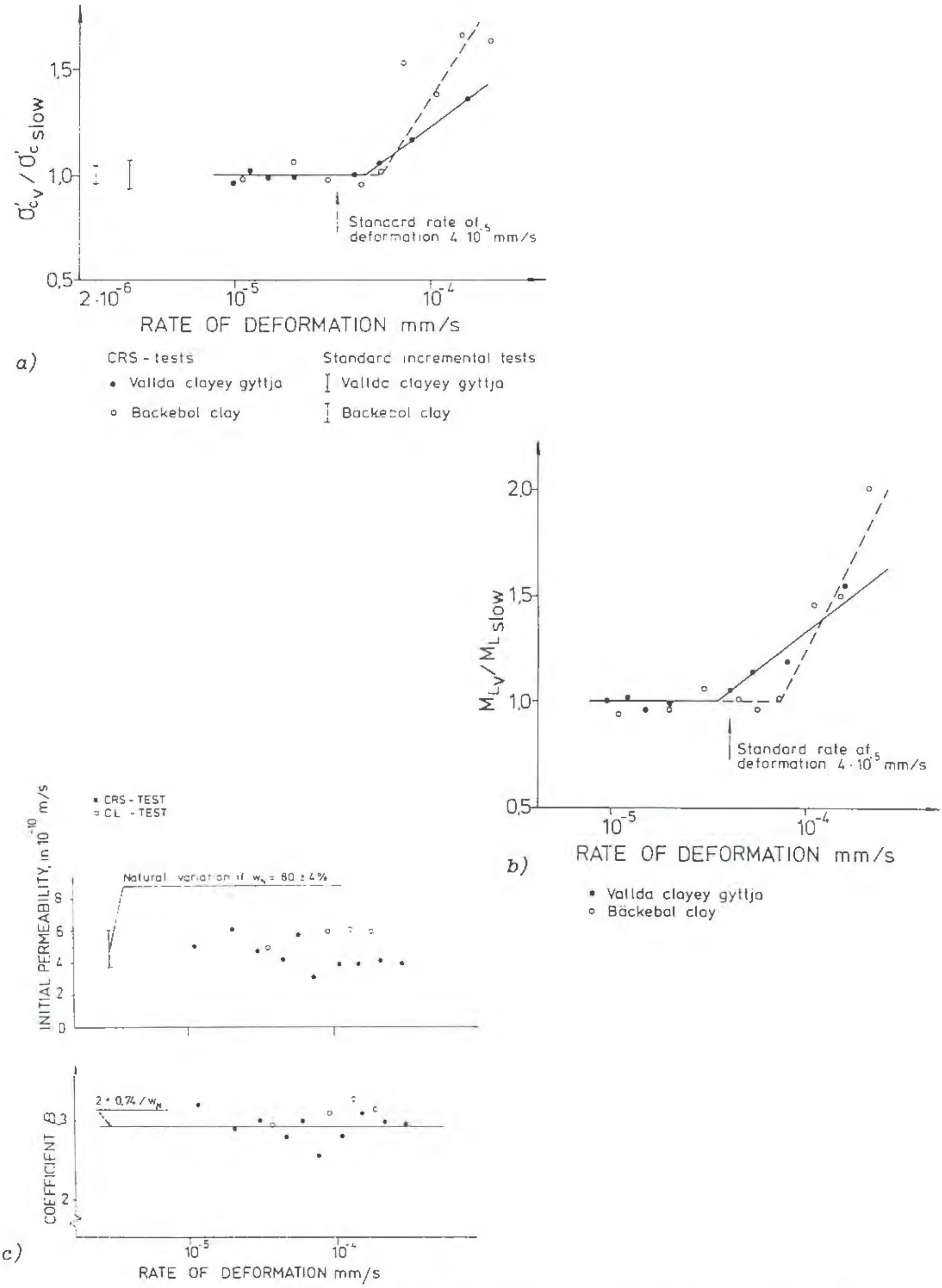


Fig. 40. Effect of rate of deformation on evaluated properties from CRS- tests. a) preconsolidation pressure, b)modulus M_L and c) permeability. (From Larsson and Sällfors 1985)

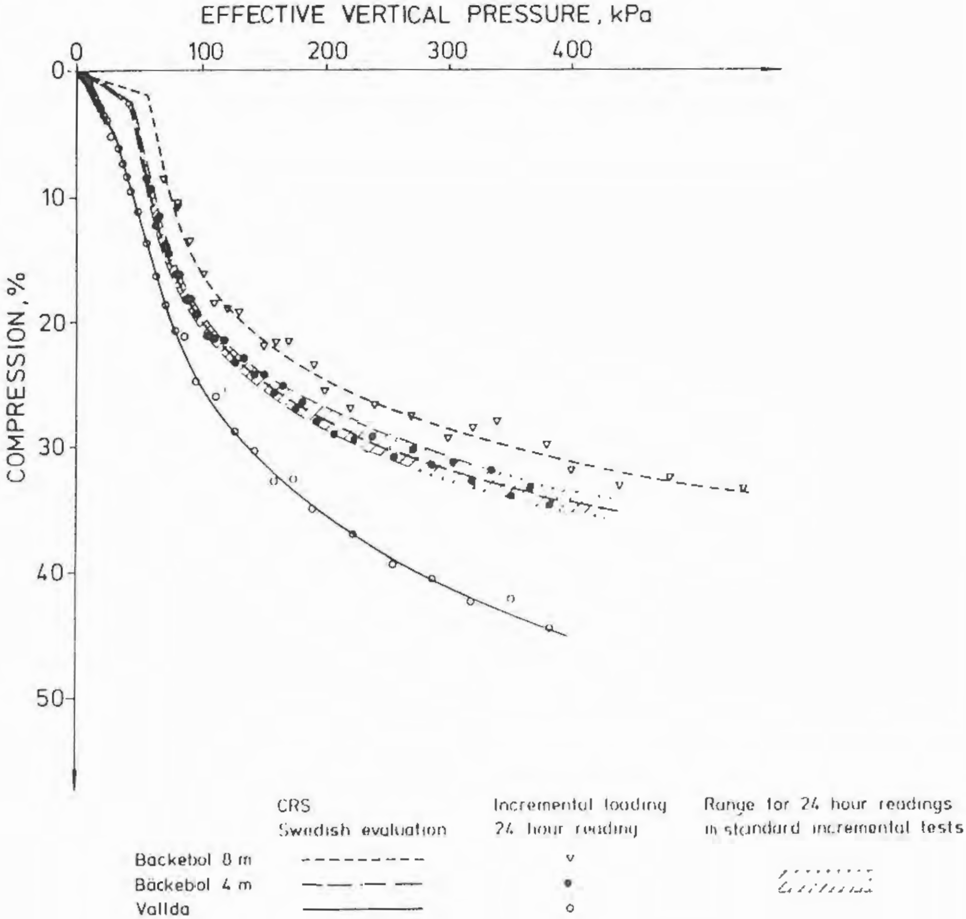


Fig. 41. Correlation between standard incremental tests and results from CRS-tests. (From Larsson and Sällfors 1985)

10.2 Rate effects on the compressibility of the soil; Coefficient of secondary consolidation

As found in the oedometer tests and in field observations, the stress-strain relation in a soil is highly time- or strain rate dependent and this fact cannot be overlooked in a settlement prediction. This rate dependency is usually expressed by the coefficient of secondary consolidation $\alpha_s = \Delta\varepsilon/\Delta\log t$. It is normally evaluated from the later parts of the time-compression curves in incremental tests, when all excess pore pressures and the hydrodynamic time-lag of the consolidation process may be assumed to have ceased to exist.

It has previously been found that the coefficient of secondary consolidation is very small for vertical stresses well below the preconsolidation pressure, increases rapidly as the preconsolidation pressure is approached, has a maximum at the preconsolidation pressure and then decreases for higher stresses. It has also been found that the description above is somewhat simplified and that the coefficient of secondary consolidation is rather a function of the relative compression or the void ratio, e.g. Larsson (1986), FIG. 42.

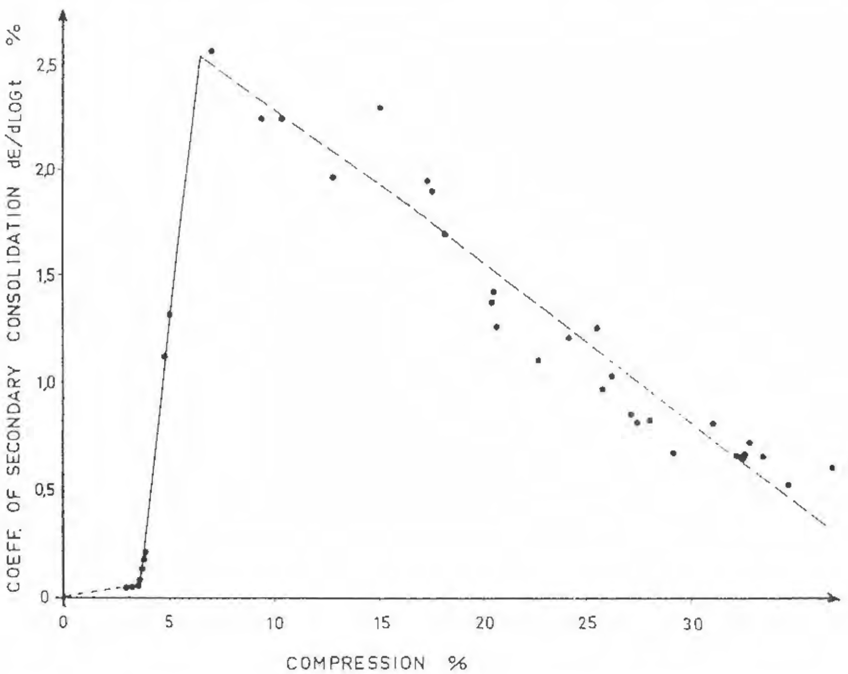


Fig. 42. Coefficient of secondary consolidation versus relative compression. (Example from Bäckebol clay, Larsson 1986)

The same description of the variation of the coefficient of secondary consolidation can basically be used also for organic soils.

The evaluated coefficients of secondary consolidation in the present investigation are shown together with the stress-strain curves in Appendix B. They deviate somewhat from the description above in that the maximum values occur at stresses somewhat higher than the preconsolidation pressures. It is difficult to evaluate if this is a characteristic of the soil or an effect of errors in the test. No measurements of ring friction were made, but previous experience has shown that the ring friction in incremental tests increases significantly at the end of each load step when the rate of deformation becomes small. This effect, coupled with the low preconsolidation pressures, may at least partly explain the deviation in the investigated organic soils.

The maximum coefficient of secondary consolidation α_s(max) and the coefficient of decrease in α_s with compression, $\beta_{\alpha_s} = -\Delta\alpha_s/\Delta\varepsilon$ (or some other description of this variation) are evaluated and used as input parameters in modern settlement calculations, Larsson (1986). Both parameters are mainly functions of the initial void ratio but are, for the sake of simplicity, usually related to the natural water content, FIG. 43.

This simplification involves large errors when the soil contains significant amounts of matter with specific densities that are different from about 2.7 t/m^3, which is the normal specific density for the mineral particles. The water content in organic soils is a function of both organic content and void ratio and is very sensitive to both parameters. To estimate the void ratio in organic soils, however, involves problems even if the organic content is known as it then becomes a matter of definition. The organic matter often has a specific density of about 1.4 t/m^3 but it does not consist of solid particles. When the deformation characteristics are considered, it is probably more relevant to use some kind of a dry particle density assuming only the open pores between the particles to be active in the time-deformation process in a shorter time perspective. The inhomogeneous particles with enclosed pores will probably also change their volume more than the instant elastic compression at a stress change, but that may be regarded as more of a geological time process. The dry particle density may roughly be assumed to be in the same order as dry wood or about 0.7 t/m^3.

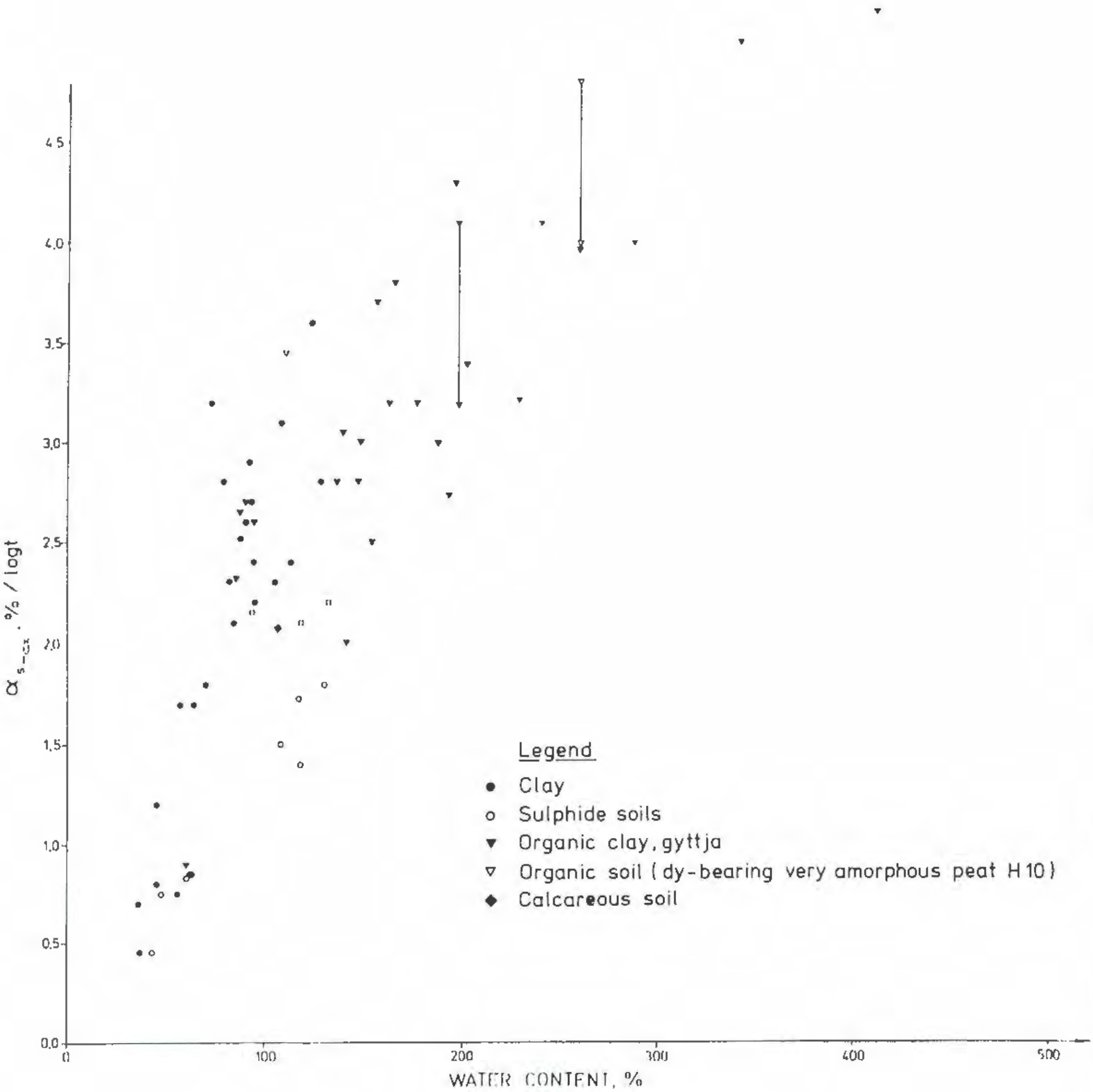


Fig. 43. Relation between maximum coefficient of secondary consolidation and water content.

The results in FIG. 43 have been recalculated with the assumptions that the mineral component has a specific density of 2.7 t/m^3 and that the organic component has a particle density of 0.7 t/m^3. The results plotted in FIG. 44 show a fairly common trend for both organic and inorganic soils and indicate that with these assumptions the maximum coefficient of secondary consolidation can be written $\alpha_{s(max)} = 1.35 \cdot e - 1$, % / logt.

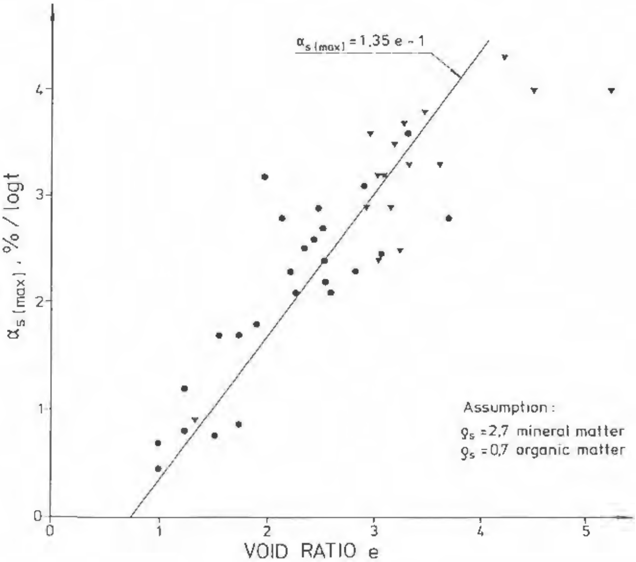


Fig. 44. Relation between maximum coefficient of secondary consolidation and estimated active void ratio.

For peats, the maximum coefficient of secondary consolidation has been found to be related more to the degree of humification than to the water content, FIG. 45. One explanation for this could be that the undecomposed peats, which usually have the highest water contents, also contain much more matter with enclosed inactive pores. When the organic matter becomes decomposed, the volume of the enclosed pores decreases at the same time as the water content decreases. The change in void ratio in terms of active voids is then not at all as dramatic as the changes in water content indicate and may even be the opposite. All empirical experience of the compressibility of peat points in the same direction, i.e. that soils with a high degree of decomposition are much more compressible than less decomposed soils with corresponding water contents, e.g. Carlsten(1988).

