STATENS GEOTEKNISKA INSTITUT
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RAPPORT
REPORT **No 40**

SHEAR MODULI IN SCANDINAVIAN CLAYS

Measurement of initial shear modulus with seismic cones

Empirical correlations for the initial shear modulus in clay

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PREFACE

This report concerns determination of the initial (maximum) shear modulus at small strains in clays.

The report is intended for engineers dealing with the task of estimating shear deformations in both static and cyclic loading conditions.

The purpose of the report is to improve the estimation of the initial shear modulus both by testing and by applying empirical correlations. It describes how this parameter can be measured in a rational way by using a seismic cone in connection with cone penetration tests in ordinary site investigations and empirical relations for the initial shear moduli obtained from tests in Scandinavian clays are also given.

The investigation is part of a larger project concerning the use of new in-situ methods for determination of stratigraphy and properties in fine-grained loose to medium stiff soils.

Other parts of the project have been reported in the following publications:

- New in situ methods for investigation of stratigraphy and properties in soil profiles, Larsson and Sällfors (1987). Swedish Geotechnical Institute, Information No. 5. (In Swedish)

- Laboratory calibration of cones for combined cone penetration testing and pore pressure sounding, Larsson and Eskilsson (1988). Swedish Geotechnical Institute, Varia 223. (In Swedish)

- Dilatometer tests in clay, Larsson and Eskilsson (1989a). Swedish Geotechnical Institute, Varia No. 243. (In Swedish)

- Dilatometer tests in organic soils, Larsson and Eskilsson (1989b). Swedish Geotechnical Institute, Varia No. 258. (In Swedish)

- The dilatometer test; an in situ method for determination of stratigraphy and properties in soils, Larsson (1989). Swedish Geotechnical Institute, Information No. 10. (In Swedish)

- Calibration of Piezocones for Investigation of Soft Soils and Demands for Accuracy of the Equipments, Mulabdić et al (1990). Swedish Geotechnical Institute, Varia No. 270.

The project is supported by grants from the Swedish Council for Building Research, the Swedish Road Administration and the Civil Engineering Institute in Zagreb, Yugoslavia and by internal funds at the Swedish Geotechnical Institute.

The authors wish to acknowledge the efforts made by Hogentogler & Co to satisfy their various wishes.

Linköping, November 1990

Rolf Larsson Mensur Mulabdić

CONTENTS

SUMMARY

Vibrations from traffic and machine foundations, more complicated constructions that are very sensitive even to small deformations and new offshore and near-shore constructions subjected to wave-loads have led to an increasing demand for an accurate estimation of the shear deformations in the soil beneath.

The shear stress-strain relation for a soil is usually assumed to have a hyperbolic or a modified hyperbolic shape and the key parameter for estimation of the shear deformations is then the initial "elastic" shear modulus at small strains, G_0.

The shear stress-strain relations in soils have mainly been studied in the laboratory. Comparisons with field tests, however, have shown that there are considerable disturbance and time-effects in the laboratory tests. The initial shear modulus should therefore preferably be determined by field tests.

A large number of investigations have shown that the various methods of field testing with cross-hole and down-hole techniques normally give compatible results. Compatible results are also obtained when a seismometer is incorporated in an ordinary CPT probe and the shear wave velocity is measured at regular intervals as the penetration test proceeds. The shear wave is then normally created by a hammer-blow on a steel-bar pressed against the soil by the weight of the drill rig. This is a cost-effective method and the equipment is commercially available.

From the results in this investigation, it appears that the shear deformations created in this way may exceed the "elastic" range of deformations in the upper parts of soft soil profiles. An evaluation of the shear strain in the test therefore ought to be made and the results corrected for excessive shear strains. Tentatively, the initial shear moduli obtained at strains larger than $1 \cdot 10^{-6}$ should be corrected according to

- $G_0 = G_{measured} \cdot C$

The correction factor C is shown in FIG. 1. This correction is coarse, however, and the shear strains should preferrably be kept as small as possible.

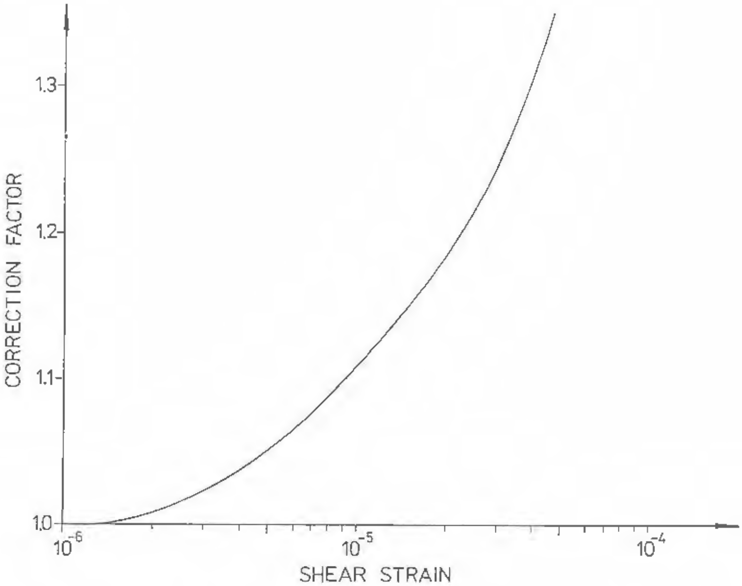


Fig. 1. Correction factor for shear moduli measured at strains larger than 10^{-6}.

From the results of numerous laboratory tests on various clays, Hardin (1978) suggested that G_0 could be determined empirically from

$$G_0 = 625 \cdot OCR^{k'} \cdot (p' \cdot p_a)^{0.5} / (0.3 + 0.7\, e^2)$$

where p' = mean effective stress = $\sigma'_v \cdot (1+2\,K_0)/3)$ kPa
(σ'_v = effective vertical stress and
K_0 = coefficient of earth pressure at rest)
p_a = atmospheric pressure = 98.1 kPa
e = void ratio
OCR = overconsolidation ratio
k' = a factor related to the plasticity index of the soil according to FIG. 2.

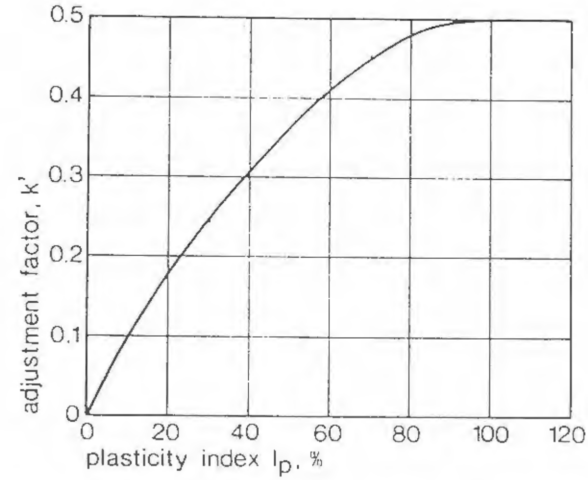


Fig. 2. Overconsolidation adjustment factor k′ versus plasticity index I_p, after Hardin 1978.

This relation is found to give results that well describe the general variation of G_0 with stress and void ratio. In high-plastic and medium- plastic clays, however, the spread is normally much greater than for relations which are based on the undrained shear strength.

In Sweden, it has traditionally been more common to relate the shear modulus to the undrained shear strength and recently Dyvik and Olsen (1989) have shown that the initial shear modulus varies with the stress history of the soil in almost exactly the same pattern as the undrained shear strength.

Based on results from direct simple shear tests (Ladd and Foott 1981), coupled with values from experience, Larsson (1986) suggested that the elastic modulus E could be evaluated from

- $$E = \frac{215}{I_p} \tau_{fu} \ln F$$

where τ_{fu} is the undrained shear strength
I_p is the plasticity index and
F is the safety factor against undrained failure.

This stress-strain relation is very close to the hyperbolic or modified hyperbolic curves that are normally assumed for this kind of relation. For saturated clays, it can be written as

- $$G \approx \frac{72}{I_P} \tau_{fu} \ln \frac{\tau_{fu}}{\gamma \cdot G}$$

where γ is the shear strain and
G is the shear modulus at shear strain γ

For most clays, this formula is found to give a better correlation with the measured values of G_0, provided that a lower limiting deformation of $\gamma = 1.5 \cdot 10^{-5}$ is introduced:

- $$G_0 = \frac{72}{I_P} \cdot \tau_{fu} \ln \frac{\tau_{fu}}{1.5 \cdot 10^{-5} \cdot G_0}$$

For smaller strains, the shear modulus is assumed constant and equal to G_0.

The best correlation for G_0 in high-plastic and medium-plastic clays is found to be

- $$G_0 = (\frac{208}{I_P} + 250) \cdot \tau_{fu}$$

which yields values that are very similar to those obtained with the previous relation.

The use of empirical relations having the form $G=f(\tau_{fu}/I_P)$, however, requires a very good estimation of the undrained shear strength as well as the plasticity index. The relations also become extremely sensitive to the latter parameter in low-plastic clays.

In low-plastic clays and in varved or otherwise inhomogeneous soils, where it can be difficult to obtain satisfactory and representative values of the undrained shear strength and the plasticity index, it may be better to use the Hardin expression. This also appears to be the case for more organic soils, such as clayey gyttja. As an alternative to Hardin´s expression, the relation

- $$G_0 = 504 \cdot \tau_{fu} / w_L$$

where w_L is the liquid limit

may be used. This relation also covers the whole range of soils from low-plastic silty clays to high-plastic clayey organic soils. The accuracy is normally compatible to Hardin´s expression, except for those cases where a good estimate of the undrained shear strength is missing.

If empirical relations are to be used to estimate the initial shear modulus in clays, <u>it is thus advisable to use</u>:

<u>For high-plastic and medium-plastic clays</u>

$$G_o \approx (208/I_P + 250) \cdot \tau_{fu}$$

or alternatively $$G_o \approx (72/I_P) \cdot \tau_{fu} \cdot \ln \frac{\tau_{fu}}{G_o \cdot 1.5 \cdot 10^{-5}}$$

and <u>for low-plastic clays and clayey gyttjas</u>

$$G_o \approx 625 \cdot OCR^{k'} \cdot (\sigma' p_a)^{0.5} / (0.3 + 0.7 \cdot e^2)$$

or alternatively $$G_o \approx 504 \cdot \tau_{fu} / w_L$$

<u>It should be observed</u> that the relations above should only be used together with undrained shear strengths determined in direct simple shear tests, corrected field vane or fall-cone tests, dilatometer tests evaluated according to Larsson (1989) or some other test that gives directly compatible results.

All empirical relations appear to somewhat overpredict the initial shear modulus in the uppermost soil layers close to the ground surface where effects of wheathering, root threads and micro fissures may occur.

1. INTRODUCTION

1.1 GENERAL

Previously, research and investigation techniques were mainly aimed at investigating large deformations occurring in consolidation or near failure conditions.

During recent decades there has been an increasing need to determine accurately the deformations in soil, even when these deformations are relatively small. There are various reasons for this growing interest in small deformations. Vibrations from traffic and machinery have become a greater problem and, in certain countries, earthquakes constitute a major engineering problem. Modern constructions are often larger and more sensitive to deformations than older designs, and the construction of offshore and near-shore structures subjected to large wave and wind loads have also increased the need for an accurate description of the shear stress-strain relations in the sub-soil for the whole range of deformations.

Very advanced calculation methods using finite elements and finite differences are available today, but the relevance of the results from these calculations depends on how accurately the stress strain relations for the soil are described, (see e.g. Jardine et al 1986 and Burland 1989).

1.2 SHEAR STRESS-STRAIN RELATIONS

The most common method of describing the shear stress-strain relation in a soil is to treat it as having a hyperbolic shape. This means that the shear modulus G $(=\tau/\gamma)$ has a high initial value (G_0), and decreases non-linearly with further stresses and strains until the deformations become very large when the shear strength is approached, FIG. 3.

The shear modulus (G) at any strain (γ) can then be calculated from

- $G = G_0/(1+\gamma/\gamma_r)$

where G_0 = initial (or maximum) shear modulus
and γ_r = reference strain τ_{max}/G_0

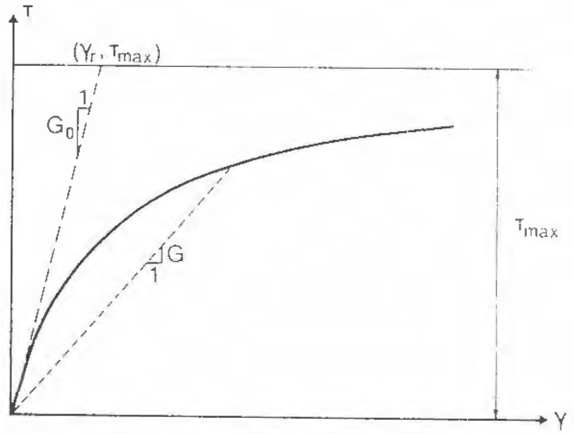


Fig. 3. Hyperbolic stress-strain relation, after Hardin and Drnevich (1972).

For clays, the value of τ_{max} is approximately equal to the undrained shear strength τ_{fu}.

The shear stress-strain relations for most soils, however, are not quite hyperbolic but deviate somewhat from this shape. Hardin and Drnevich (1972) therefore introduced the concept of a hyperbolic shear strain (γ_h) to describe a slightly modified hyperbolic stress-strain relation. The hyperbolic shear strain is written

- $$\gamma_h = \frac{\gamma}{\gamma_r}\,[1+a\cdot e^{-b(\frac{\gamma}{\gamma_r})}]$$

where a and b are constants for the particular type of soil and e is the base of the natural logarithm.

The expression for the shear modulus G is then altered to

- $$G = G_0/(1+\gamma_h)$$

and the decrease in shear modulus with increasing hyperbolic strain can be seen in FIG. 4.

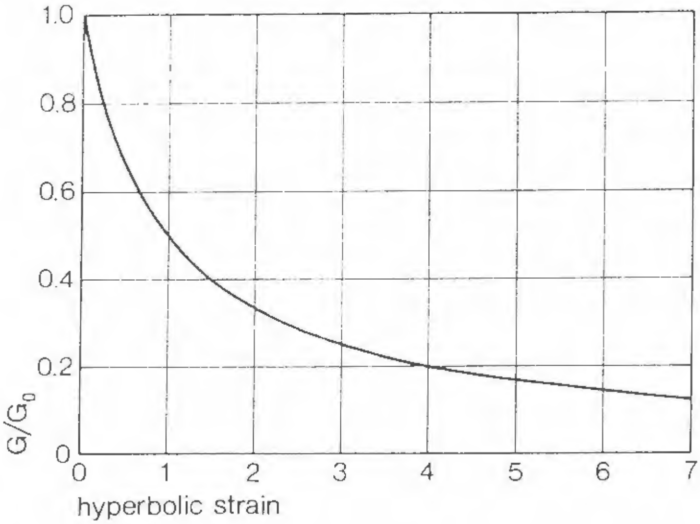


Fig. 4. Normalized shear modulus versus hyperbolic strain, after Hardin and Drnevich (1972).

Hardin and Drnevich (1972) also gave values for the parameters a and b for different soils and types of loading, TABLE 1.

Table 1. Soil parameters a and b for estimation of shear modulus, after Hardin and Drnevich (1972).

Soil type	a	b
Clean dry sand	-0.5	0.16
Clean saturated sands	-0.21 log N	0.16
Saturated cohesive soils	1+0.25 log N	1.3

Where N is the number of load cycles.

The modified hyperbolic stress-strain relations for a first loading are then as shown in FIG. 5.

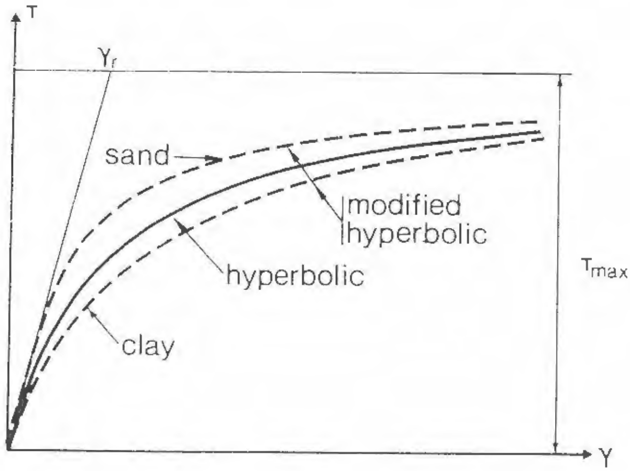


Fig. 5. Hyperbolic stress-strain relation and modified hyperbolic relations for sand and clay, after Hardin and Drnevich (1972).