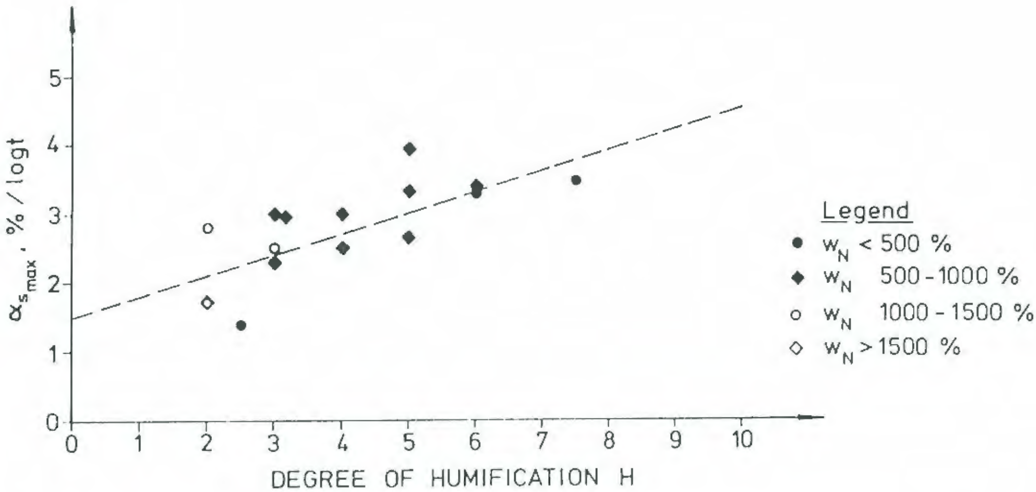


Fig. 45. Maximum coefficient of secondary consolidation versus degree of humification in peat.

10.3 Permeability and relation between vertical and horizontal permeability

The permeability in the fine grained and organic soils is generally low. Various reasons for this are possible. The permeability of a soil is related to its void ratio but in spite of the high water contents in the organic soils, the active void ratio is not so high because of the low density and large volume of the organic particles. The shape of the organic particles is also often such that the water is forced to make rather long detours to pass around them and the flow paths become longer. Another possibility suggested by Hansbo (1960) and Pusch (1973) is that unfixed organic colloids may clog the pores. The general trend of a permeability decreasing with relative compression (or void ratio) according to

$$\log k = \log k_i - \beta_k \cdot \Delta\varepsilon$$

is found to be applicable also to organic soils.

The relation between the horizontal and the vertical permeability of the soil is found to be dependent on the organic content and the degree of decomposition of the material. Undecomposed organic material normally has a preferred horizontal orientation which brings a higher permeability in the horizontal direction. This effect decreases when the amount of decomposed material in the soil increases and the orientation becomes less pronounced.

In homogeneous inorganic clays there is usually no significant difference between vertical and horizontal permeability. In the three clayey gyttjas from Kungsbacka and sections 6/900 and 7/600 where a pronounced orientation was observed, the horizontal permeability was typically two times the vertical. In the even more organic gyttja from Kristianstad which also contained a larger proportion of decomposed organic matter, this relation had decreased to 1.5, TABLE 4.

To further illustrate this, a number of results from an investigation on the permeability in various kinds of peat, (Inganäs 1978), are included. The results scatter, but a clear trend can be observed of a high relation between the horizontal and the vertical permeability in undecomposed peats, decreasing with further humification to about unity in completely decomposed material.

Table 4. Permeability and relation between horizontal and vertical permeability in soils with different kinds and amounts of organic matter.

Sites with clay and gyttja	$w_N \approx w_L$ %	Organic content %	k_V $m/s \cdot 10^{-10}$	k_H/k_V
Bäckebol	85	< 1	9	1
Mellösa	110	3-4	6	-
Välen	115	5-6	~10	-
Kungsbacka	150	9-10	3-7	1.7
Vallda 7/600	155	9-10	2-6	2.0
Vallda 6/900	180	10-11	2-5	2.0
Kristianstad	350	27-38	10-30	1.5

Peat sites	w_N %	Degree of humification	k_V $m/s \cdot 10^{-10}$	k_H/k_V
Skeda Udde	601	9-10	1	1
Kolbyttemon	612	9	20	2
Rosenkälla	972	8	100	6
Stavsätter	1051	5-6	1400	2
Borås	1365	5	140	10
Jordbro	1420	5	360	14

10.4 Coefficient of consolidation

The coefficient of consolidation is a measure of how long it will take for the soil to consolidate for an applied load. The coefficient of consolidation is not a separate soil property, but the product of the compression modulus and the permeability. Normally, it is the coefficient of consolidation at vertical compression and vertical water flow c_v that is of interest.

$$c_v = \frac{M \cdot k_v}{g \cdot \varrho_w}$$

where M = modulus of compression (oedometer modulus)
k_v = vertical permeability
g = specific gravity
ϱ_w = density of water

Neither the modulus nor the permeability is a constant but varies with compression, as does the c_v-value. It is relatively high for stresses up to the preconsolidation pressure and then drops to a low value as the modulus drops. In organic soils it usually stays at very low values at further compression as the decrease in permeability at the large deformations in these soils more or less compensates for the gradual increase in modulus. The c_v-value of most importance for both settlement calculations and estimation of gain in undrained shear strength, as well as estimation of relevant time aspects on the time dependent undrained shear strength, is the c_v-value after the preconsolidation pressure has been exceeded. Due to the low permeability and the low compression modulus, this value is often 5-10 times lower in organic clay and gyttja than is normal for soft inorganic clays, TABLE 5.

Table 5. Coefficients of consolidation and of secondary consolidation in clays and gyttjas with varying organic contents.

Soil	Organic content	Coeff. of consolidation $\sigma' > \sigma'_c$ c_v(min) m^2/s	Coeff. of consolidation α_s % / log t
Typical soft clay	0	$\approx 20 \cdot 10^{-9}$	≈ 2
Bäckebol clay	<1	$10 \cdot 10^{-9}$	2.5
Lilla Mellösa	3-4	$4 \cdot 10^{-9}$	3.0
Section 7/600	9-10	$2\text{-}5 \cdot 10^{-9}$	2.9
Kungsbacka	9-10	$5 \cdot 10^{-9}$	3.2
Section 6/900	10-11	$2\text{-}4 \cdot 10^{-9}$	3.5
Kristianstad	27-38	$5 \cdot 10^{-9}$	4.1

The rate of consolidation, however, is not only dependent on the coefficient of consolidation but also on the coefficient of secondary consolidation. At the early and intermediate stages of a consolidation process, when the excess pore pressure is dissipated at a rate governed by the coefficient of consolidation, new excess pore pressures are created at a rate governed by the coefficient of secondary consolidation (e.g. Larsson 1986). The latter coefficient is often about 2 times higher in organic clays and gyttjas than in soft clays and the consolidation process as expressed by pore pressure dissipation may therefore be more than 10 times slower if the geometries are similar. The gain in shear strength mainly follows the rate of compression rather than the rate of pore pressure dissipation (Bjerrum 1967 and 1972, Larsson 1986), and is therefore not directly hampered by the slower pore pressure dissipation because of the secondary effects. The undrained shear strength at load application, however, may depend on them. At load application the pore pressure in the soil increases. For a certain time after load application, the pore pressures continue to rise because of the further development of excess pore pressures with time. At the critical time the generation of excess pore pressures becomes compensated by the pore pressure dissipation because of consolidation. The normally used testing rates for determination of undrained shear strength in soft soils have empirically been found to be applicable to inorganic soils, but for organic clays and gyttjas slower rates or appropriate correction factors for rate effects should be used (e.g. Berre 1981 and Slunga 1983).

10.5 Coefficient of earth pressure at rest, K_o

The relation between the effective horizontal and vertical stresses is called the coefficient of earth pressure and this relation in the natural ground before any external interference has been made is the coefficient of earth pressure at rest K_0.

The value of K_0 has a minimum when the soil is normally consolidated. If the soil is unloaded, e.g. by erosion, it becomes overconsolidated and K_0 increases as the horizontal stresses do not decrease in the same proportion as the vertical stress.

For clay, the value of K_0 at varying overconsolidation ratios is normally expressed as

$$K_0 = K_{0_{nc}} \quad OCR^n$$

where $K_{0_{nc}}$ = coefficient of earth pressure at OCR
K_0 = coefficient of earth pressure in normally consolidated state (OCR=1)
OCR = overconsolidation ratio (σ'_c/σ'_v)
n = exponent ≈ 0,5 à 0.6

For organic clays and gyttjas, this relation should be used with caution as there are only a few actual measurements to support its validity in these kinds of soil.

Overconsolidation is often created by processes other than preloading, e.g. secondary consolidation, fluctuating ground water levels etc. How this affects the coefficient of earth pressure is not clarified. In fact, an intense discussion on the subject is going on at present. In practice no separation is usually made for the causes of overconsolidation, especially since they are normally not clarified or well understood.

In this project no measurements of the horizontal stress in situ have been made. The horizontal and vertical preconsolidation pressures have been measured in oedometer tests and in some cases in triaxial tests. The relation between them has then been used as an indication of the coefficient at rest in the normally consolidated state $K_{0_{nc}}$, Fig. 46.

In inorganic clays the coefficient of earth pressure at rest has been found to vary strongly with the consistency limits of the soil. In the organic soils the estimated values of $K_{0_{nc}}$ were found to be between 0.5 and 0.6, with only a small tendency to decrease with increasing organic content. These soils all had organic contents of 4% or more and significant amounts of horizontally oriented fibres.

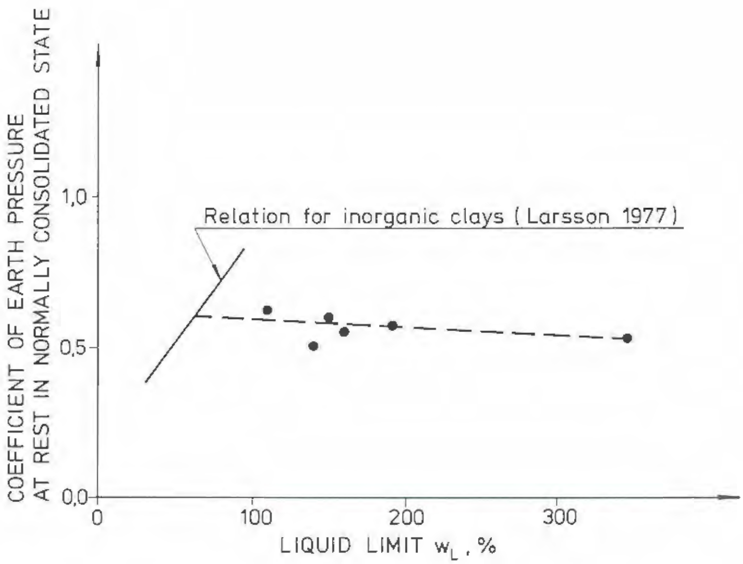


Fig. 46. Coefficient of earth pressure at rest in normally consolidated organic soils, K_{onc}, estimated from the relation between preconsolidation pressures in the horizontal and the vertical directions.

11. STUDIES OF THE RESULTS OF SHEAR STRENGTH TESTS

11.1 Rate effects

The rate effects in undrained triaxial tests on organic clay and clayey gyttja have been studied in detail by Slunga (1983). From large test series of active compression tests on each of the materials in the Finnish investigation, it was deduced that the rate dependency could be described by an equation given by Hansbo (1975)

$$\frac{\tau_{tf}}{\tau_{ref}} = B_1 + B_2 \log (1/t_f + B_3)$$

where τ_{tf} = undrained shear strength at time to failure t_f

τ_{ref} = reference shear strength

t_f = time to failure

B_1, B_2 and B_3 are constants

This equation implies that the rate effects even out at very slow rates and the ultimate undrained shear strength at extremely low rates of deformation approaches

$$\frac{\tau_{min}}{\tau_{ref}} = B_1 + B_2 (\log B_3)$$

where τ_{min} = absolute minimum of undrained shear strength when all rate effects have evened out.

Slunga used the results from standard field vane tests as reference strengths and then obtained the results shown in TABLE 6.

The results entail that, in comparison to standard triaxial tests run with a deformation rate of 0.6 %/hour, a tenfold decrease in testing rate brings a decrease in evaluated undrained shear strength of 10%, and the minimum undrained shear strength is 85% of the value estimated from the standard test. The results are very uniform and the scatter is small. Only one material (Raastala 2.75 m) gives somewhat higher reductions. The reason for this may be that fewer tests were run on this material, which had an undrained shear strength of only 5-6 kPa, and some scatter must therefore be expected.

Table 6. Constants for the time dependency of the undrained shear strength in triaxial compression for soils with different organic contents. (From Slunga 1983)

Site	Organic content,%	B_1	B_2	B_3
Raastala 5.25 m	< 2	1.12	0.11	$2.5 \cdot 10^{-4}$
Raastala 2.75 m	5-6	1.16	0.16	$1.7 \cdot 10^{-4}$
Seinäjoki	3-4	1.11	0.11	$1.9 \cdot 10^{-4}$
Siuntio	8-9	1.21	0.21	$10 \cdot 10^{-4}$
Vanhankaupunginlahti	7-8	1.17	0.17	$5 \cdot 10^{-4}$
Lokalahti	13-15	1.11	0.10	$1 \cdot 10^{-4}$

The rate effects have also been investigated for the Välen organic clay (organic content 5-6%) at Chalmers University. Series of undrained triaxial compression tests were run at the standard rate 0.6 %/hour, ten times faster and a hundred times slower. Also these results indicate that the rate effects even out and that the decrease in shear strength is about 10% for a tenfold decrease from the standard rate and that the minimum undrained shear strength is about 85% of the value estimated from standard tests, FIG. 47.

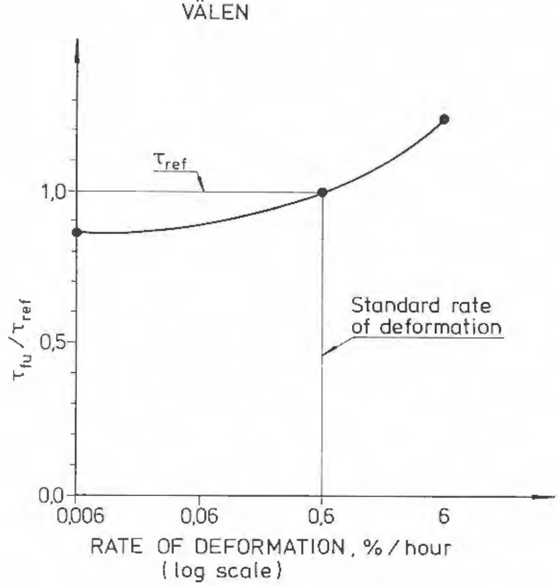


Fig. 47. Undrained shear strength as a function of rate of deformation in Välen organic clay. Averages of 5 test series.

Also for the organic clay at Lilla Mellösa a test series with different rates of deformation was run. As only one series was run, the results scatter somewhat, but they generally confirm the previous findings, FIG. 48.

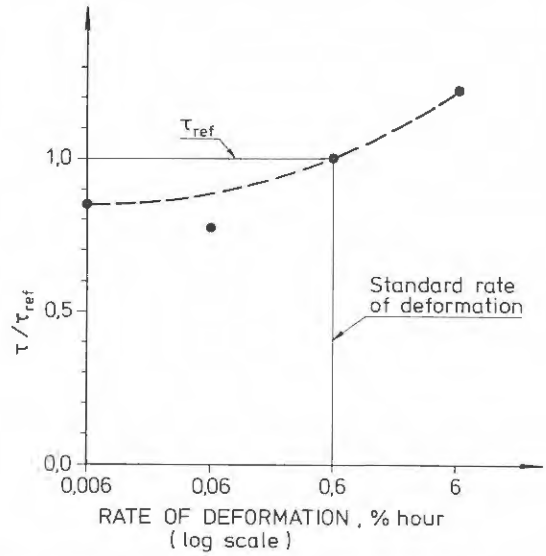


Fig. 48. Undrained shear strength as a function of rate of deformation in Lilla Mellösa organic clay.

For the other materials in the investigation both compression and extension tests were run both at the standard rate and 10 times slower. The scatter makes it impossible to give exact numbers for the various soils, but in general the slow tests gave about 10% lower strength values.

Results similar to those for the organic clays were also obtained for the "svartmocka" from Umeå by Schwab (1976).

11.2 Stress-paths and mode of failure in triaxial tests

In undrained triaxial tests in inorganic clays, the general development of pore pressures and effective stresses is such that within an "elastic" stress space the pore pressure development is adjusted to keep the mean effective stress constant. The border lines of this "elastic" stress space, the so called "yield surface", are made up of the preconsolidation pressures in vertical and horizontal directions and the failure lines for active or passive failures (Scofield and Wroth 1978, Larsson 1977, Tavenas and Leroueil 1977 and 1979, Larsson and Sällfors 1981), FIG. 49.

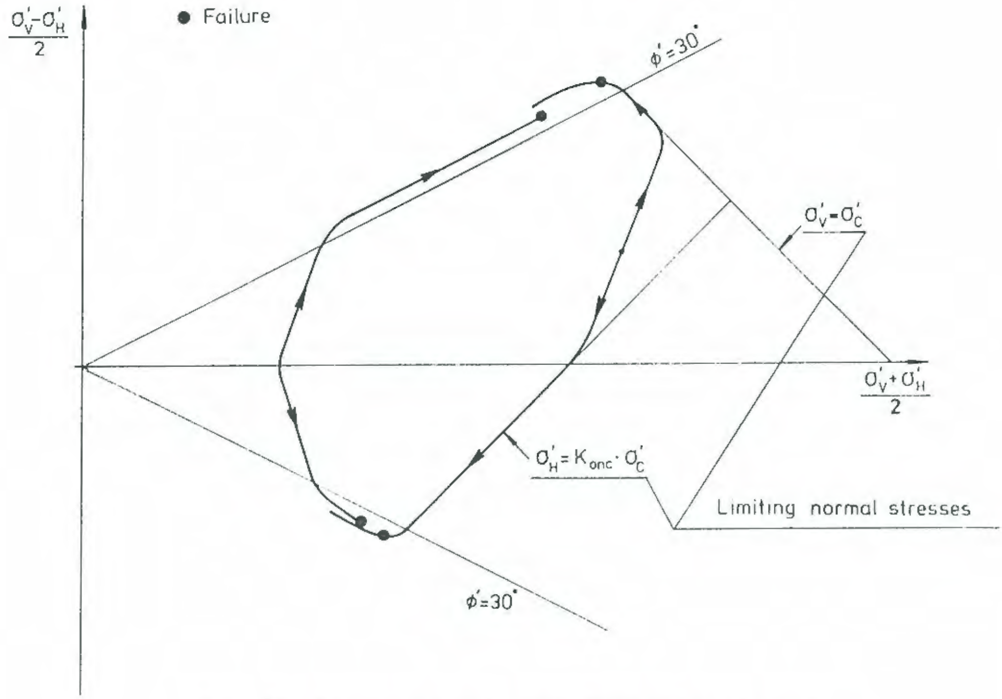


Fig. 49. Stress paths in undrained triaxial tests.