Massarsch (1981 and 1985) and Dobry and Vucetic (1987) observed that the shape of the stress-strain curve for clays was significantly affected by the plasticity index of the soil.

These stress-strain curves were obtained from laboratory tests with relatively small deformations, mainly in the range of 10^{-5}-10^{-3}. Because of disturbance and time effects, it is very difficult to recreate in situ conditions in the laboratory and the relevance of the stress-strain relations has therefore been discussed. Drnevich and Massarsch (1978) suggest that the initial modulus G_0 should be measured by in situ tests and that the modified-hyperbolic stress-strain relations could be used together with this value. Other approaches to adjust and increase the laboratory determined moduli have been proposed.

The most common approach (according to Andréasson 1979) is simply to increase the laboratory determined moduli by a certain percentage P_r

- $G_{field} = G_{lab} \cdot P_r$

and assume that the disturbance effects are proportionally the same for the entire range of deformations.

Larkin and Taylor (1979) suggest a modification of the curve with emphasis on the low strain range. The disturbance effects would then gradually diminish with increasing strains to disappear completely at a shear deformation of 1 per cent. Andersson and Stokoe (1978), on the other hand, suggest that the laboratory determined moduli should be increased by a constant amount A_r

- $$G_{field} = G_{lab} + A_r$$

where the factor A_r is ascribed to ageing effects in the field.

The last two approaches are contradictory to each other concerning the shape of the stress-strain curve, although both increase the laboratory determined moduli. The simple approach with a percentage increase gives results that lie between them.

Another shear stress-strain relation was suggested by Larsson (1986). On the basis of the results of direct simple shear tests reported by Foott and Ladd (1981), coupled with empirical experience from the field, it was suggested that the undrained modulus of elasticity (E) in clay could be written

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

where τ_{fu} = undrained shear strength
I_P = plasticity index
F = factor of safety against undrained failure (τ_{fu}/τ)

For saturated clays, this formula can also be written as:

- $$G \approx \frac{72}{I_P} \cdot \tau_{fu} \cdot \ln \frac{\tau_{fu}}{G \cdot \gamma}$$

where G is the shear modulus at shear strain γ.

The expression was intended to yield moduli for calculation of fairly large initial deformations e.g. in the construction of road embankments.

Both the laboratory values and the empirical data were obtained at relatively large stresses and strains ($\gamma \approx 5 \cdot 10^{-4} - 5 \cdot 10^{-2}$).

Also this expression yields stress-strain relations that are fairly close to the hyperbolic or modified hyperbolic shapes, FIG. 6, but the expression does not explicitly assume a maximum shear moduli G_0

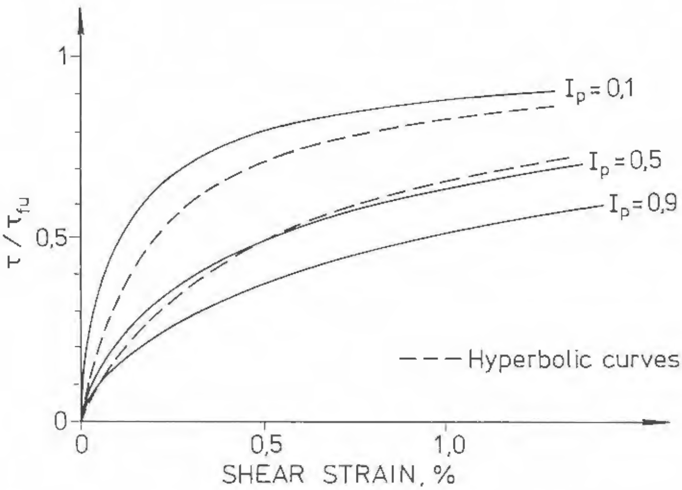


Fig. 6. Stress-strain curves for clays obtained by the relation suggested by Larsson (1986). Curves with hyperbolic shapes are inserted for comparison.

The use of hyperbolic (or modified hyperbolic) stress-strain formulations implies that there should be a range of strains where the soil behaves as almost linear-elastic, with a shear modulus equal to G_0. This assumption appears to be verified by laboratory tests. Hardin and Drnevich (1972) suggested that a practical limit for the elastic range would be strains in the order of $2.5 \cdot 10^{-5}$ and Hardin (1978) later reduced this value to $1 \cdot 10^{-5}$. Dobry and Vucetic (1987) indicate values of the same order but suggest that the elastic range increases with increasing plasticity of the soil. There are, however, field observations indicating that not even this is a lower limit. Thus, for example, Seed and Idriss (1970) presented data indicating a continuous increase in shear modulus for deformations decreasing to below $4 \cdot 10^{-6}$, FIG.7.

Seed and Idriss (1970) also compiled a large amount of data from the literature in a diagram showing the gradual decrease in shear modulus in clay with increasing deformations. This diagram has later been supplemented with data from Westerlund (1978), FIG. 8.

The test results in the diagram mainly follow the trend given by the equation suggested by Larsson (1986) and no upper limit for the shear modulus at very low strains can be observed. The data, however, are all obtained at shear strains greater than 10^{-6} and it is generally assumed that the shear modulus is more or less constant for strains smaller than 10^{-6}, e.g. Det Norske Veritas, 1977.

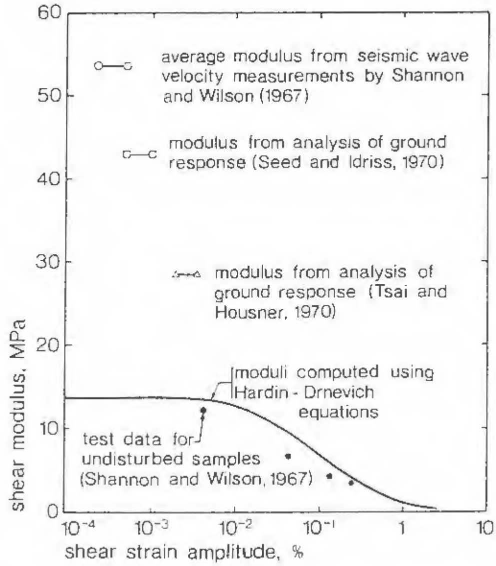


Fig.7. Shear moduli determined at various strain amplitudes in field and laboratory for Union Bay clay at 24 m depth, after Seed and Idriss (1970).

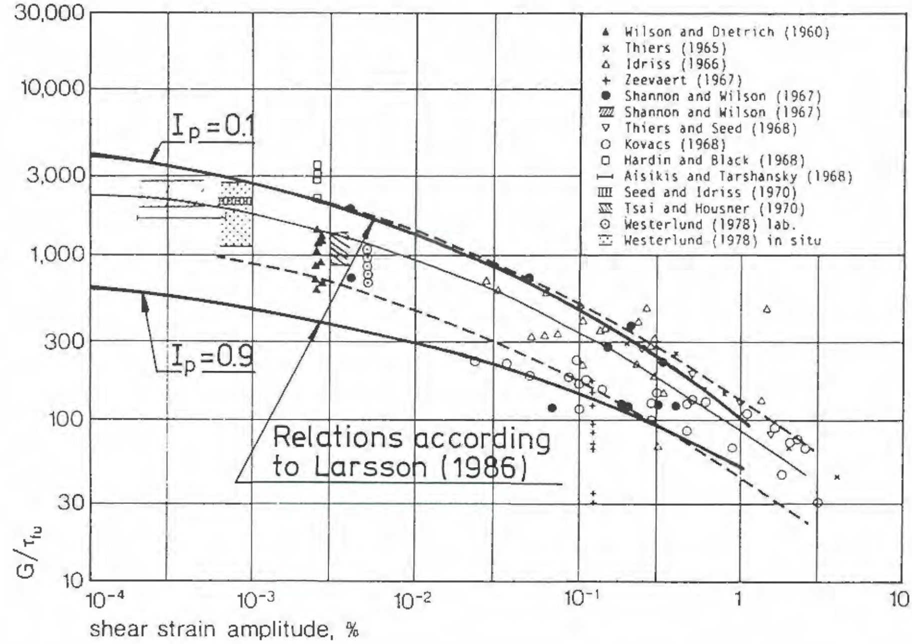


Fig.8. Normalized G/τ_{fu}-values for a variety of clays, after Seed and Idriss (1970), Westerlund (1978) and Andréasson (1979). The relation suggested by Larsson (1986) is also shown.

1.3 EMPIRICAL RELATIONS FOR THE INITIAL SHEAR MODULUS G_0

The comprehensive laboratory investigations have yielded a number of empirical relations for the maximum shear modulus (Hardin and Black 1968 and 1969, Anderson 1974 and Hardin 1978). The test results indicate that the shear modulus is a function of void ratio (e), the mean effective stress (p') and the overconsolidation ratio OCR. For clays, Hardin (1978) gave the relation

- $$G_0 = 625 \cdot OCR^{k'} \cdot (p' \cdot p_a)^{0.5} / (0.3 + 0.7\, e^2)$$

where p' = mean effective stress = $\sigma'_v \cdot (1+2\,K_0)/3)$ kPa
(σ'_v = effective vertical stress and
K_0 = coefficient of earth pressure at rest)
p_a = atmospheric pressure = 98.1 kPa
e = void ratio
OCR = overconsolidation ratio
k´ = a parameter dependent on the plasticity index of the soil, FIG. 9.

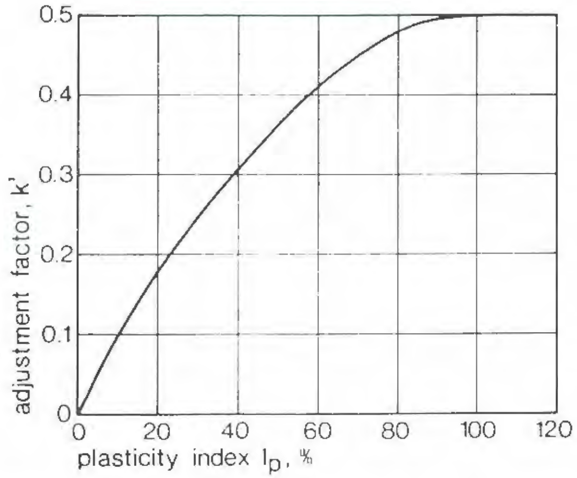


Fig. 9. Overconsolidation adjustment factor k´ versus plasticity index I_p, after Hardin (1978).

In this formula, the shear modulus is related to the square of the void ratio, the square root of the effective stress and to some extent to the overconsolidation ratio. However, these three parameters are not independent of each other. For a natural clay, the void ratio is mainly a function of the effective consolidation stress and to some extent of time and of possible stress relief causing overconsolidation effects. This function varies with the composition of the soil, which in turn determines its structure (or void content at various conditions) which can be expressed by the consistency limits of the soil. A schematic variation of the void ratio with stress history is shown in FIG. 10.

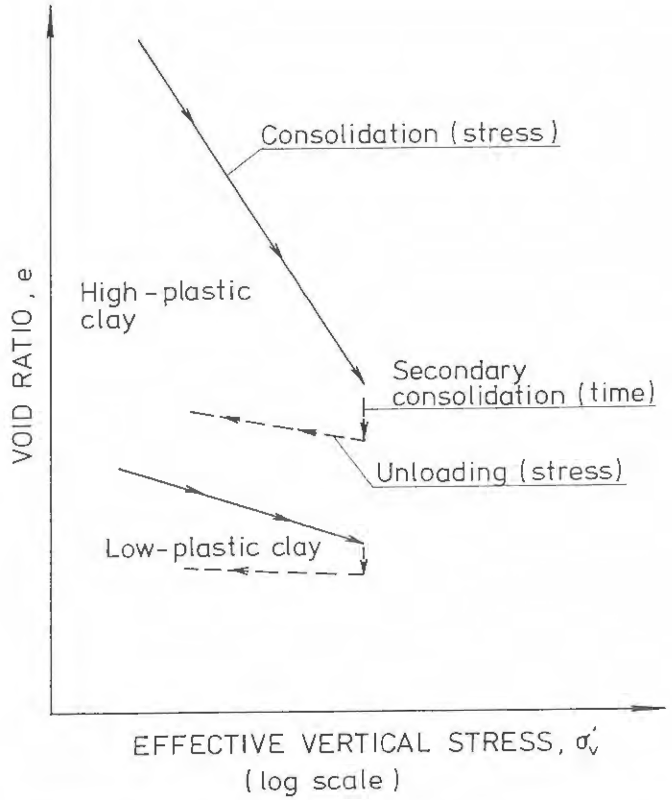


Fig. 10. Schematic variation of void ratio with stress and time for different types of clay.

It might thus be expected that another empirical relation could be obtained in which G_0 is a function of the effective consolidation pressure, the plasticity index of the soil and, to some degree, the overconsolidation ratio.

In Sweden, as in many other parts of the world, the elastic properties of clays have often been related to the undrained shear strength.

Locally, simple thumb rules such as $E = 250\ \tau_{fu}$ (or $G \approx 80\ \tau_{fu}$) have been used to calculate relatively large strains. For the maximum shear modulus, a factor of 10 is usually applied, which in the corresponding case would give $G_0 \approx 800\ \tau_{fu}$. In a study on two high-plastic clays in the Gothenburg area, Andréasson (1979) found that the initial shear modulus varied between 200 and 800 τ_{fu}, with an average of 441 τ_{fu} and a standard deviation of 135 τ_{fu}.

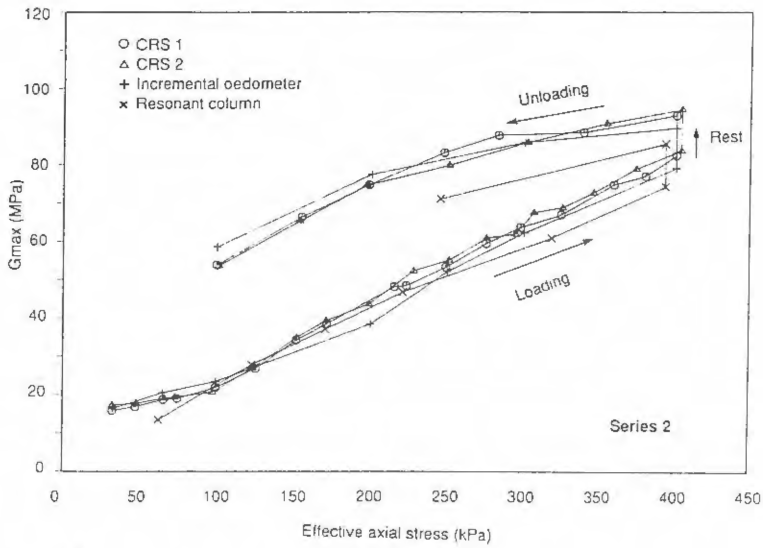
Such relations, however, can only be used locally and then with great caution. Experience has shown that the elastic properties vary strongly with the plasticity index of the soil as shown by the empirical relation

- $$E = \frac{215}{I_P} \cdot \tau_{fu} \ln F$$ (Larsson 1986)

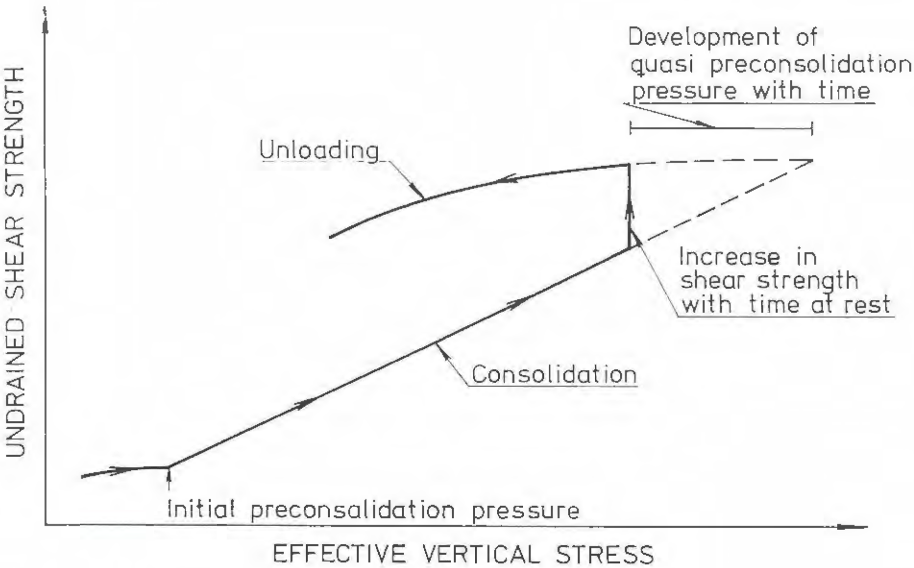
This relation shows that, especially for low-plastic clays, there is a very large influence of the plasticity index on the modulus of elasticity and hence on the shear modulus.