When the effective stress path reaches one of the limiting preconsolidation pressures the pore pressure development changes so that the preconsolidation pressure is not exceeded. Instead, the stress path follows this line to the intersection with the failure criteria where failure occurs. In soils with high overconsolidation ratios the failure criteria is reached before the preconsolidation pressure. The stress path then follows this line up towards the intersection with the preconsolidation pressure and actual failure occurs some distance before the intersection is reached. The "elastic" response of the soil within the yield surface can be observed by plotting the stress paths in a q-p' space, Fig. 50, where the vertical stress paths show that the mean effective stress is constant.

This general description is valid at standard rates of testing. All stress-strain curves in soils are time dependent and so are both the stress paths and the yield surfaces. At higher testing rates the yield surfaces expand and the pore pressure response becomes somewhat lower. Correspondingly, at lower rates the yield surfaces decrease somewhat. Because of secondary effects the pore pressure development is larger and the stress paths are no longer quite vertical in the q-p' space. A lower limit for the yield surface at 80 per cent of the preconsolidation pressure in any direction is often used (Larsson 1977).

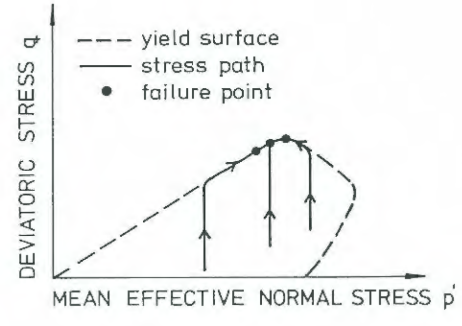


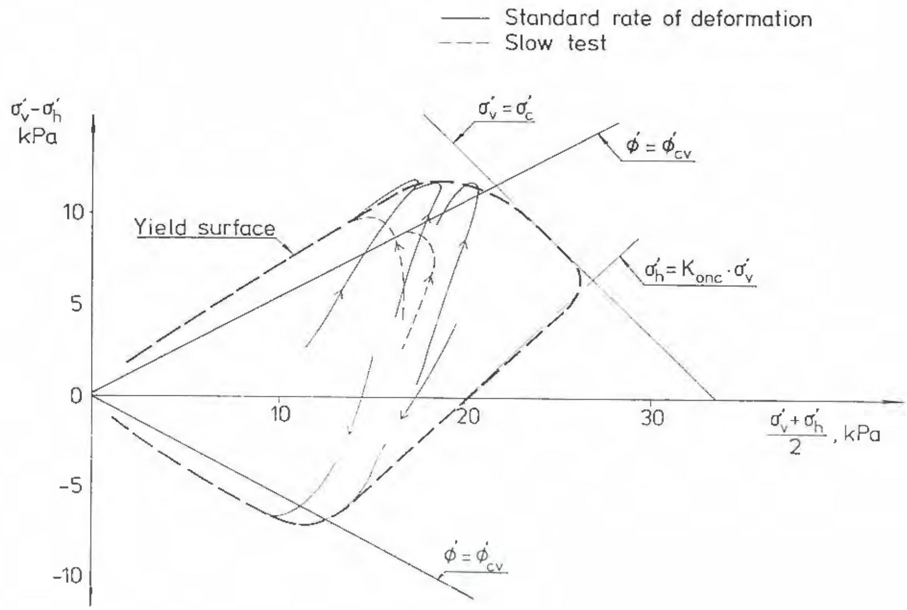
Fig. 50. Undrained stress paths in a q-p' stress space.

This general behaviour also fits the results from the mainly non-fibrous organic clay at Lilla Mellösa, FIG. 51.

In the more fibrous soils at Välen, sections 7/600 and 6/900, Kungsbacka and Kristianstad the mode of failure was different. Instead of developing failure surfaces inclined $(45-\phi'/2)^0$ towards the direction of the major stress, as inorganic soils do, these samples split up vertically in compression tests and were cut off horizontally in extension tests. The pore pressure response in the elastic range and along the limiting preconsolidation lines was similar to the usual pattern described above, but failure did not occur until the effective horizontal stresses approached zero in compression tests or the effective vertical stress was close to zero in extension tests.

The strains at failure in these soils were fairly large. As the volume is constant during undrained tests, this implies that the specimens were considerably elongated in the dirction of the minor stress. As explained by Janbu (1979), this elongation (or tensile strain) may bring cracks in the major stress direction, which can occur even if the minor stress is a compressive stress.

This behaviour in the more organic soils, which also makes the usual interpretation in terms of effective parameters c' and ϕ' according to the Mohr Coulomb criterion less meaningful, can be ascribed to the influence of the abundance of fibres in these materials . Friction angles ranging from 60^0 to 90^0 only indicate that the effective stresses in the minor stress directions are very small or zero, FIG. 52.



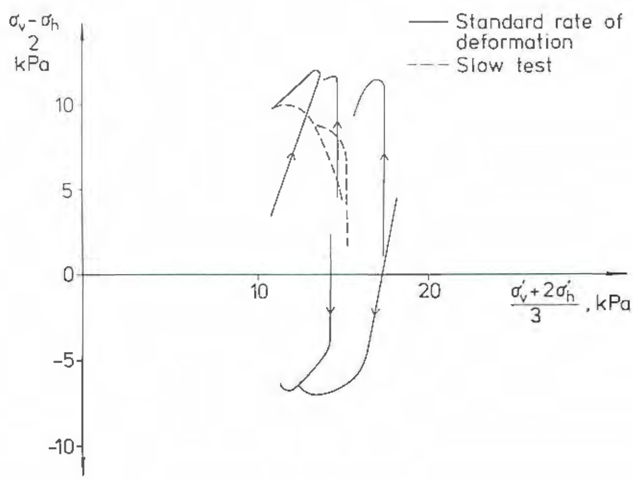


Fig. 51. Stress paths in undrained triaxial tests on organic clay from Lilla Mellösa.

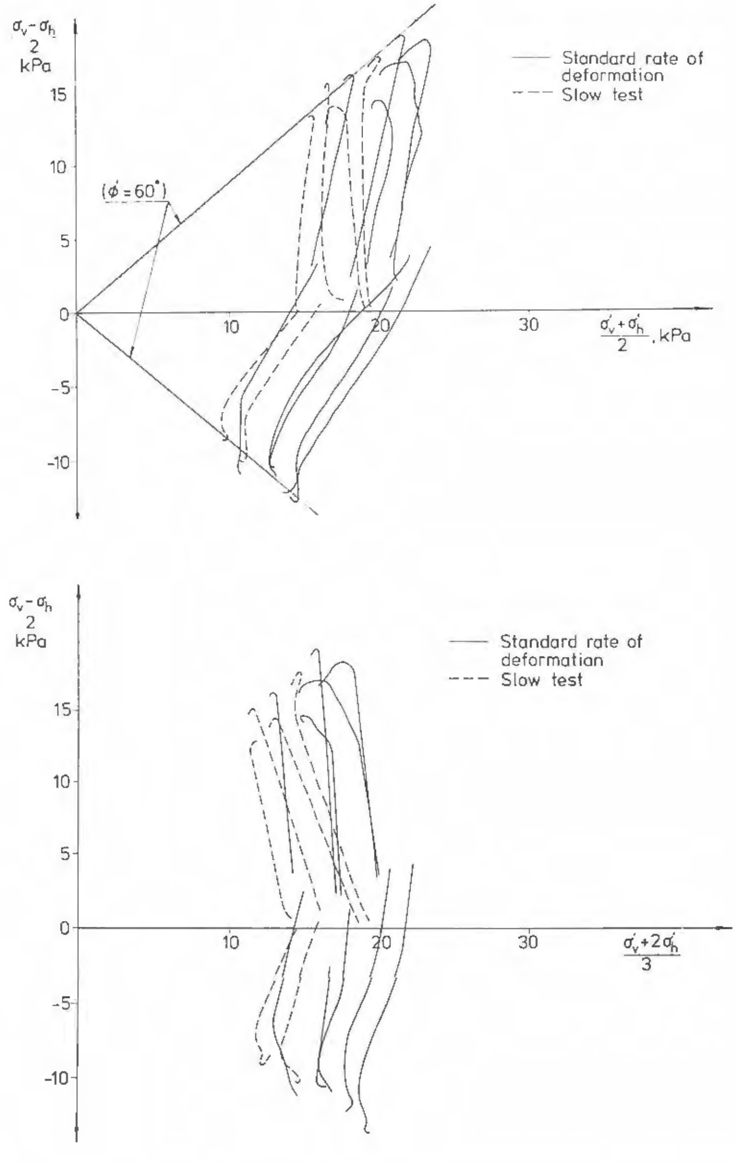


Fig. 52. Stress paths in undrained triaxial tests on clayey gyttja from section 7/600.

11.3 Stress-strain curves and failure strains

The deformation behaviour in shear is strongly influenced by the organic content of the soil. The influence of the organic content on the failure strains in unconfined compression tests was shown by Pusch (1973) and the influence on the failure strains in triaxial compression tests was shown in the report by the Geotechnical Laboratory at the Technical Research Centre of Finland (1982). This influence is also accounted for in empirical correlations for the modulus of elasticity and the shear modulus which give low moduli for high-plastic and organic soils e.g. Larsson 1986

$$E = \frac{215 \cdot \tau_{fu} \cdot \ln F}{I_P}$$

where E = modulus of elasticity
τ_{fu} = undrained shear strength
F = safety factor against undrained shear failure
I_P = plasticity index (increasing with increasing organic content

In the present investigation both triaxial compression and extension tests have been performed, FIG. 53.

As for tests on inorganic clays, there is no greater difference between the failure strains in the two types of tests. Moreover, in the slow tests, which are considered most relevant, the stress strain curves are fairly elastic-plastic without pronounced peaks or sudden drops in shear strength after failure, e.g. FIG. 54.

There is thus no reason for using a strain compatibility technique for evaluating the shear strength in a uniform soil of this type. For large shear surfaces passing through both this kind of soil and more brittle soils this could be justified, however.

The axial deformations at failure in the triaxial tests are plotted in FIGS. 55 and 56 as functions of the organic content and of the liquid limit. The results from both the Finnish and the Swedish investigations are included. As can be seen, the failure deformation in both active and passive tests increases fairly linearly with the organic content from about 1.5% for inorganic soil to about 10% at an organic content of about 13%. Thereafter, the failure strains do not seem to be much affected by higher organic contents, but stay high between 10 and 13%. It should be observed that all the organic matter in the investigated soils was of the gyttja type.

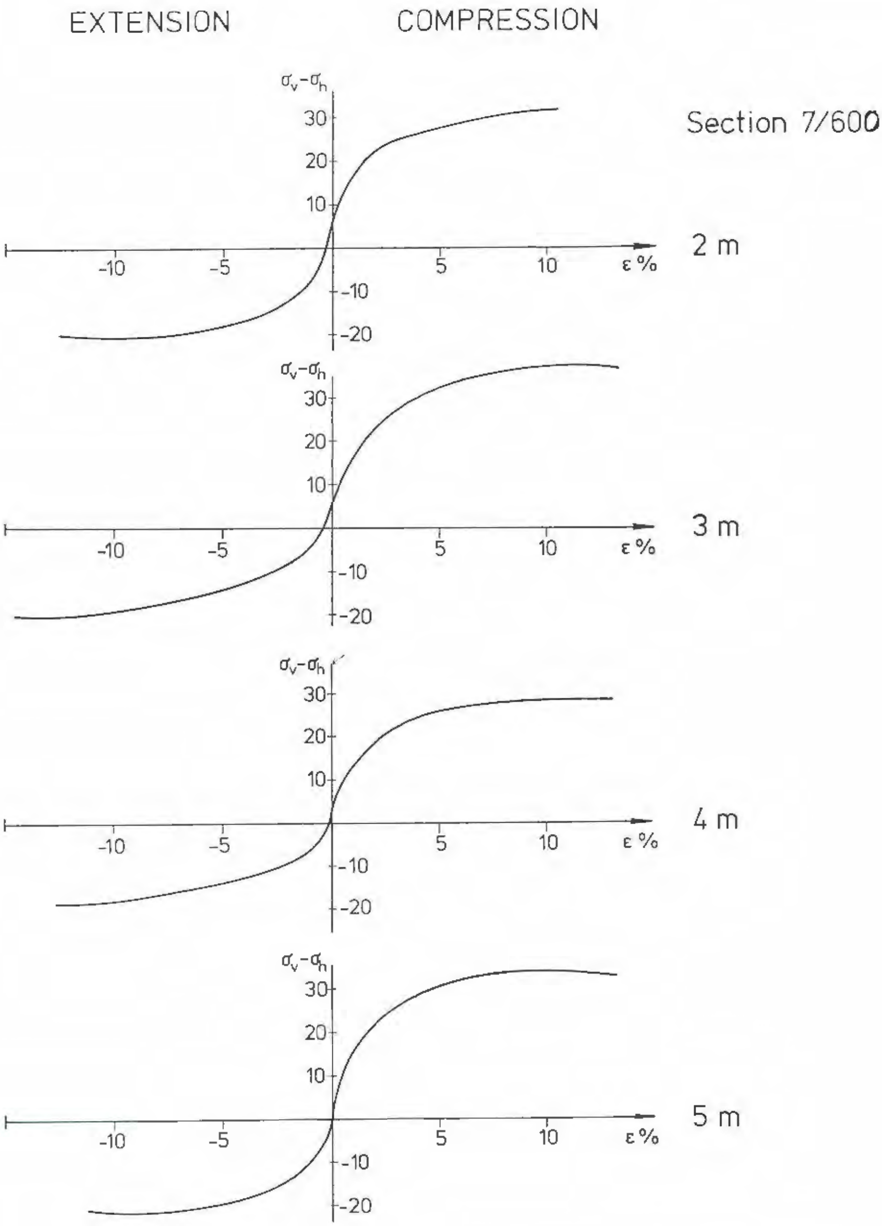


Fig. 53. Results from undrained triaxial compression and extension tests on clayey gyttja from 2 - 5 m depth at section 7/600.

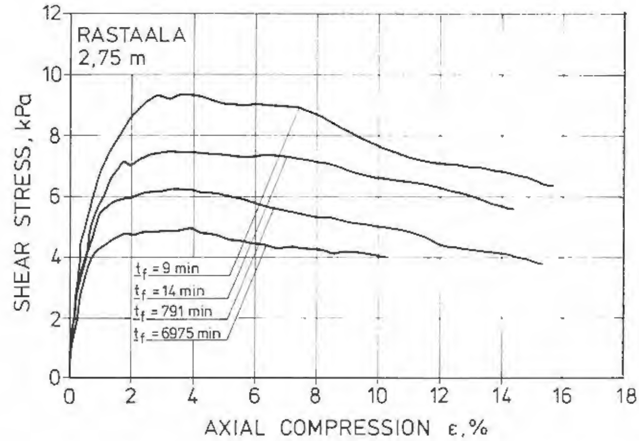


Fig. 54. Stress-strain relations in undrained triaxial compression tests at different rates of loading for Raastala organic clay. (From Geotechnical laboratory of the Technical Research Centre of Finland 1982)

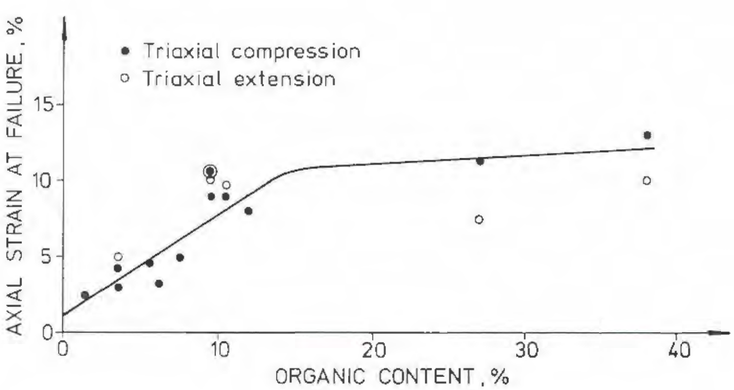


Fig. 55. Axial deformation at failure in undrained triaxial tests as a function of organic content.