For a particular clay, however, there is a direct relation between the undrained shear strength and the shear modulus. Recent investigations by Dyvik and Olsen (1989) have shown that the maximum shear modulus varies with stress history in almost exactly the same pattern as the undrained shear strength, FIG. 11.

The general pattern of shear strength increase with consolidation and time was outlined by Bjerrum (1967 and 1972) and the reduction in undrained shear strength at a subsequent decrease in vertical stress was shown by Ladd et al (1977). This pattern has been further elaborated to show the influence of the plasticity index and mode of shear on the relation between preconsolidation pressure and undrained shear strength (Larsson 1980, Jamiolkowski et al 1985). Additional data regarding the decrease in undrained shear strength at unloading have been gathered (Jamiolkowski et al 1985) and the increase in undrained shear strength with time has been further analysed and verified (e.g. Larsson 1986, Mesri 1987).



a)



b)

Fig. 11. a) Variation of initial shear modulus with stress and time (Dyvik and Olsen 1989)

b) Variation of undrained shear strength with stress and time.

The fact that the initial shear modulus varies with stress history in almost exactly the same pattern as the undrained shear strength implies that it should be possible to express this modulus as a function of undrained shear strength and plasticity index only. It should be observed, however, that the undrained shear strength has to be defined because this parameter varies with mode of shear and thus with the testing technique.

This empirical initial shear modulus can then be converted to the shear modulus at an arbitrary deformation by using an empirical relation for the general shape of the stress-strain curve.

1.4 DYNAMIC VERSUS STATIC SHEAR MODULUS

Traditionally, the shear modulus has been divided into two categories: dynamic moduli for small strain problems (mainly vibrations) and static moduli for calculation of deformations at construction loads where the safety factor against undrained failure is typically in the order of 1.3-3.0. The ratio between these moduli has often been assumed to be in the order of 10 to 1.

Later research has shown that there is no fundamental difference between the moduli, but they mainly constitute the extreme ends of the same curve where the modulus continuously decreases with increasing deformations (e.g. Andréasson 1979).

Differences mainly occur at large shear stresses where repeated dynamic or static load cycles bring an increase in deformations because of accumulated non-recovering plastic strains and sustained static loads bring further deformations because of consolidation and/or undrained creep deformations.

The latter type of deformation is not treated in this report.

2. *METHODS FOR MEASUREMENT OF INITIAL SHEAR MODULUS*

2.1 GENERAL

Measurements of initial shear moduli are performed either in situ or on "undisturbed" samples reconsolidated to in situ stresses in the laboratory. In situ tests are mainly favoured, partly because of the difficulty of obtaining truly undisturbed samples and of recreating the properties in the laboratory, and partly because new and cost effective in situ methods have been developed.

The in situ measurements are in reality carried out as measurements of the shear wave velocity (V_s) in the soil, which is then converted to a shear modulus using the theory of elasticity

- $G = V_s^2 \cdot \rho$, kPa

where ρ is the density of the soil in t/m^3 and V_s is expressed in m/s.

The measurements should be made at small shear strain amplitudes to ensure that it is a fairly constant "initial" modulus that is measured. A general aim should be to perform the measurements at shear strains in the order of 10^{-6} or lower.

2.2 CROSS-HOLE TESTS

The cross-hole technique entails that two holes are drilled vertically into the ground at some distance (a few metres) from each other. The verticality of the holes must be carefully checked as the distance between them at all test levels must be known. At the same level in the holes, an impulse source and a receiver (geophone) respectively are installed . These are brought into close contact with the soil and are connected to a registration unit, e.g. a memory oscilloscope.

The oscilloscope is triggered instantaneously when the shear wave is created by the impulse source and the time for the arrival of the shear wave to the other hole is measured by the signal from the geophone and the clock in the oscilloscope. FIG. 12.

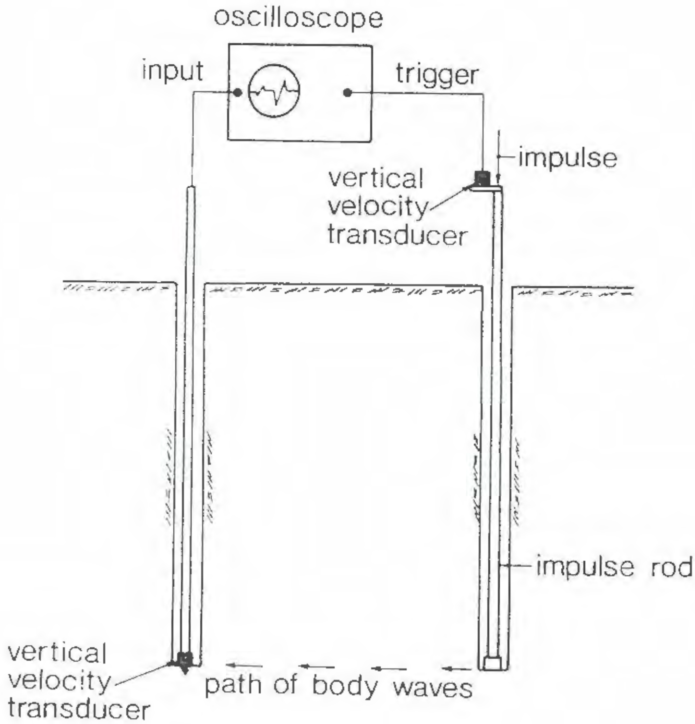


Fig. 12. Principles of seismic cross-hole survey technique (Andréasson 1979).

This technique is often considered the most accurate method and is therefore often used as a reference test. Nevertheless, it has some weaknesses. When prebored holes are used, there is always some disturbance due to stress relief in the soil adjacent to the holes. Good contact between the impulse source and geophone and the soil may also involve problems. Comparisons by Andréasson (1979) indicate that the disturbance effects are approximately equal when the holes are predrilled and when the instruments are simply pushed into the ground.

Another shortcoming is that the variation in shear modulus with depth becomes subdued because the shear waves travel faster in stiffer layers and the travel path is not necessarily horizontal. Abrupt changes in soil properties with depth therefore become smoothed and weaker layers with limited thicknesses are not fully detected, FIG. 13.

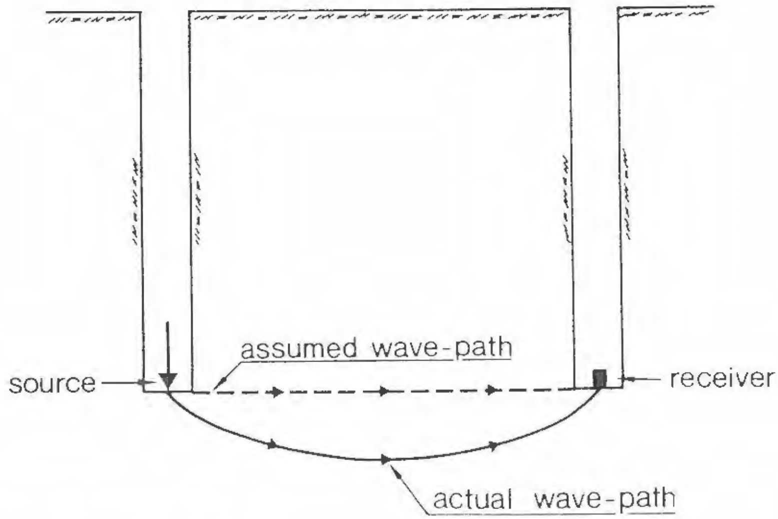


Fig. 13. Curved wave-path encountered in a soil deposit where stiffness increases with depth (Andréasson 1979).

2.3 DOWN-HOLE TESTS

The down-hole test requires only one bore-hole into which a horizontally oriented velocity transducer is lowered. The horizontal shear impulse is then created at the ground surface and the time for the vertical shear wave propagation down to the level for the receiver is measured by the memory oscilloscope, FIG. 14. (The test can be reversed with the impulse source in the hole and the receiver on the ground and is then termed an up-hole test.)

Similar disturbances around the bore-hole occur as in cross-hole testing. Direct comparisons by Andréasson (1979) showed that no difference was observed between tests in predrilled holes and tests where the receiver had simply been pushed into the ground without predrilling. The latter method was much more time and labour effective (compare seismic sounding). The down-hole method is potentially able to pick up variations in the soil stratigraphy, but very high demands must be put on the equipment. In the test, it is the difference in arrival time from one level to another that is measured, which puts high demands on exact depth recording, verticality, triggering mechanism and resolution in time measurements, especially if only one receiver which is moved from level to level is used.

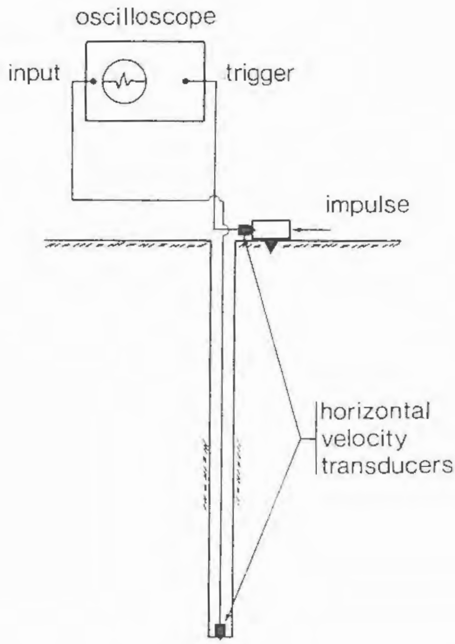


Fig. 14. Principles of seismic down-hole survey technique (Andréasson 1979).

Another problem is that the distance from the impulse source to the receiver increases and the signals gradually become weaker with depth. Different impulse sources may therefore have to be employed at various depth levels.

Further details on the down-hole and cross-hole techniques in clay can be found in e.g. Andréasson (1979).

2.4 SEISMIC CONES

The seismic cone has been developed since 1980, mainly at the University of British Columbia in Vancouver, Canada (Campanella et al 1986). The cone is an ordinary ø 35.7 mm CPT or CPTU cone into which a miniature velocity seismometer has been incorporated, FIG. 15. In this way, seismic down-hole tests can be carried out at regular depth intervals during and as part of an ordinary soil investigation with cone penetration testing. This procedure reduces the need for extra equipment to a minimum and provides a rapid and economic method of obtaining information on the initial shear modulus and its variation with depth.

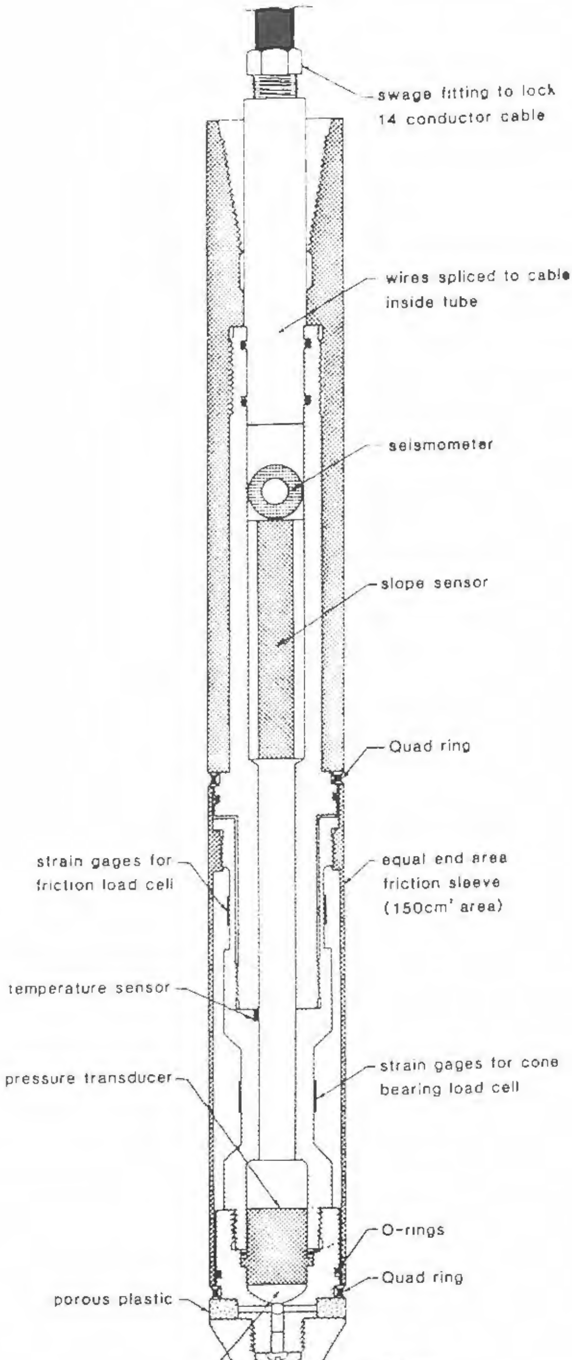


Fig. 15. The UBC seismic cone (Campanella et al 1986).

The extra equipment needed, apart from the built-in seismometer, is a memory oscilloscope and an impulse source with a trigger for the oscilloscope. The latter usually consists of a steel beam pressed against the ground by the weight of the drill rig and a sledge-hammer with a trigger. The horizontal shear wave is created by hitting the beam ends horizontally and axially to the beam with the sledge-hammer, FIG. 16. The equipment is now commercially available and the memory oscilloscope has been incorporated into the field computer, which collects and processes all the data from the ordinary cone penetration test.

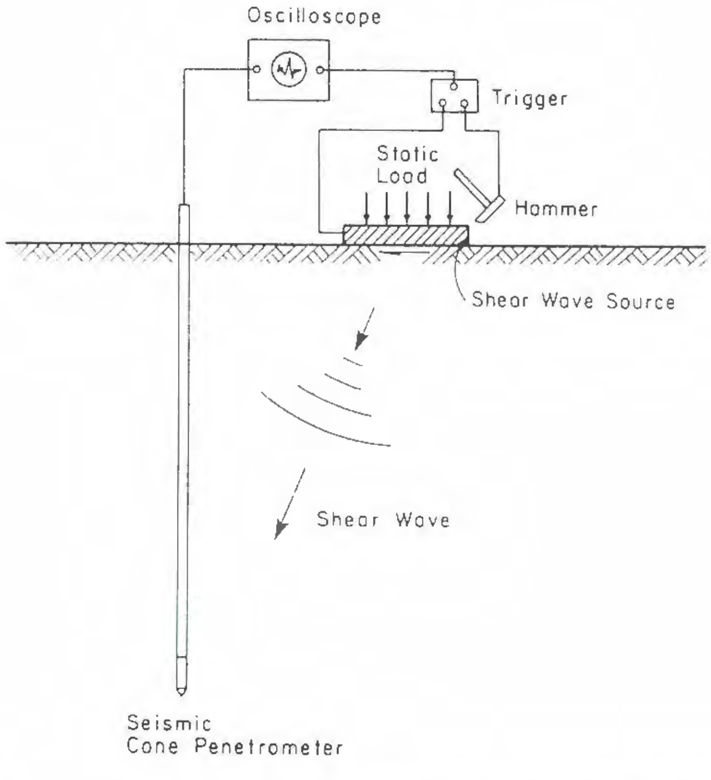


Fig. 16. Principles of the seismic cone survey technique (Campanella et al 1986).