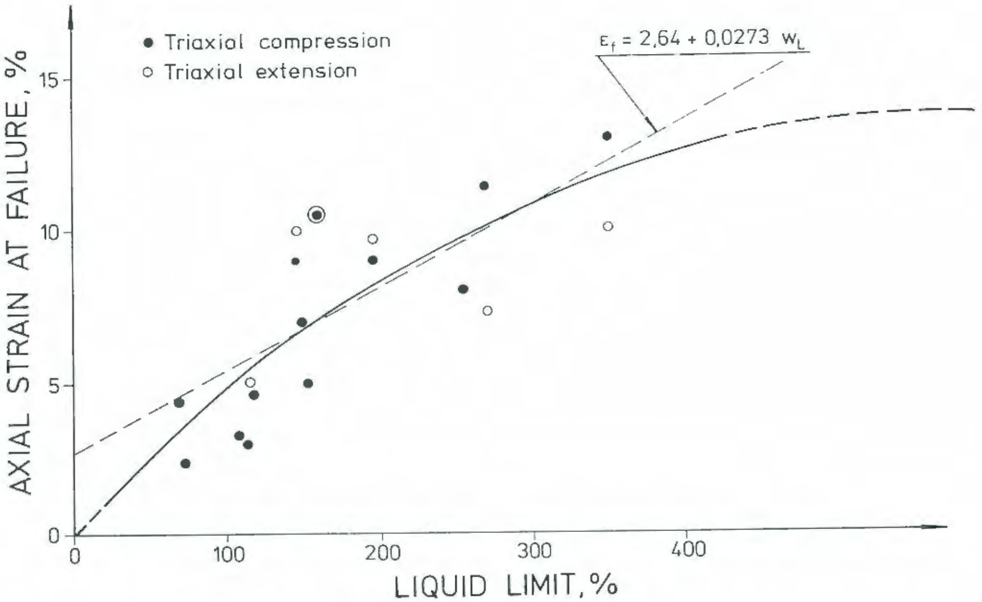


Fig. 56. Axial deformation at failure in undrained triaxial tests as a function of liquid limit.

A more uniform relation is obtained if the failure strains are plotted versus the liquid limits. A linear regression then gives

$$\varepsilon_f = 2.64 + 0.0273\, w_L$$

where ε_f = failure strain in per cent
w_L = liquid limit in per cent

The best fitting curve is slightly curved, however, and gives zero failure deformation at zero liquid limit and a maximum failure strain of about 14%.

11.4 Effective strength parameters in direct simple shear tests

Direct simple shear tests are performed with the intention of simulating shear along horizontal planes. With the preferred horizontal orientation of the fibres, it should be possible to perform and evaluate these tests on organic soils in the same way as on inorganic soils. Similar testing methods are also often used for peat e.g. Landva 1980.

In the direct simple shear test the sample is first consolidated for either the in situ vertical stress or for a desired loading history before it is sheared. The horizontal shear force is then applied and, as there is no forced shear surface in the test, the usual result is an angular rotation of the sample. In drained tests in overconsolidated soils the height of the sample remains almost constant or there may even be a small expansion. In these cases a maximum shear stress is reached and failure can be clearly defined. In more normally consolidated soils the sample starts to consolidate and becomes further compressed when the horizontal stresses are applied, the soil then gradually hardening with the deformations. Both vertical and horizontal deformations become very large in soft soils and for practical reasons the tests are stopped at an angular rotation of the sample of 0.15% radians (horizontal displacement of the upper part ≈ 15% of the sample height). If no failure stress is reached before this deformation, failure is evaluated as the shear stress at this stage.

In soft clays, has been found that the drained shear strength evaluated in this way becomes about equal to the undrained shear strength for vertical stresses over $0.5 \cdot \sigma'_c$, (e.g. Larsson 1977). For lower stresses where actual failure occurs the drained shear strength becomes lower than the undrained strength.

In design it is the lowest available shear strength at any time for the applied loading case that should be used in the calculations. Therefore, in soft low permeable soils the drained shear strength is normally of interest only when the stresses are well below the preconsolidation pressure. In the low stress range the drained shear strength in inorganic Swedish clays can empirically be expressed by the parameters $c' \approx 0.03\ \sigma'_c$ and $\phi' \approx 30^0$, (Larsson et al 1984). In calculations, the resulting shear strengths from these parameters and the effective stresses are compared to the undrained shear strengths in the corresponding parts of the soil mass and the lower value is used as design shear strength; combined analysis (Larsson 1983 and 1984).

From the drained tests also the angle of friction at critical density may be evaluated. This is the friction angle that can be mobilized when the soil is sheared at constant volume ϕ'_{cv}.

This angle is evaluated at either real constant volume or, more often, by correcting the mobilized friction angle at a certain deformation for the volume changes that occur simultaneously. The evaluation of ϕ'_{cv} from direct simple shear tests is not quite correct theoretically, but for low values of this angle of 30^0 or less the difference is relatively small (Rowe 1969).

The drained and undrained direct simple shear tests on the organic clays and gyttjas gave the same type of results as for inorganic clays, FIG. 57.

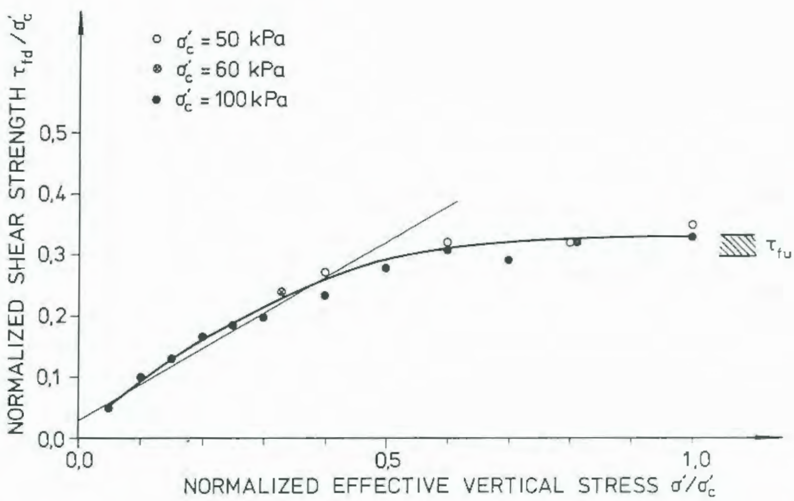


Fig. 57. Results from drained direct simple shear tests on clayey gyttja from 5 m depth at section 7/600.

The only significant difference was that the undrained shear strength in relation to the preconsolidation pressure was higher in organic clays and gyttjas and that this relation to a significant degree decreases with increasing preconsolidation pressures.

The results in terms of effective strength parameters and friction angles at constant volume are shown in TABLE 7.

Table 7. Effective strength parameters at low stress levels and friction angles at constant volume evaluated from direct simple shear tests on soils with different organic contents.

Soil	Organic content %	c' kPa	ø' Degrees	$ø'_{cv}$ Degrees
Inorganic soft clay	0	$0.03\ \sigma'_c$	30	30
Lilla Mellösa	3-4	-	-	32
Section 7/600	9-10	$0.03\ \sigma'_c$	30	31
Kungsbacka	9-10	$0.03\ \sigma'_c$	30	30
Section 6/900	10-11	$0.05\ \sigma'_c$	30	28
Kristianstad	27-38	$0.05\ \sigma'_c$	30	30 -32

As can be seen from the table, there is no significant difference between the effective strength parameters in direct simple shear tests in organic clays and gyttjas as compared to inorganic soft clay, except possibly a slightly higher c' component in gyttjas.

11.5 Sensitivity

The sensitivity S_t of organic clay and gyttja is relatively low in comparison to normal values for inorganic soft clays in Sweden. The sensitivities as determined by fall cone tests mainly varied between 5 and 14 for the investigated organic clays and gyttjas from Finland and Sweden. According to the Swedish classification system (Karlsson & Hansbo 1984) the soils are

Low sensitive when $S_t < 8$

Medium sensitive when $8 \leq S_t \leq 30$

and High sensitive when $S_t > 30$

When the sensitivity is ≥ 50 and the remoulded shear strength is less than 0.37 kPa the soil is denoted as "quick". A prerequisite for a soil to be quick is that the natural water content is higher than the liquid limit and the liquidity index

$$I_L = \frac{w_N - w_P}{I_P} > 1$$

The soils in these investigations with significant organic contents are classified as low or medium sensitive and most of them have liquidity indices less than 1.

One of the "svartmockas" was quick, but it also had a liquidity index of around 1.5. When the sensitivities are plotted versus the liquidity indices the usual trend with increasing sensitivity at increasing liquidity index is found with the usual rather large scatter, FIG. 58. The values for the "svartmockas" seem to be fairly normal and the values for organic clay and gyttja are somewhat on the low side in comparison to inorganic clays.

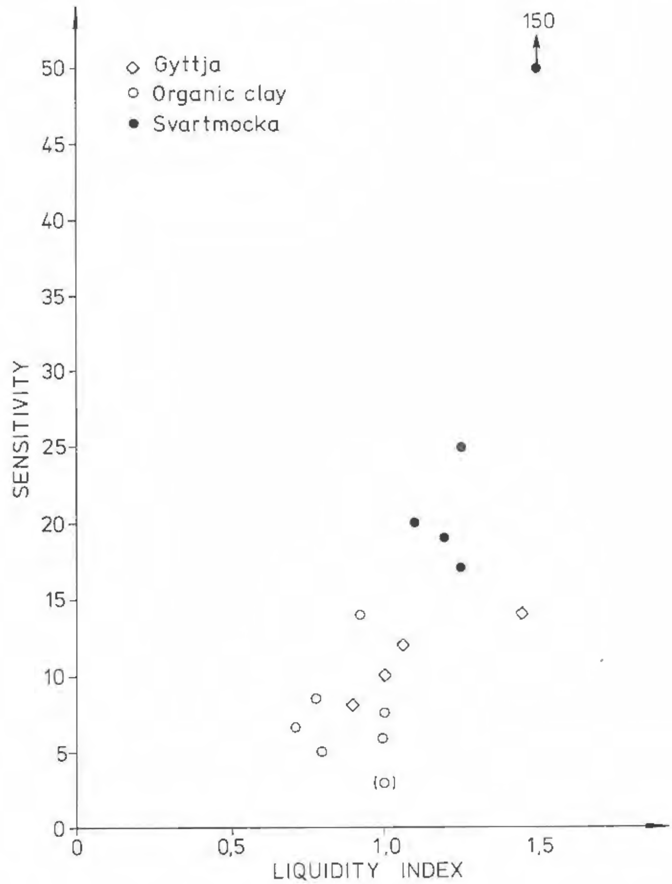


Fig. 58. Sensitivity as a function of liquidity index for organic clay, gyttja and "svartmocka".

11.6 Normalized undrained shear strength versus organic content and liquid limit

The undrained shear strength of a soil is largely a function of its stress history. For normally consolidated soils and only slightly overconsolidated soils, that is within the stress range where the undrained shear strength is the design strength, the undrained shear strength in a particular soil can often be expressed as a direct function of the preconsolidation pressure, e.g. Larsson 1980 and Jamiolkowski et al 1985.

In soft soils, this may be a too coarse description if a very wide range of effective stresses is considered as the structures of the soils may become seriously altered at the large volume changes that occur in them. For normal soil profiles and stress ranges this is usually a minor problem.

In organic soil and gyttjas, however, this problem is accentuated. The stress levels in situ are normally very low because of the geological history and the low densities. They are even so low that testing at in situ stresses with standard apparatuses may become impractical and construction on these soils often brings changes in effective stress levels of many times the original preconsolidation pressure. Another problem is also introduced in that the nature of the shear strength may change. At increasing stresses more and more of the load may be expected to be carried by the inorganic material in the soil and at the same time the limited strength in the fibres may be expected to play a smaller role in the shear strength. A significant decrease in normalized undrained shear strength with increasing preconsolidation pressures has been found for organic soils by Bergdahl et al (1987). In the present investigation was found that in the testing range 20-100 kPa a doubling of the preconsolidation pressure decreased the normalised undrained shear strength by about 10% in direct simple shear tests. This fact was used in the evaluation of shear strengths at in situ conditions for soils with extremely low preconsolidation stresses.

The results from the undrained triaxial tests and the direct simple shear tests have been normalized against the preconsolidation pressures and are plotted versus the organic content and the liquid limit in FIGS. 59 and 60.

It must be observed that the normalized values are valid for the in situ preconsolidation pressures which vary between 15 and 40 kPa and become lower at increasing preconsolidation pressures.

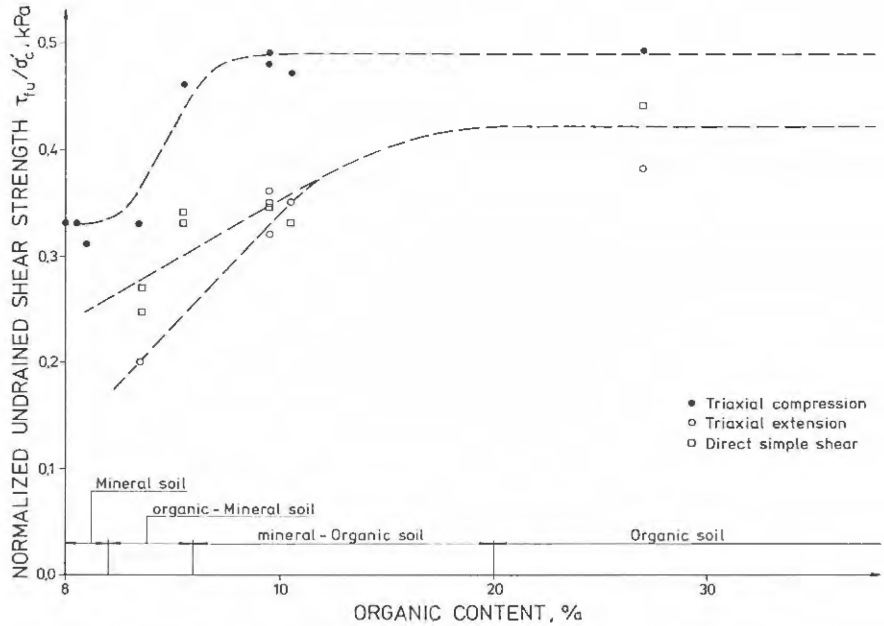


Fig. 59. Normalized undrained shear strength versus organic content.

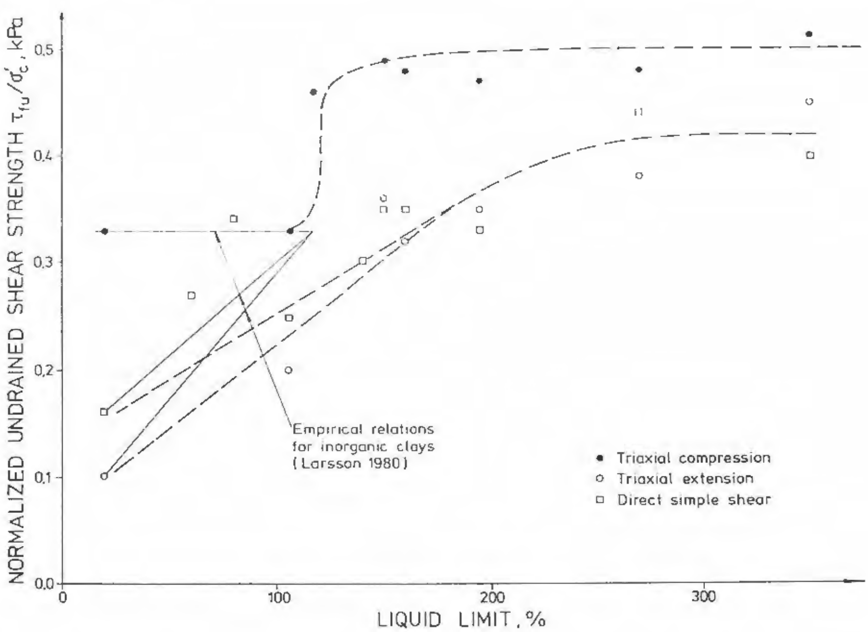


Fig. 60. Normalized undrained shear strength versus liquid limit.

In the first plot with normalized shear strength versus organic content it can be seen that the normalized active shear strength (triaxial compression) has a rapid transition from about 0.33 σ'_c to about 0.5 σ'_c when the organic content increases from about 2 to 6%. The value 0.33 corresponds to Mohr-Coulomb failure at $\phi' \approx 30^0$ and 0.5 corresponds to failure when the effective horizontal stress is zero.

Also the normalized passive shear strength (triaxial extension) and the strength in direct simple shear increase with organic content but somewhat more gradually. The anisotropic preconsolidation stresses ($K_{0nc} \approx 0.5$-0.6) would normally entail significantly higher strengths in direct simple shear than in triaxial extension, but in the organic soils these two strengths seem to be about equal. This may be attributed to the preferred horizontal orientation of the fibres.

In FIG. 60, where the normalized shear strengths are plotted versus the liquid limit, can be seen that in general the normalized strengths in the organic soils are higher than in the inorganic clays. At a limited organic content, however, the normalized shear strengths in triaxial extension and direct simple shear may be lower than the corresponding stengths in inorganic clays at compatible liquid limits. This may be attributed to those cases where the organic content is high enough to influence the plasticity of the soil, but where the fibre content is too low to affect the shear strength. That the liquid limit (or the plasticity index) is less suited as a basis for normalized undrained shear strength in soils with organic matter is also obvious from the results from the organic silt and silty clay with liquid limits of 60 and 80%. In these soils the normalized strengths are much higher than may be predicted from their liquid limits.

There is a considerable scatter in the results. This can be expected as neither the organic content nor the liquid limit can fully describe the nature of the soil. The scatter is also influenced by the fact that very small values of both shear strengths and preconsolidation pressures, which in relative numbers have unusually large scatter, are compared and thereby the scatter in these parameters is magnified.

The normalized active shear strength from the Finnish investigation was generally somewhat lower. The reason for this is not known. In both cases the shear strengths were evaluated at similarly slow rates. Possible differences may exist in the testing procedures and the evaluation of the preconsolidation pressures.

11.7 Normalized shear strength values from field vane tests and fall cone tests

According to Swedish experience, the field vane tests and the fall cone tests normally give compatible results. In fact, the evaluation of the fall cone test is calibrated against the field vane test, (Hansbo 1957). Differences in the results are mainly associated with inhomogeneous soils, poor quality of the samples and excessive stress relief in samples from greater depths (>10 à 15 m).

The uncorrected results generally follow Hansbo's relation $\tau_{vane} \approx \sigma'_c \cdot 0.45\ w_L$. This relation has been found to be valid for liquid limits up to about 200%. It is often used to obtain a rough estimate of the preconsolidation pressure from the routine tests and alternatively to obtain an estimate of normal shear strength values on the basis of the preconsolidation pressure and the liquid limit. At a liquid limit around 200% the relation τ_{vane}/σ'_c tends to curve towards a maximum value of about 0.8 - 0.9, but the empirical experience for higher liquid limits is very limited, (Larsson et al 1984).

The Hansbo relation is also used to estimate the applicability of the normal correction factors to the results from field vane tests and fall cone tests. If the shear strength values obtained from the tests are significantly higher than this relation yields, there is a great risk that the strength values will have to be reduced more than the case in application of the general correction factor. If the shear strength values from the tests are considerably lower than those obtained by the Hansbo relation, it is probable that supplementary or more qualified tests will give higher shear strengths.

The relations obtained for organic clays and gyttjas in the present investigation and the Finnish tests are plotted in FIG. 61 together with results from tests on "svartmocka".

From this figure it can be seen that the results from the present investigation in organic clay and gyttja closely follow the Hansbo relation up to liquid limits around 200%. The relation of $\tau/\sigma'_c \approx 0.85$ at a liquid limit around 350% may also be considered normal. Two results from the Finnish investigation were on the low side and especially so in the case of the relation for the Lokalahti site with an organic content of 13-15% and a liquid limit of around 246%.

The results from tests in "svartmocka" in most cases were high or very high, indicating that the corrected strength values probably have to be further reduced before they can be used for design calculations.

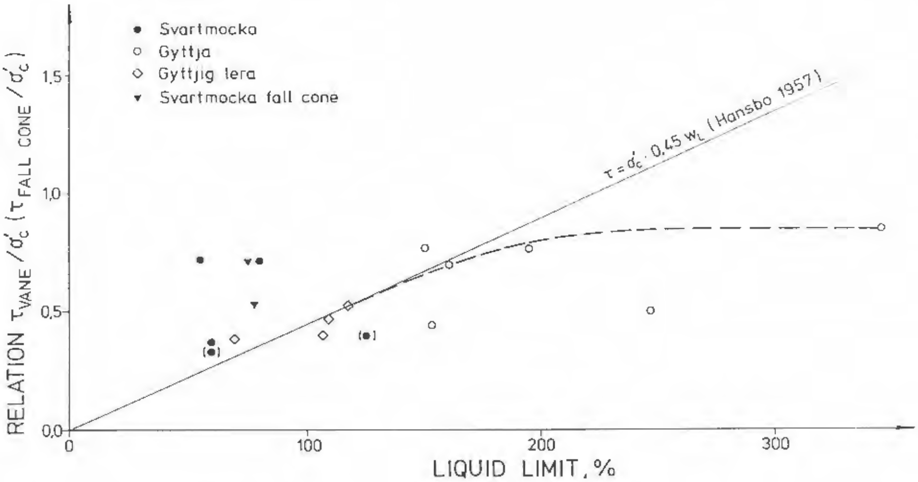


Fig. 61. Normalized shear strength values obtained in field vane tests (or fall cone tests) in organic clay, gyttja and "svartmocka".

11.8 Correction factors for evaluation of undrained shear strength from field vane tests and fall cone tests

Results from both field vane tests and fall cone tests have to be corrected in order to obtain relevant undrained shear strengths. Internationally, this correction is often based on the plasticity index of the soil, but in Sweden the liquid limit is used. The general correction recommended by the Swedish Geotechnical Institute is

$$\tau_{fu} = \mu \cdot \tau$$

where τ_{fu} = undrained shear strength
μ = correction factor
τ = strength value obtained in field vane or fall cone test

The correction factor μ is calculated from

$$\mu = \left(\frac{0.43}{w_L}\right)^{0.45} \qquad 1.2 \geq \mu \geq 0.5$$

The empirical background for this correction and associated rules for estimation of its applicability are described in detail by Larsson et al (1984).

The results from the Finnish investigation and some of the results in the present investigation were included in the empirical data base for the recommended correction.

In FIG. 62 the deduced correction factors to be applied to the results from the field vane tests in the different test fields are plotted. The correction factors have been calculated from comparisons with full scale failures in the field or from comparisons with the average of active and passive triaxial test and direct simple shear tests and in one case only direct simple shear tests depending on the reference data that were available.

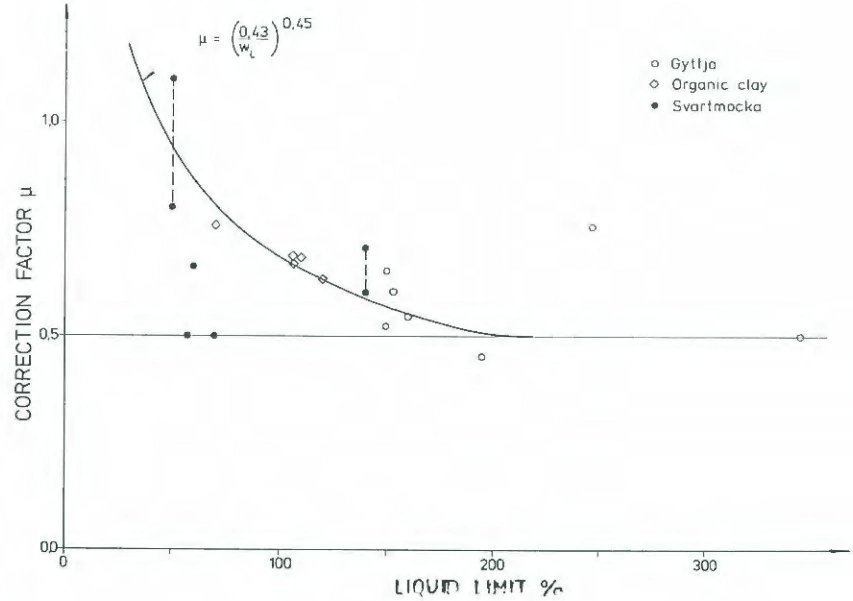


Fig. 62. Deduced correction factors to be applied to the shear strength values obtained by field vane tests in the various test fields.

The deduced correction factors closely conform with the general correction factors except for a few cases.

For the organic soil at Lokalahti the shear strength values do not have to be reduced as much as the general correction specifies. This is in accordance with what could be expected from the low value of τ/σ'_c in comparison to Hansbos relation.

For the "svartmockas" from Umeå and Gideåbacka, the shear strength values had to be reduced more and much more respectively than the general correction factors specify. Also this is in accordance with the check versus Hansbo's relation, which indicated a great risk that this would be the case.

The procedure recommended by SGI in 1984 for correction and estimation of the relevance for shear strength values from field vane tests thus seems to give satisfactory results in all the investigated soils. It has to be stressed, however, that the relevance of the results from the particular type of soil at the site has to be checked before the corrected strengths are used in design calculations.

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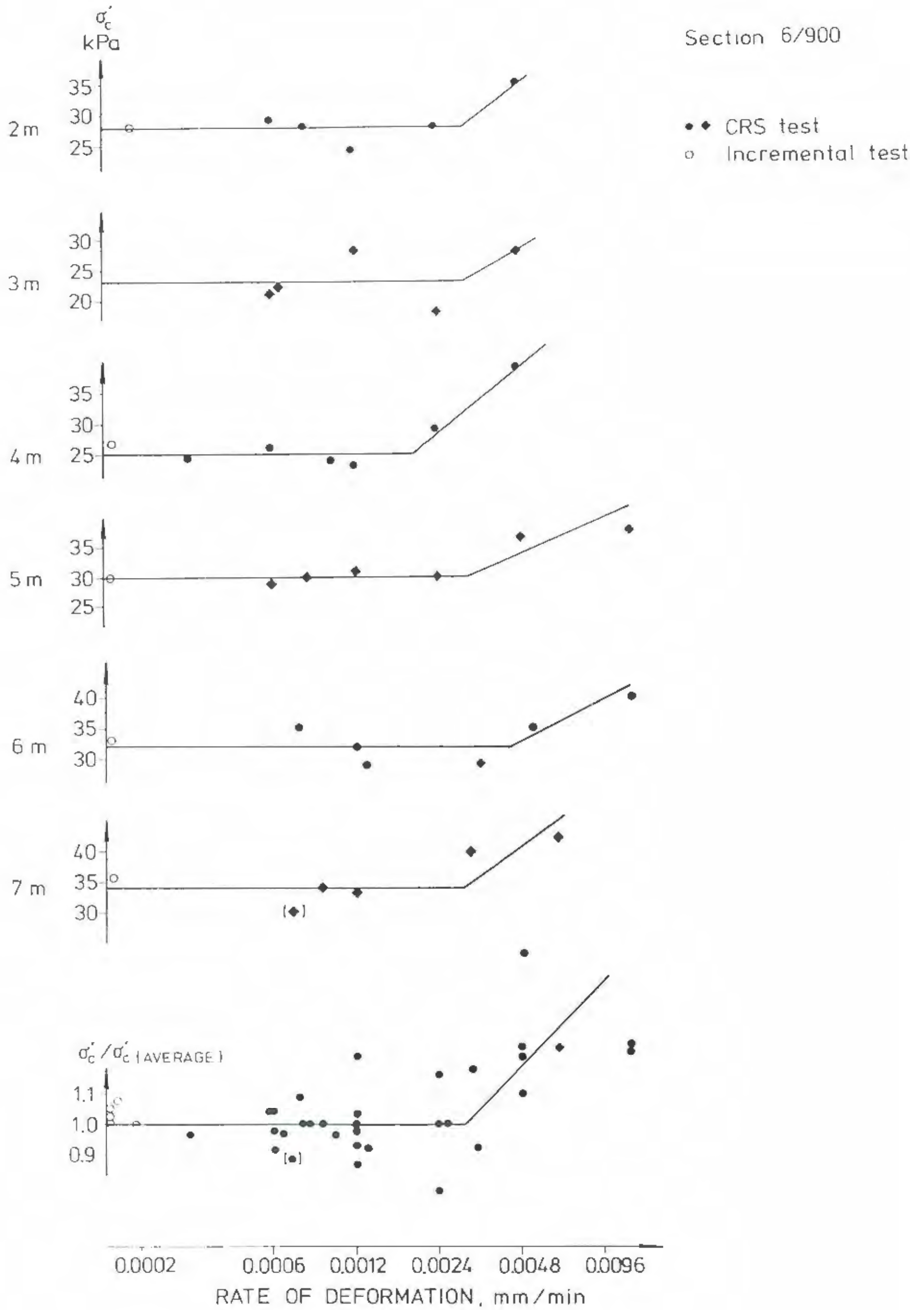
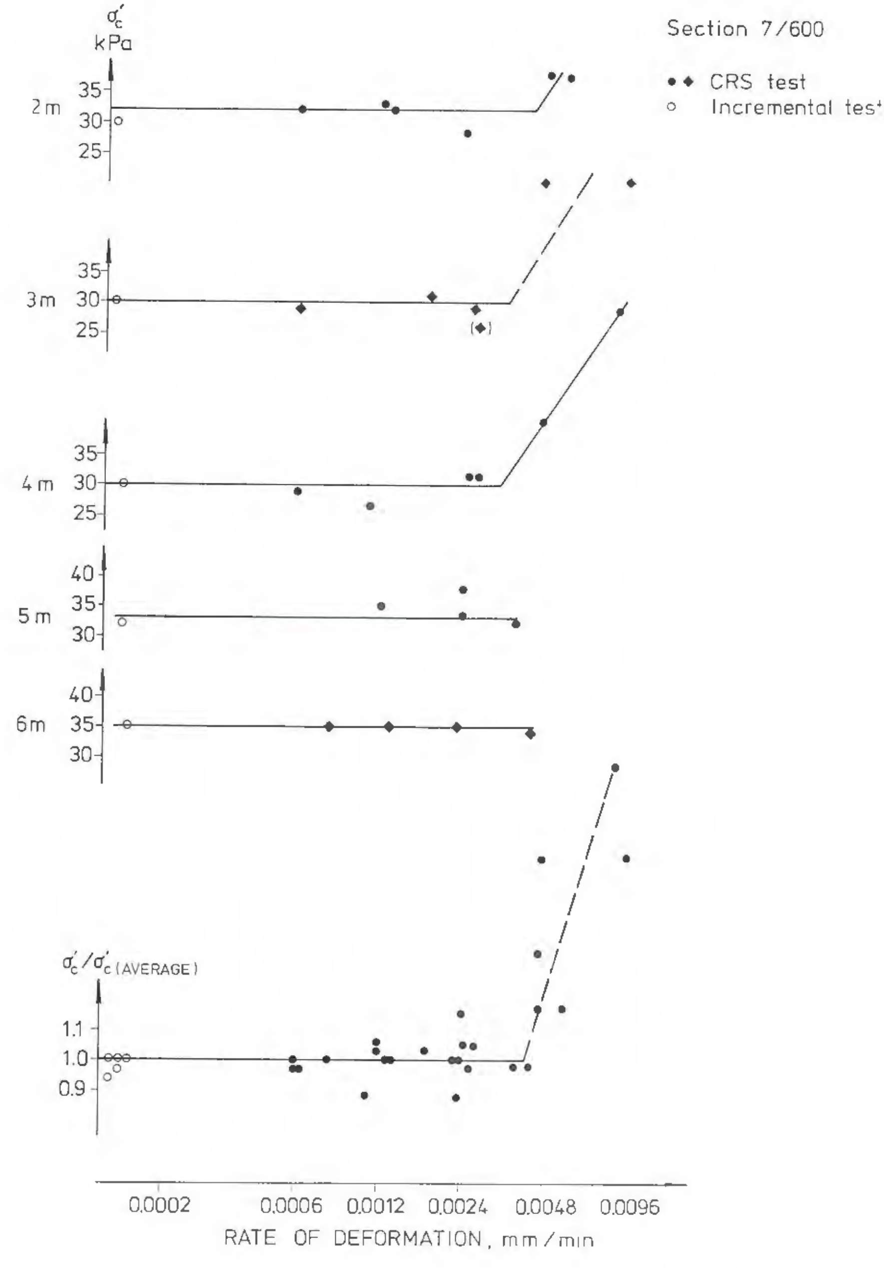
APPENDIX A

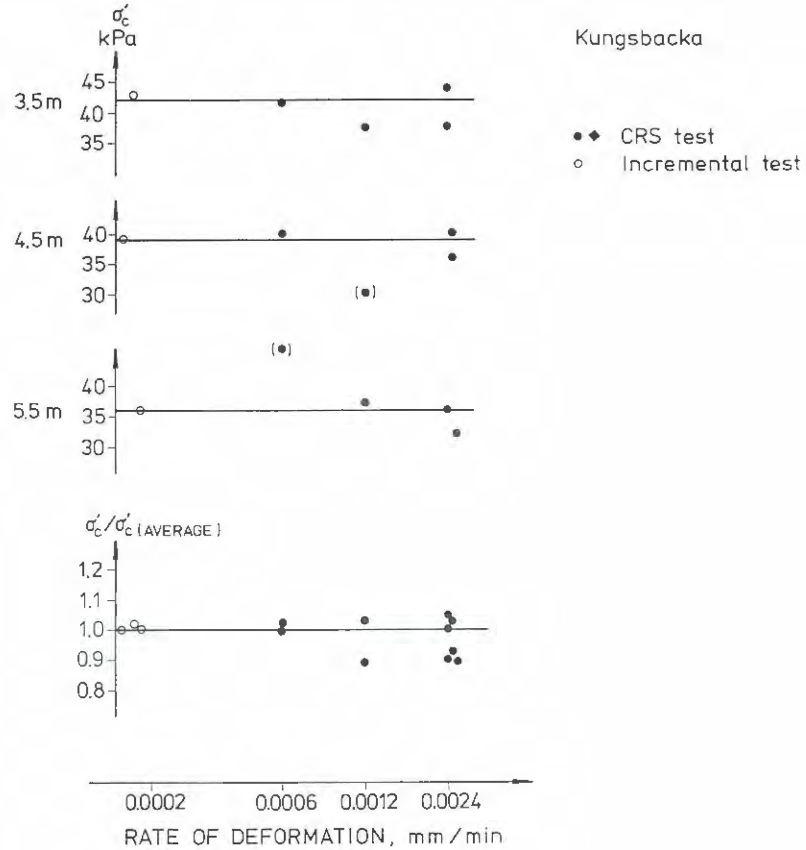
Evaluated preconsolidation pressures from CRS- and incremental oedometer tests as a function of rate of deformation.