Requirements on the cone equipment are that

- the signals from the cone to the field computer are transferred by cable
- the depth recording must be very accurate
- the cone should incorporate an inclinometer for checking the verticality of the sounding
- when the seismometer in the cone is of a one-directional type, provision should be made so that the measuring direction can be kept parallel to the beam on the ground.

As in ordinary down-hole tests, the requirements on the electronic ground equipment are that the trigger is very fast and that the oscilloscope has a high resolution on the time scale. The seismometer and the oscilloscope should also be calibrated so that the oscillation velocity of the cone at the measuring level can be evaluated.

The test is normally performed in such a way that the cone is first oriented with the axis of the seismometer parallel to the beam axis and then pushed down into the soil at the normal rate of penetration, 20 mm/s. During this penetration, readings of tip resistance, friction, pore pressure, inclination and possibly other parameters are taken as in normal penetration testing. At regular intervals, usually 1 metre, the penetration is momentarily stopped and the oscilloscope is switched in. The beam is hit with a single blow on one end and the signal from the seismometer is recorded and stored in a memory. This procedure is then repeated with a blow on the other end of the beam. Thereafter, the normal penetration recording program is resumed and the cone is pushed down to the next seismic test level.

The seismic signals become continuously weaker with depth, FIG. 17, but testing down to at least 40 m has been performed with sufficient signals being obtained.

The results are then evaluated in terms of difference in arrival time of the shear wave from level to level, FIG. 18. It is often difficult to determine the exact time of the shear wave arrival and different procedures may be employed. The most common technique is to plot the results from the two blows in different directions at each level on a common graph and to use the first cross-over points for comparison. Other significant points, e.g. the first or second peak, may also be used.

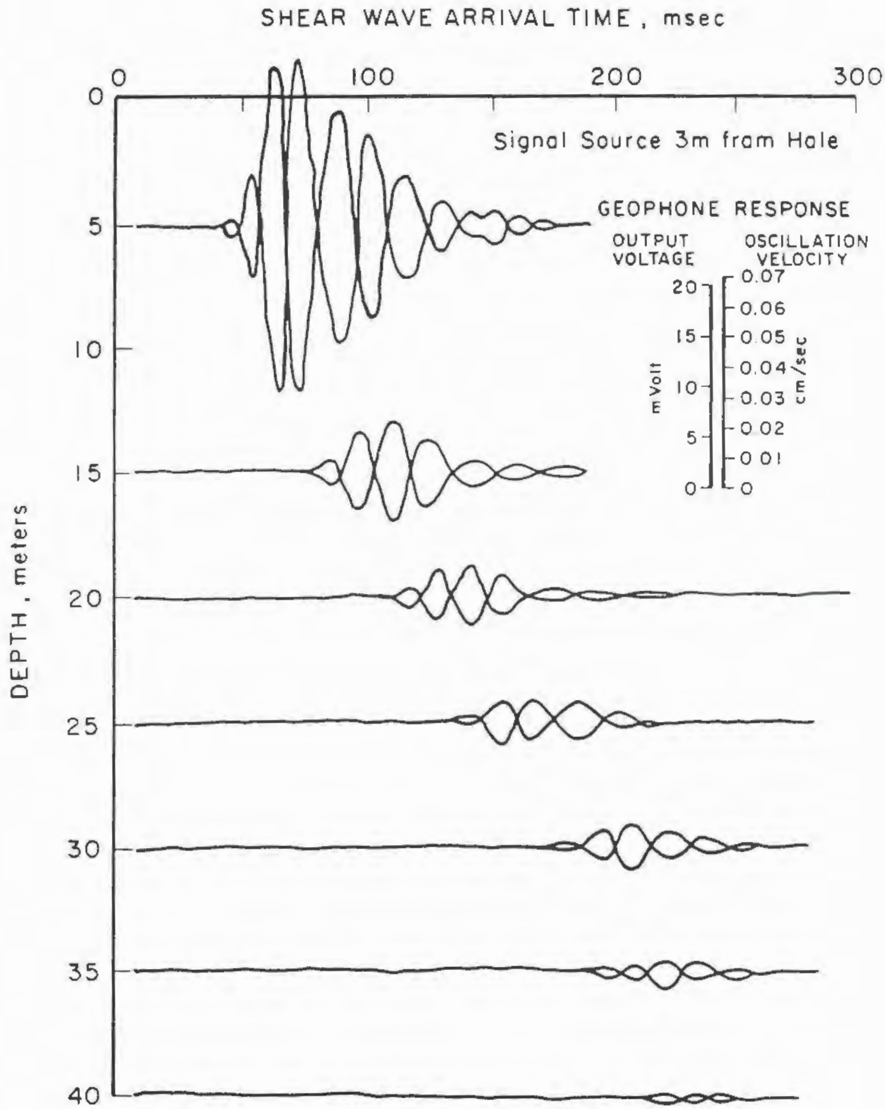


Fig. 17. Comparison of shear wave signals obtained at various depths in a soil profile (Campanella et al 1986).

However, these evaluation techniques can only be used when the two compared sets of curves are obtained in fairly similar types of soils. At intersections between soil layers of widely different character, e.g. when passing from a stiff dry crust into very soft organic clay or from soft clay into sand layers, the first arrival times of the shear waves have to be compared. In those cases, other points on the curves will give significant time differences that become more deviating the further away they are located from the point of first arrival of the shear wave. The times of first arrival also have to be used when a significant correction for non-vertical travel paths is applied. Otherwise the correction becomes erroneous.

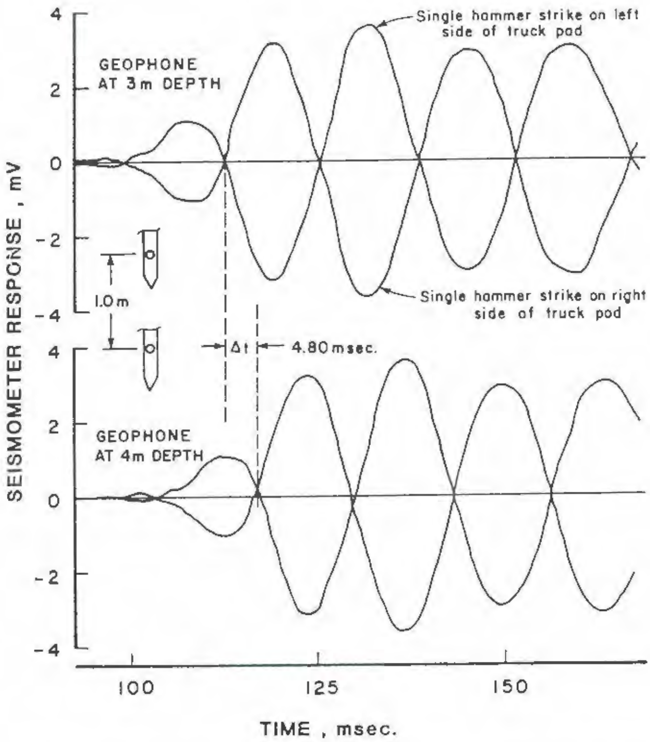


Fig. 18. Principle for evaluation of travel times for shear waves in the seismic cone test (Campanella et al 1989).

The curves are sometimes affected by noise (other vibrations and possible electronic instability). Special filtering techniques have therefore been developed (Campanella et al 1989). In clays, this is usually not required. Some disturbing noise may occur due to vibrations from the drill rig travelling through both the soil and the rods and from vibrations from the hammer blows travelling through the drill rig and the rods. This problem is confined to the upper soil layers (normally above 3-4 m depth) and can be largely eliminated by shutting off the rig engine during the upper seismic tests.

For practical reasons, the beam on the ground normally has to be placed some distance away from the point of penetration. A distance of up to 3 metres is considered acceptable (Campanella et al 1986) and even longer distances have been suggested for ordinary down hole tests (Hoar and Stokoe 1978). However, this entails that the travelling paths of the shear waves are not truly vertical and a correction has to be made. Assuming a linear travel path between the beam and the geophone and that there is no wave refraction at different soil boundaries the arrival times can be corrected to correspond to a vertical travel path, FIG. 19.