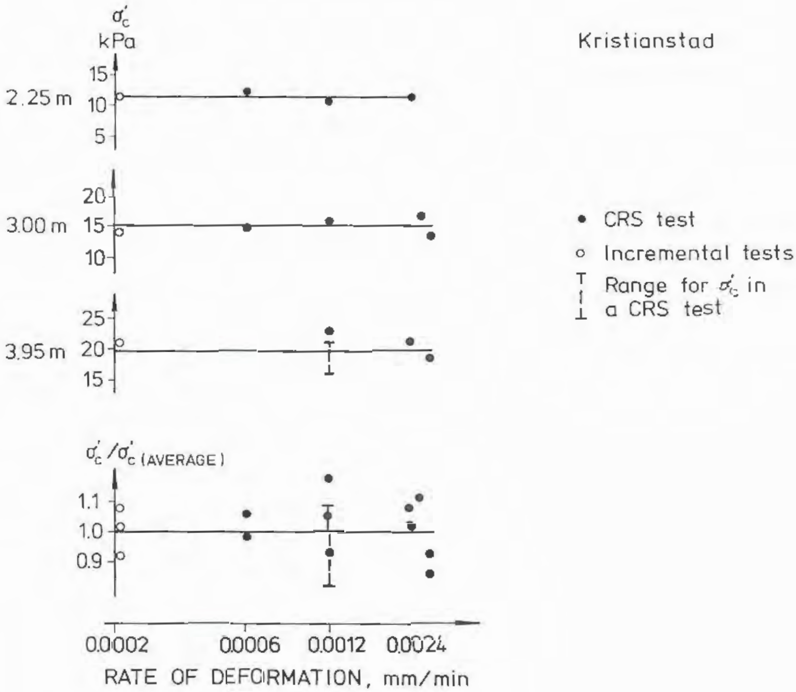
σ'c kPa
Section 6/900
CRS test
Incremental test
2 m
3 m
4 m
5 m
6 m
7 m
σ'c / σ'c (AVERAGE)
0.0002
0.0006
0.0012
0.0024
0.0048
0.0096
RATE OF DEFORMATION, mm/min



σ_c' kPa
Section 7/600
CRS test
Incremental test
2 m
3 m
4 m
5 m
6 m
35
30
25
40
σ_c' / σ_c' (AVERAGE)
1.1
1.0
0.9
0.0002
0.0006
0.0012
0.0024
0.0048
0.0096
RATE OF DEFORMATION, mm/min



Kungsbacka
CRS test
Incremental test
σ_c' kPa
3.5 m
4.5 m
5.5 m
45
40
35
30
σ_c'/σ_c' (AVERAGE)
1.2
1.1
1.0
0.9
0.8
0.0002
0.0006
0.0012
0.0024
RATE OF DEFORMATION, mm/min

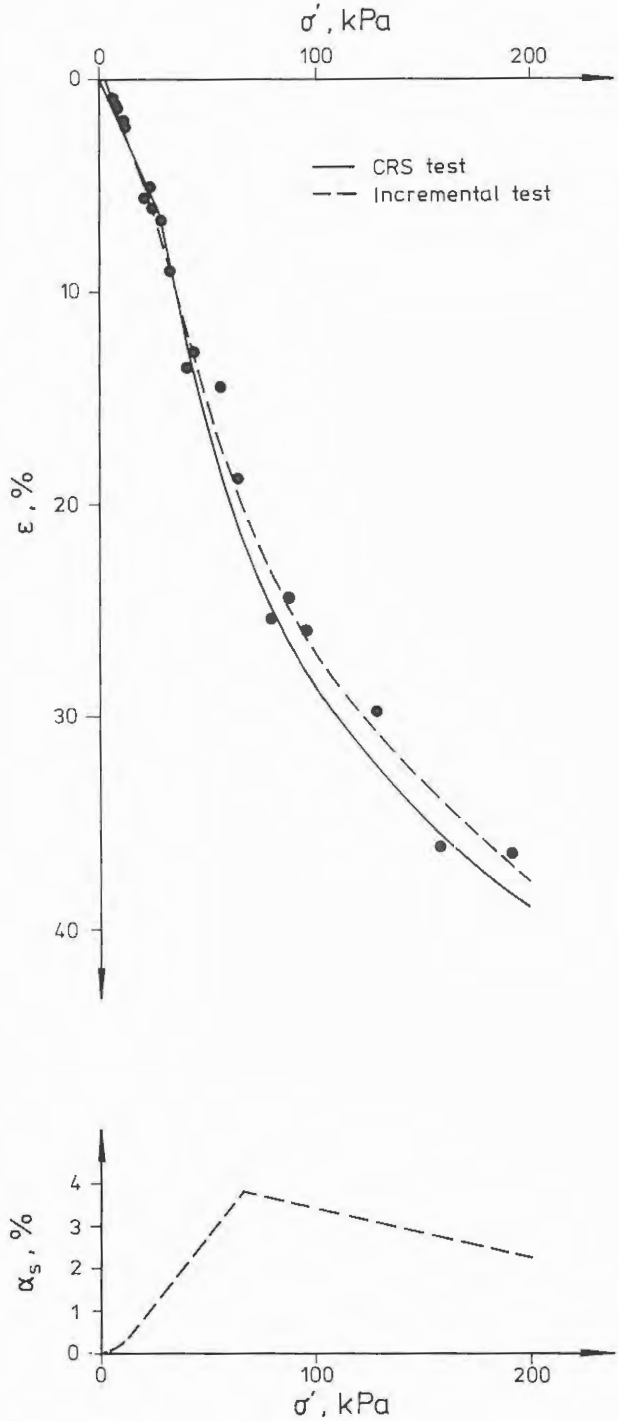


Kristianstad
σ'_c kPa
2.25 m
3.00 m
3.95 m
15
10
5
20
15
10
25
20
15
$\sigma'_c / \sigma'_{c\,(AVERAGE)}$
1.1
1.0
0.9
0.0002
0.0006
0.0012
0.0024
RATE OF DEFORMATION, mm/min
• CRS test
o Incremental tests
Range for σ'_c in a CRS test

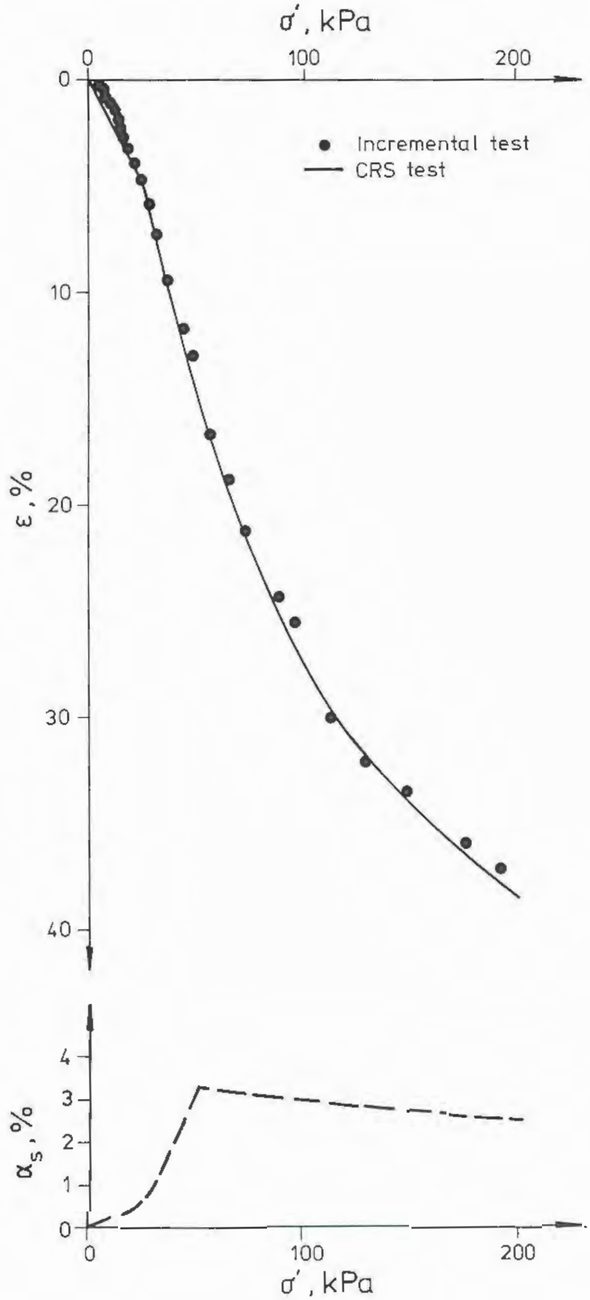
APPENDIX B

Correlations between stress-strain relations obtained in CRS- and incremental oedometer tests together with evaluated coefficients of secondary consolidation from the incremental tests.

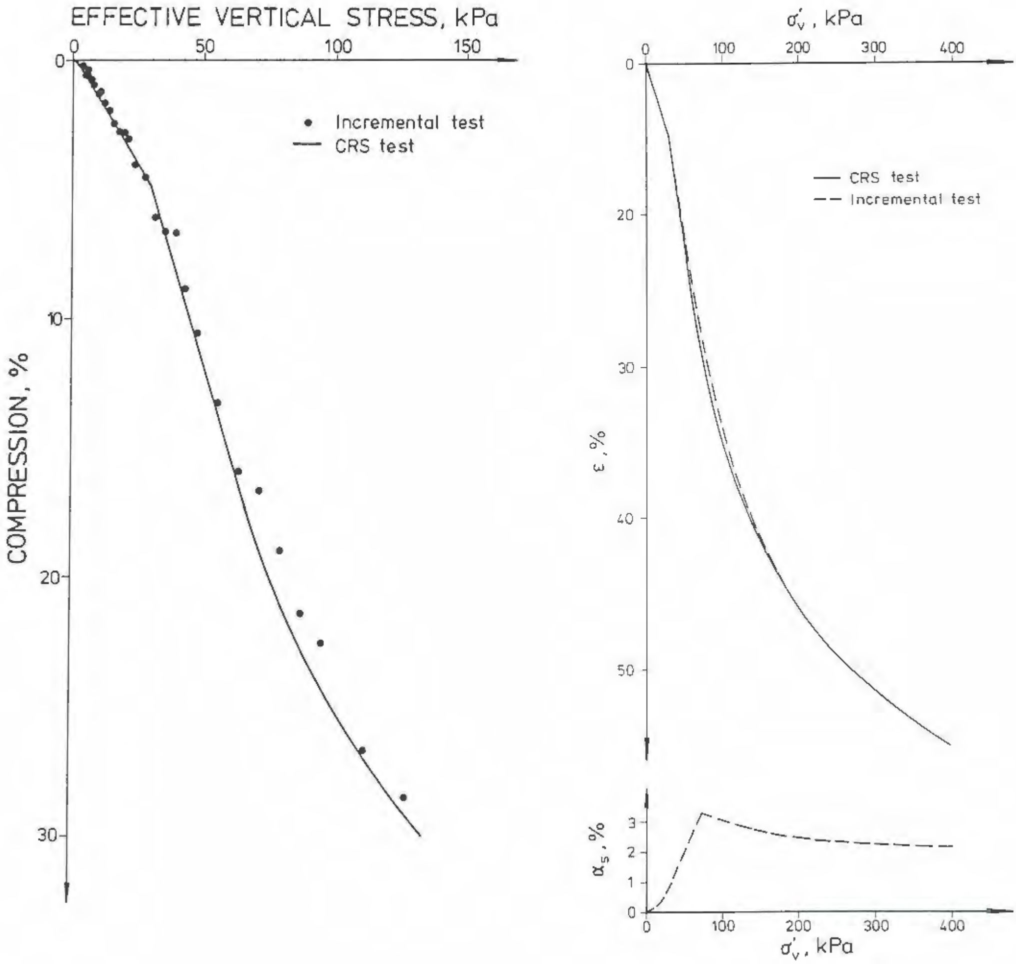
The stress strain relations from CRS- tests have been adjusted for rate effects as described in Section 7.2.4.



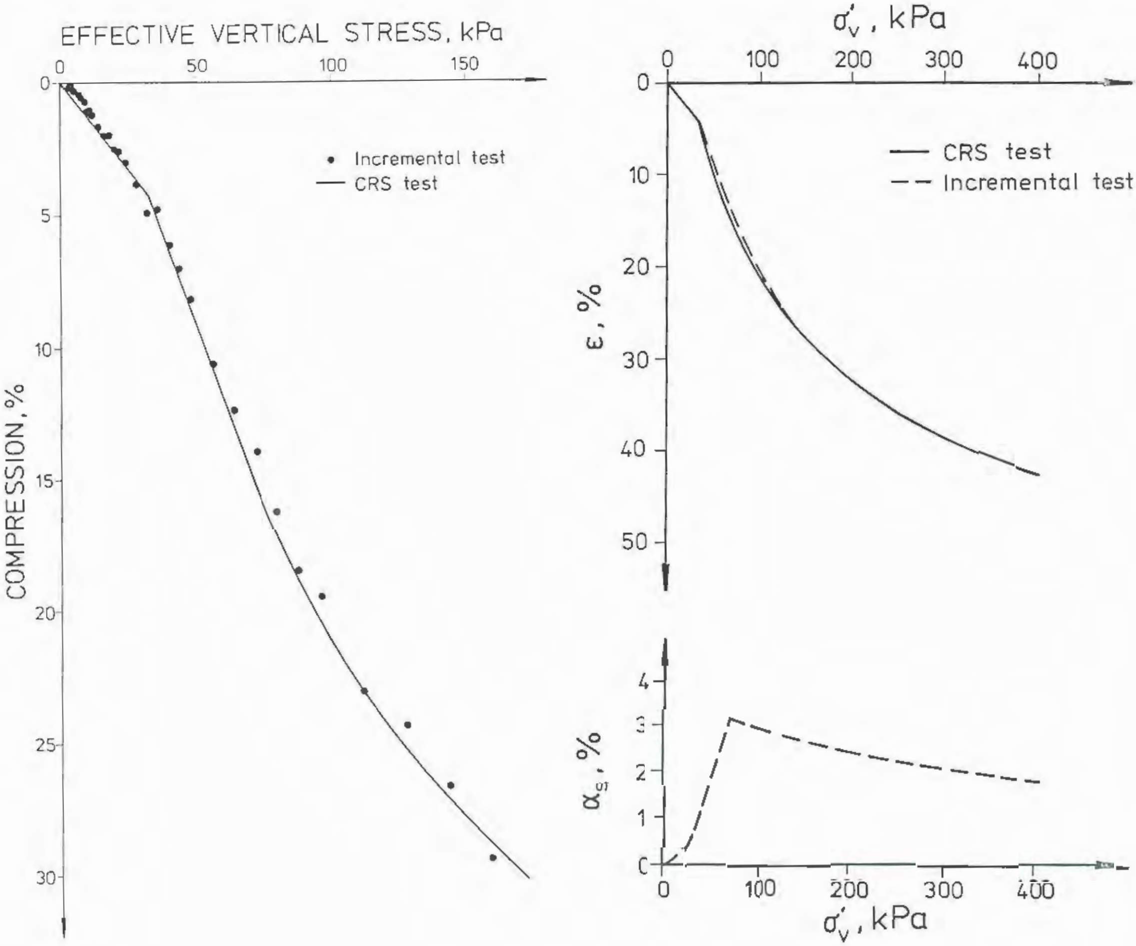
Section 6/900 2 m



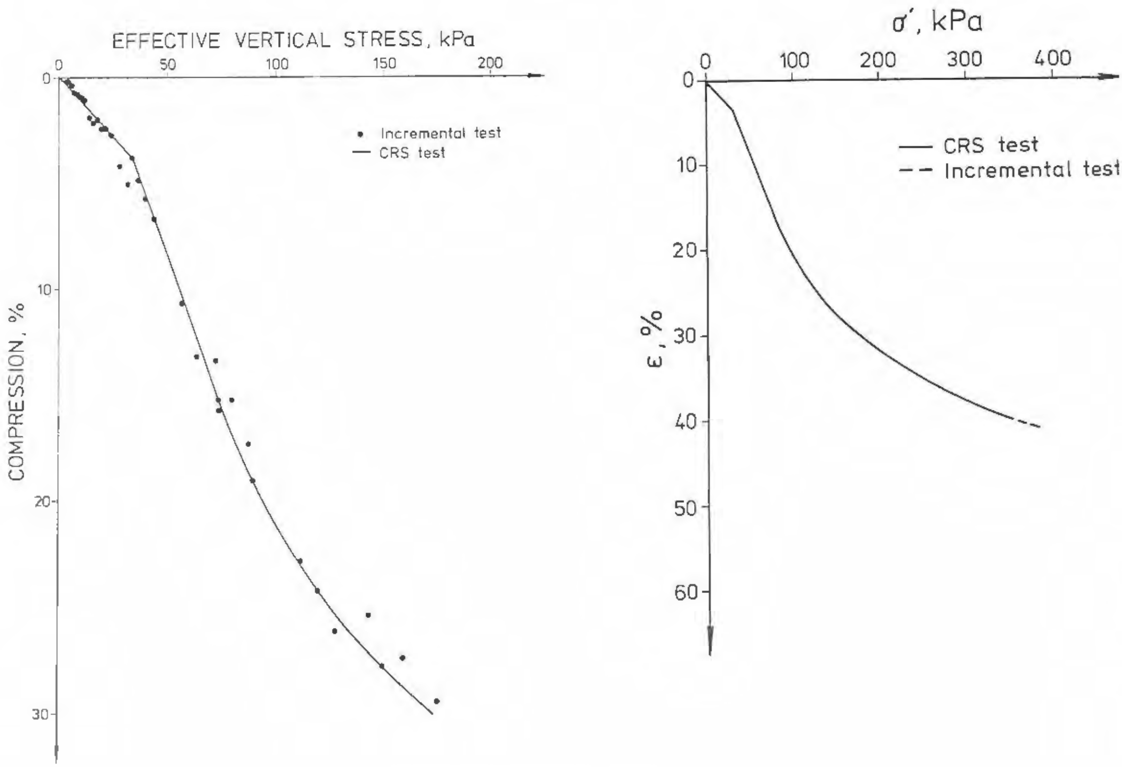
Section 6/900 4 m



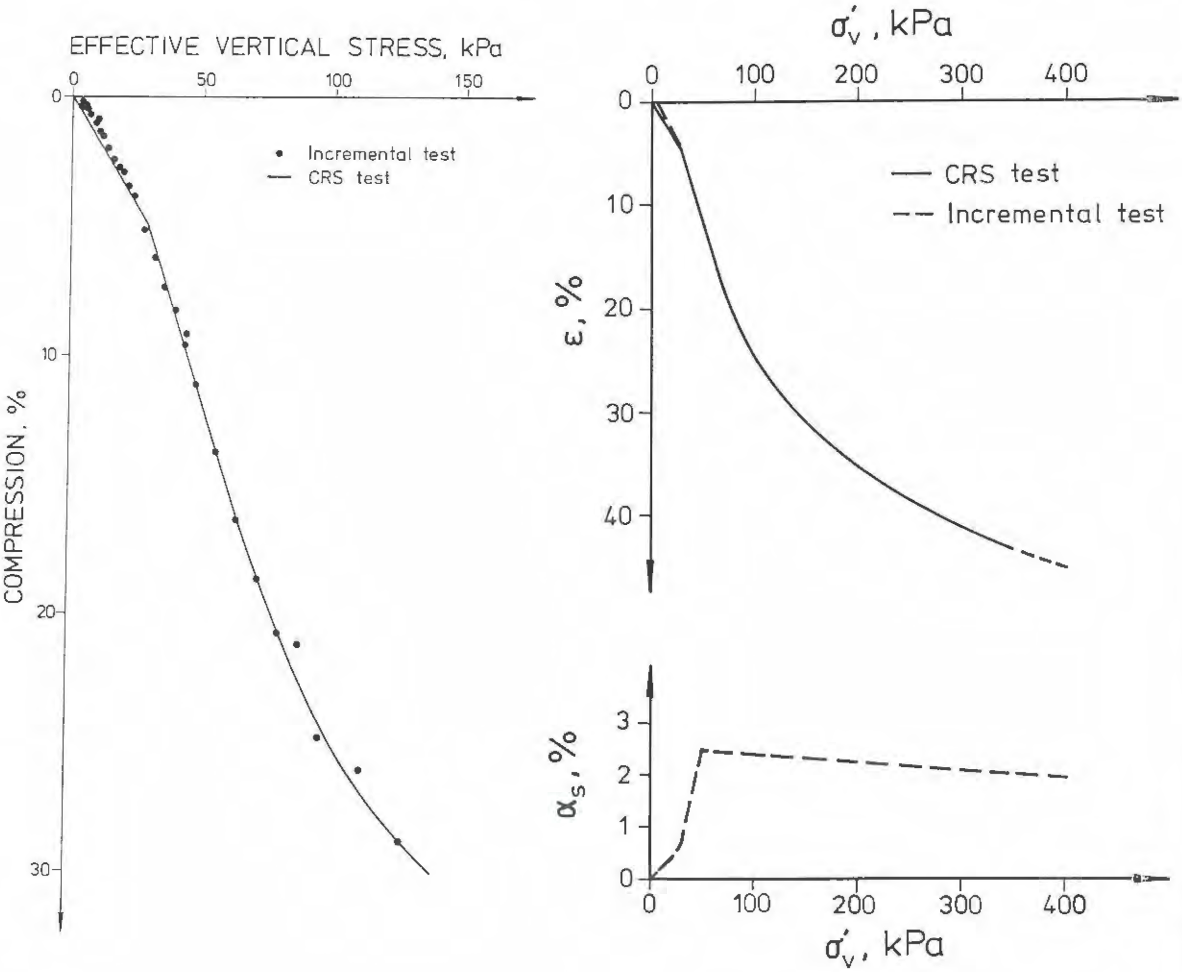
Section 6/900 5 m



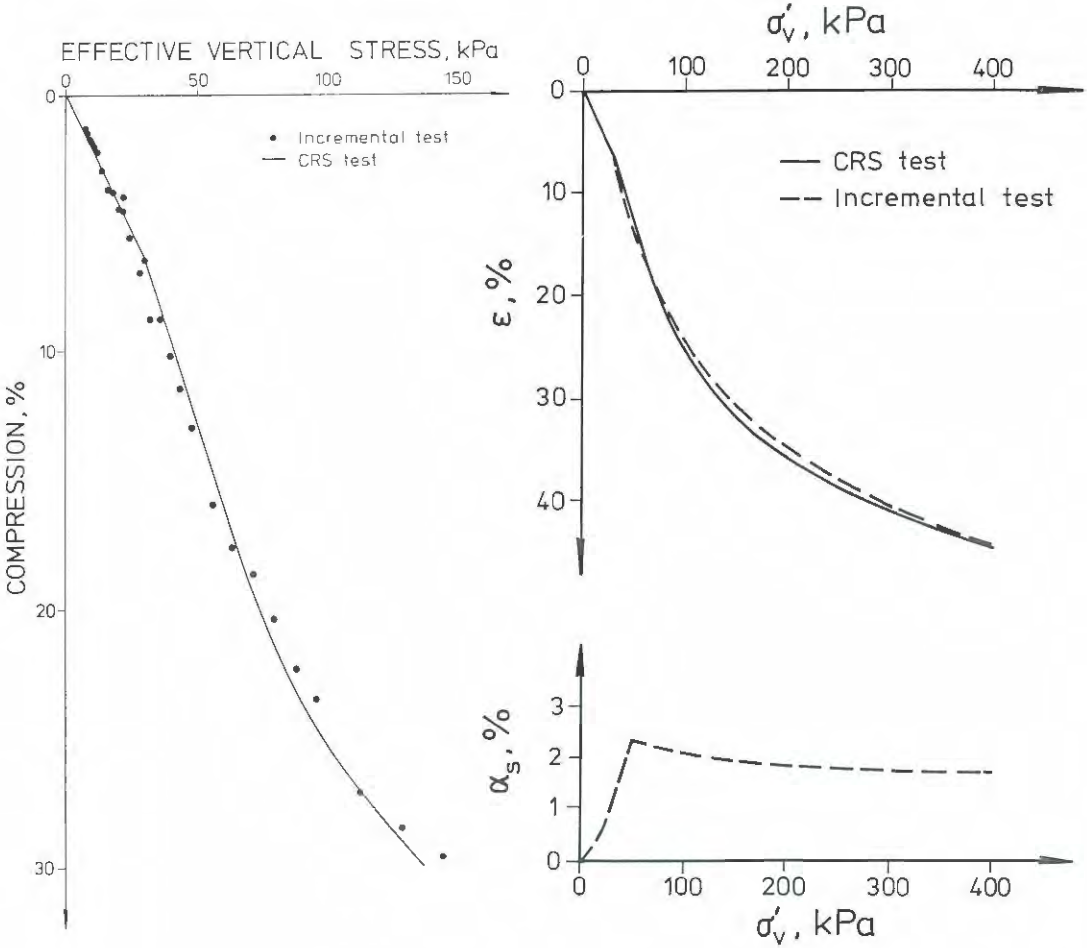
Section 6/900 6 m



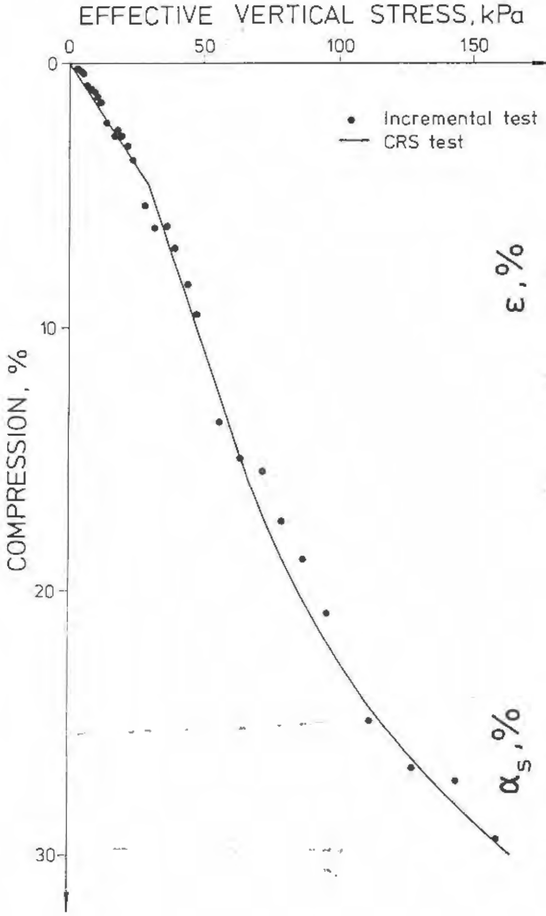
Section 6/900 7 m

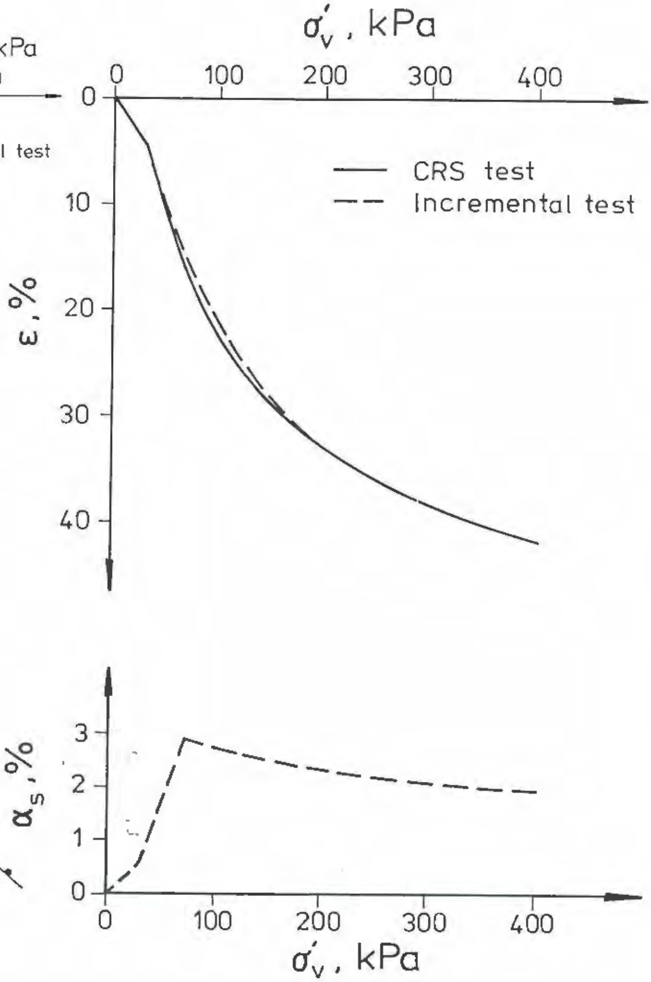


Section 7/600 2 m

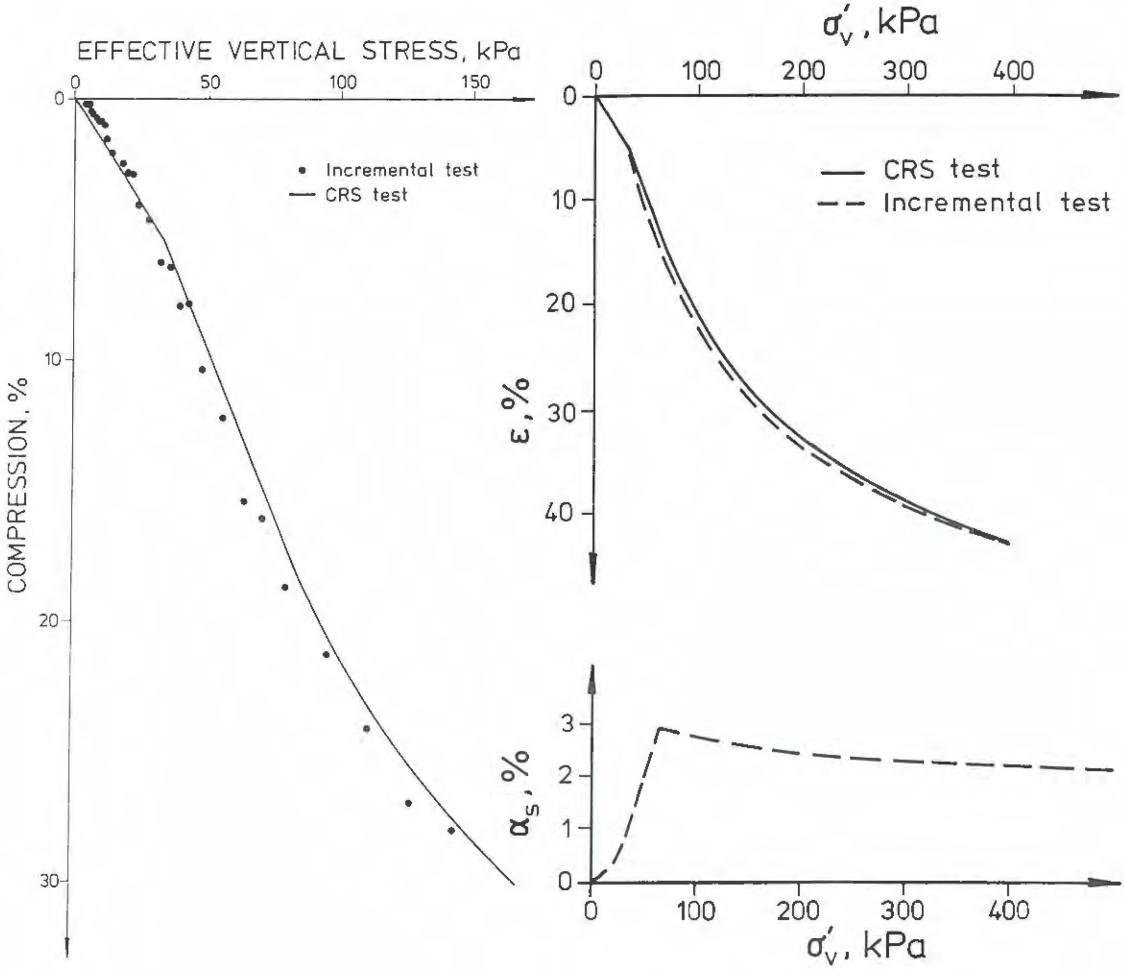


Section 7/600 3 m

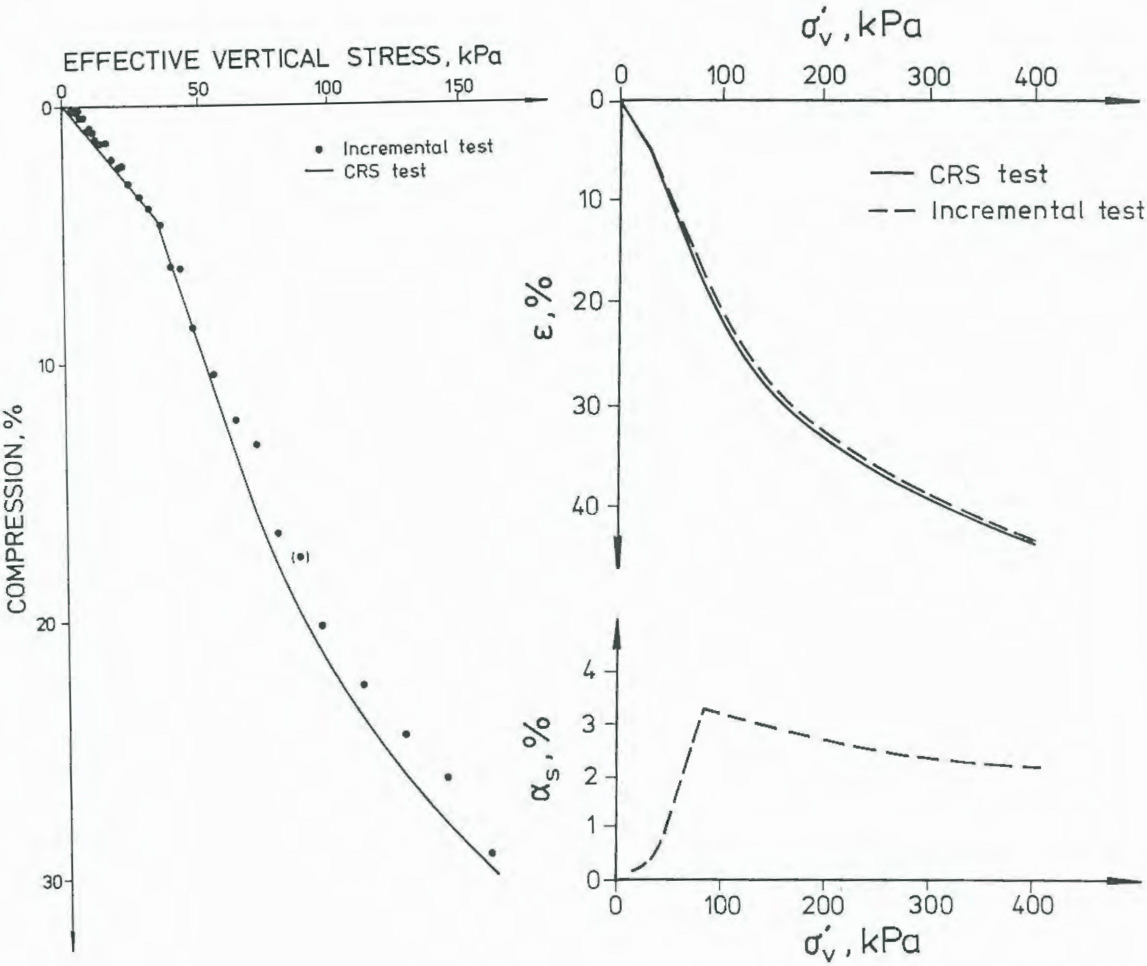




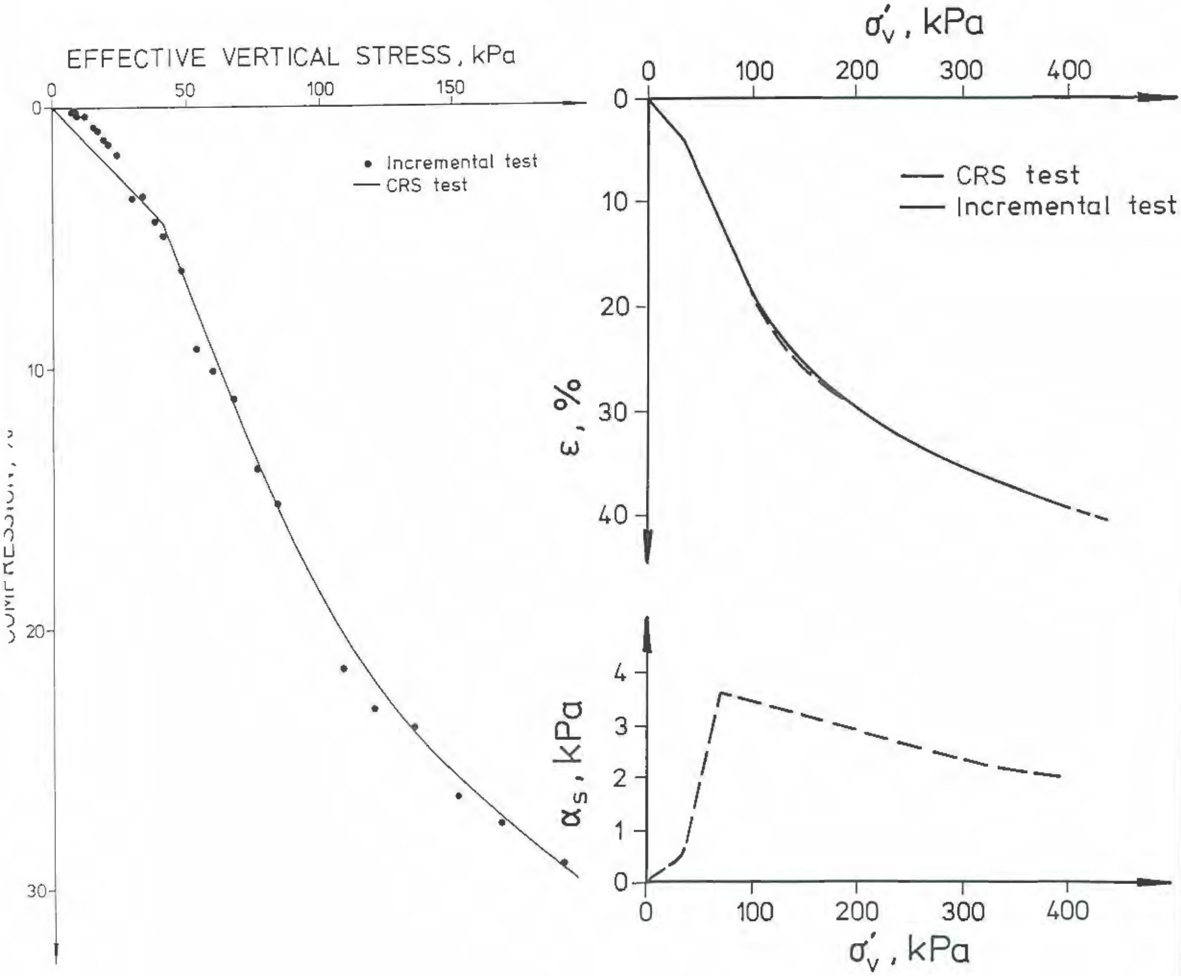
Section 7/600 4 m



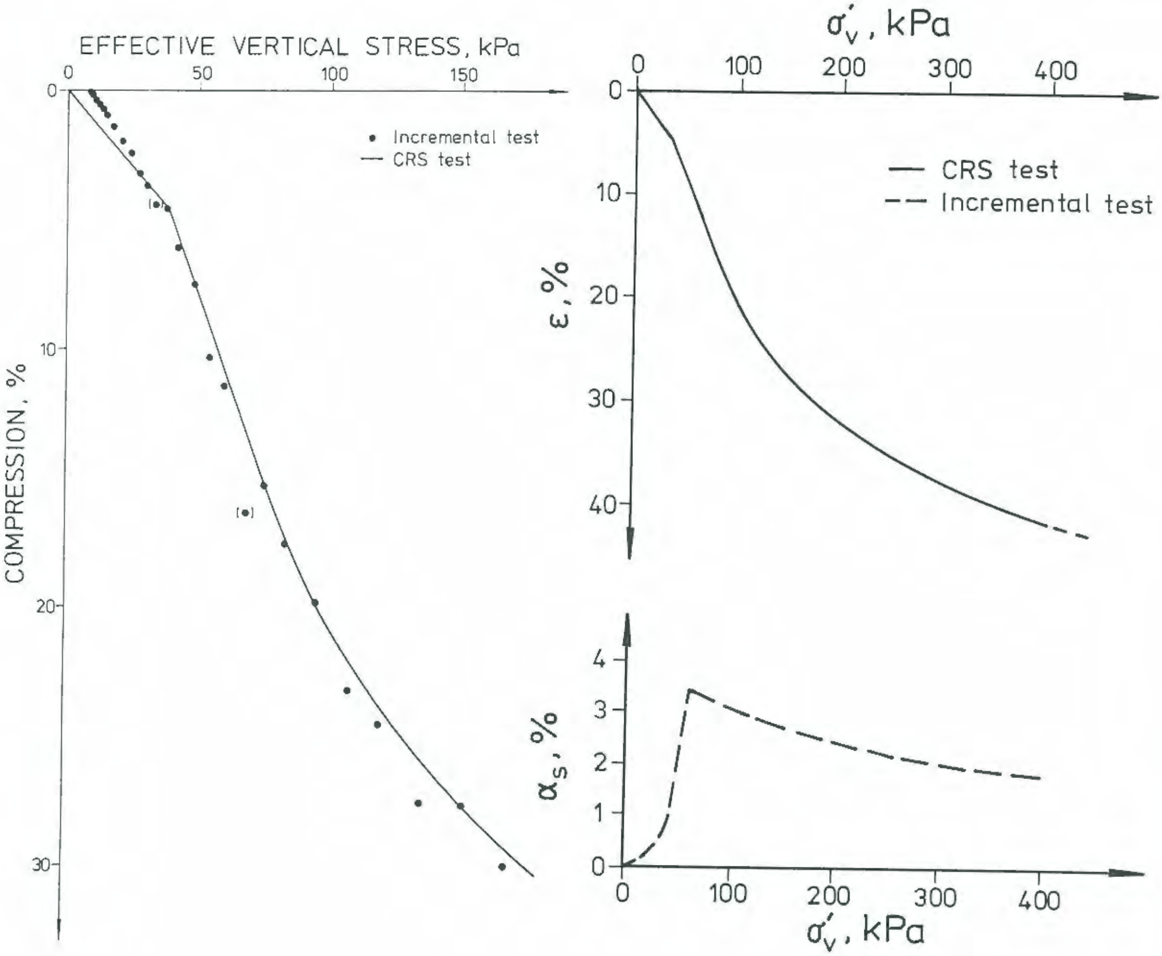
Section 7/600 5 m



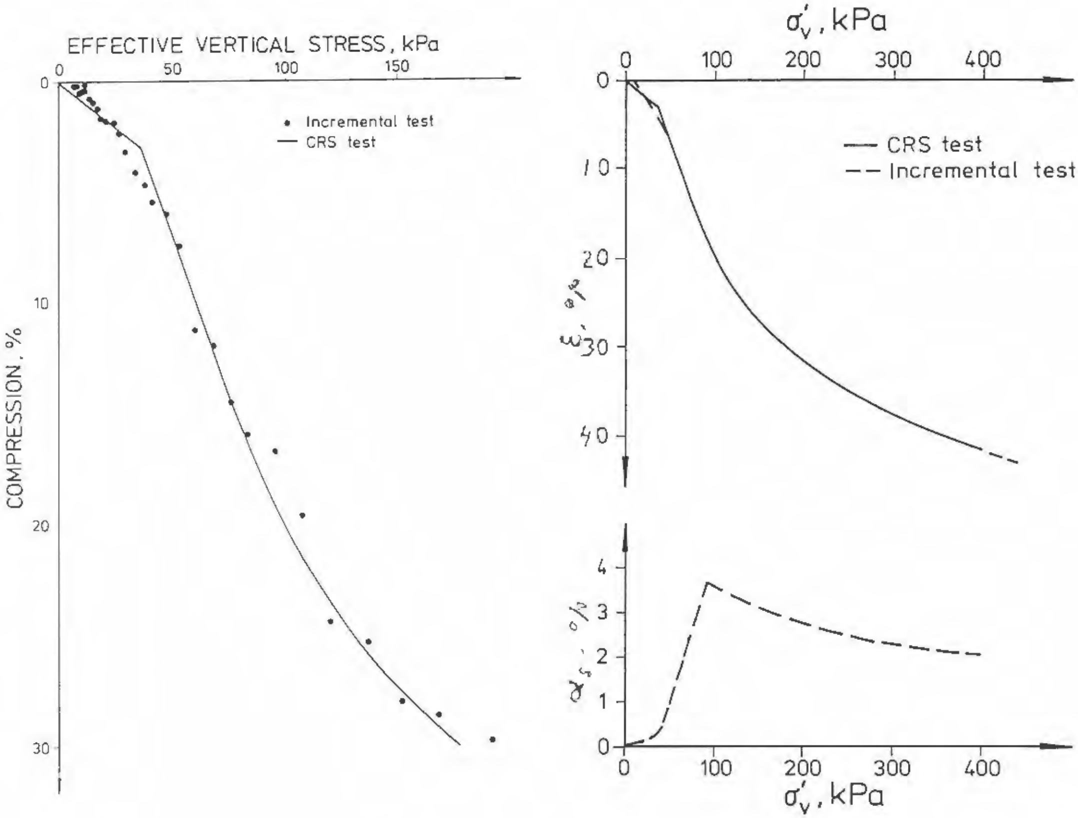
Section 7/600 6 m



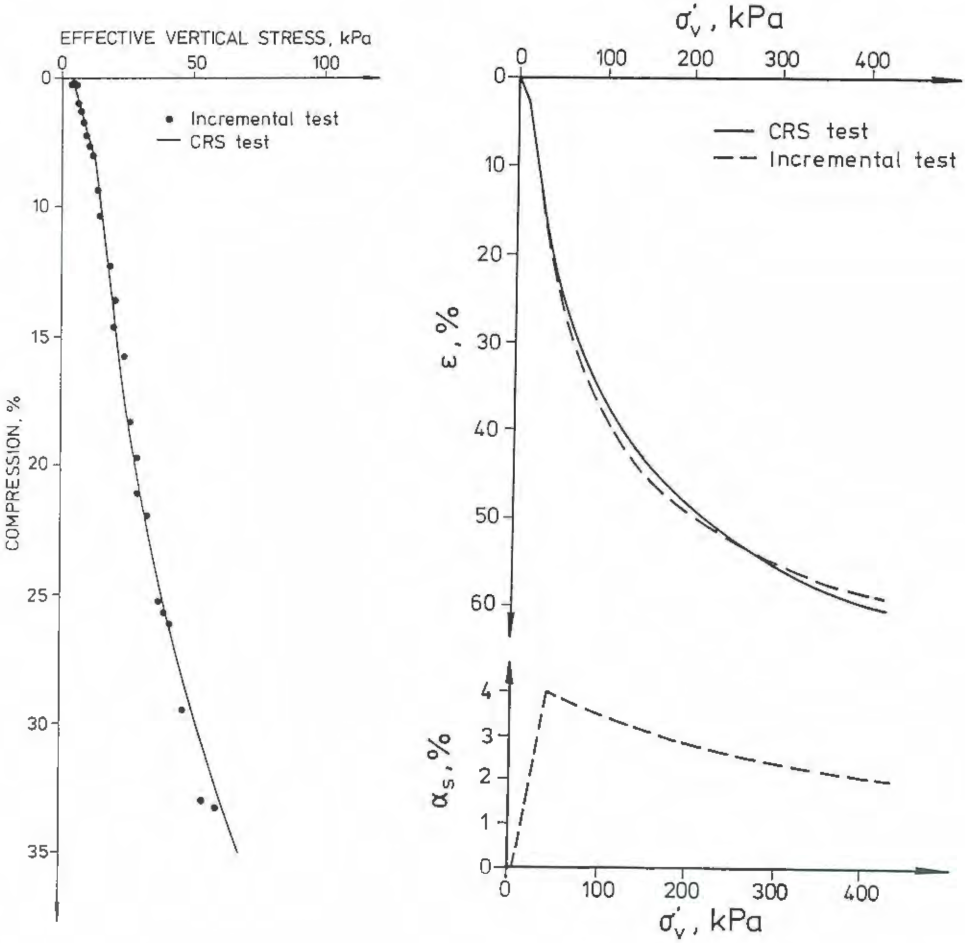
Kungsbacka 3.5 m



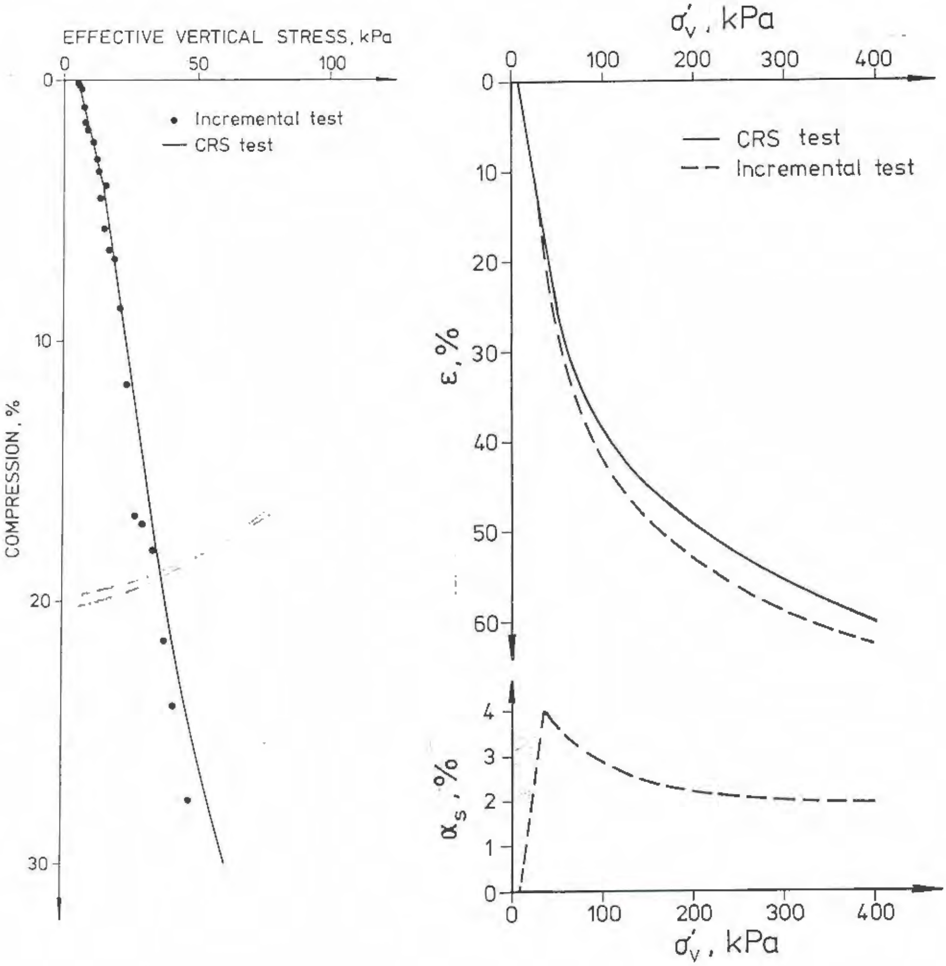
Kungsbacka 4.5 m



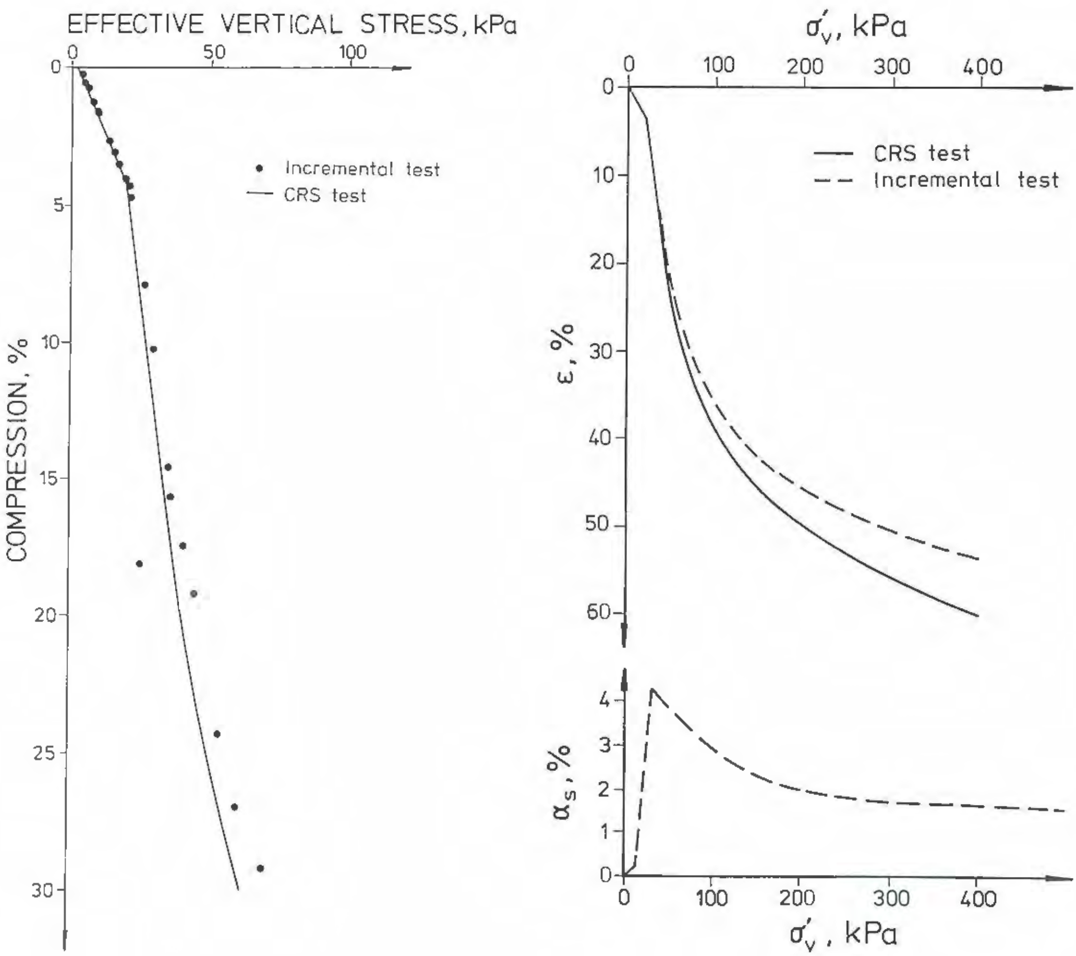
Kungsbacka 5.5 m



Kristianstad 2.25 m



Kristianstad 3.0 m



Kristianstad 3.9 m

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4.	**Basic behaviour of Scandinavian soft clays.** R. Larsson	1977
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