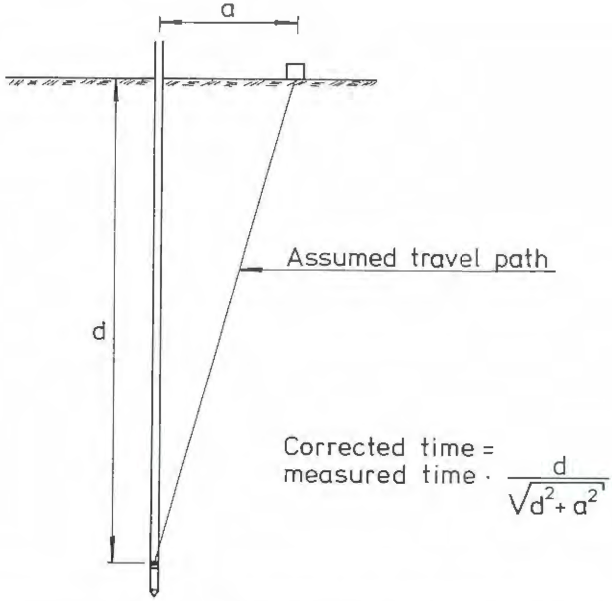


Fig. 19. Correction of measured travel time for non-vertical travel path, after Eidsmoen et al (1985).

When the distance between the beam and the point of penetration is kept small, this correction is not significant, except for the very first metres of depth in the profiles.

The shear wave velocity V_s in a soil layer is calculated as

$$V_s = \frac{\Delta d}{\Delta t}$$

where Δd = the vertical distance between the test levels at the top and bottom of the layer

Δt = difference in corrected arrival times at the top and bottom of the layer.

Assuming that the horizontal velocity measured by the seismometer in the cone corresponds to the particle velocity ($V_{particle}$) in the soil,

the shear strain in the test can be calculated according to

$$\gamma = \frac{V_{particle}}{V_s}$$

where $V_{particle}$ is the peak oscillation velocity.

The tests should preferably be performed at shear strains of 10^{-6} or lower, which can be checked in this way. In very soft clays, this aim can be difficult to achieve, even with restricted blows, and the results may have to be corrected for the strain level.

The seismic cone has been described in further detail by Campanella et al (1986).

2.5 CORRELATION BETWEEN RESULTS FROM SEISMIC CONE TESTS AND CROSSHOLE AND DOWN-HOLE TESTS

The three types of test are specially designed to measure the initial shear modulus (shear modulus at very small strain), even if cross-hole tests have been used to measure the shear wave velocity also at larger strains. A number of investigations in clays (and also silt and sand) have shown that the three methods give compatible results, e.g. Andréasson (1979), Aas et al (1984), Eidsmoen et al (1985) and Campanella et al (1986). The results obtained in the present investigation also indicate that the results from the three types of test are compatible. The disturbance effects due to pushing the seismic cone into the soil are limited because of the small diameter of the cone. As explained by Andréasson (1979), it is uncertain whether this disturbance has any significant effect on the measured shear velocity and, if it does, this effect appears to be of the same order as corresponding disturbance effects in hole drilling for the other two types of test.

2.6 OTHER FIELD TESTS

Shear stress-strain properties of soil can also be evaluated from a large variety of other field tests and observations. Large-strain properties can be observed at various test loadings or full scale constructions. Small-strain properties can be evaluated from very carefully conducted static or dynamic loading tests and by observation of the soil response to vibrations. A special type of test with a dynamically loaded screw plate has been tried out in Sweden (Andréasson 1979 and Bodare 1983). The purpose of this test is to obtain the shear modulus in the field over a wide range of deformations.

These other tests, however, cannot be directly compared to the tests which are used specifically to measure the variation of the initial shear modulus with depth in a soil profile as part of an ordinary soil investigation. They partly measure different parameters, are generally much more expensive and, if employed at all, are usually performed at a later stage of the design process.

2.7 LABORATORY TESTS

The main technique for measuring shear strain-stress properties at small strains in the laboratory has been to use the resonant column method. The usual procedure in this test is that a cylindrical column of soil is encased in a rubber membrane, placed in a triaxial cell, consolidated for a desired stress condition and then subjected to shear stresses in the mode of torsional vibration. The frequency of the vibrations is regulated until resonance is achieved and the shear wave velocity of the soil in the particular stress condition and shear deformation can then be evaluated. Detailed descriptions of the resonant column test can be found in e.g. Richart et al (1970).

This method of measuring shear moduli can mainly be used in the strain range 10^{-5} - 10^{-3}. For measurements in larger strain ranges, ordinary direct simple shear tests or triaxial tests may be used. Results from resonant column tests form the basis for most empirical relations concerning the shape of the shear stress-strain curve and for many empirical relations concerning the initial shear modulus.

Correlations with field measurements have shown that the initial shear moduli measured in the laboratory are generally lower than those measured in the field. This is attributed to disturbance effects at sampling. These disturbance effects cannot be eliminated by reconsolidation to in situ stresses only. The soil in the field has developed a further stiffness due to time effects during its geological history and various approaches have been suggested for correcting the laboratory data with respect to the age of the deposit. The suggested approaches are partly contradictory (see Section 1.2), but the method most often used appears to be to multiply the measured moduli at all strain levels with the same factor, which is estimated from the geological history of the soil deposit. The time effects appear to be most pronounced for high-plastic soils (Kokusho et al 1982).

A new method of measuring the initial shear modulus in the laboratory by using so-called "bender elements" has recently been developed at the Norwegian Geotechnical Institute (Dyvik and Madshus 1985, Dyvik and Olsen 1989). The bender element is a very small rectangular plate which can be made to bend by an electrical excitation signal. If, on the other hand, the plate is bent mechanically, it produces a corresponding electrical signal.

The very small elements can be built into the base and top caps in triaxial cells or direct simple shear apparatuses and into the base and piston in oedometers. They are then mounted in such a way that the edges of the vertically positioned plates protrude one or two millimetres into each end of the soil specimens. The maximum shear modulus can be measured at any stage of consolidation in the various apparatuses. This is done by applying an electrical signal to one of the elements, which then sends a shear wave through the soil specimen, and measuring the time for the arrival of the wave at the other end of the specimen using the electrical signal produced by the second element. For measurement of G_0 only, this is a much simpler and faster method than the resonant column test and appears to give almost identical results.

3. *FIELD INVESTIGATIONS IN SCANDINAVIAN CLAYS*

3.1 PREVIOUS INVESTIGATIONS

At the end of the 70s, two parallel investigations comprising field and laboratory tests in clays were performed at the Norwegian Institute of Technology in Trondheim and Chalmers University of Technology in Gothenburg, (Westerlund 1978 and Andréasson 1979). In these investigations, resonant column tests were used in the laboratories and cross-hole and down-hole tests were used to measure the initial shear modulus in the field. The Norwegian investigations were made on three low-plastic clays in the Trondheim area and the Swedish investigations were made at three locations with high-plastic clays in the Gothenburg area.

In 1984, the seismic cone developed at the University of British Columbia was tried out in cooperation with the Norwegian Geotechnical Institute at three test sites in the Oslo area; one site with sand and two with clay profiles (Eidsmoen et al 1985). Cross-hole tests had previously been performed at two of these sites.

All these investigations are well documented and the data have been used together with the results in the present investigation to form a broad basis for the empirical correlations.

The test sites in Gothenburg in Andréasson´s (1979) investigation have also been used in the present investigation.

3.2 SCOPE OF THE PRESENT INVESTIGATION AND EQUIPMENT USED

The present investigation with seismic tests is part of a larger project concerning the use of new in situ methods in soft to medium stiff soils.

The main purpose of the seismic tests was to

- test the general usefulness of the seismic cone equipment
- obtain an estimate of the repeatability of the test results and their correlation to other test results
- if possible, to obtain a database to check existing empirical relations and, if need be, improve them with reference to Swedish clays.

The tests have been performed during the period 1989-1990.

Prior to the field tests, a careful calibration of the seismometer and the field oscilloscope was performed in the laboratory (Mulabdić et al 1990).

In the larger project, cone penetration tests have so far been performed with different equipments and cone types in 7 well documented test sites in clay areas and 2 areas with clayey organic soils (clayey gyttja). Among the equipments and cones used is the seismic cone equipment manufactured by Hogentogler & Co. The design of this equipment is based on the experience gained at the University of British Columbia, and the field oscilloscope is incorporated in the field computer used for collecting, storing and processing the usual cone penetration data.

The seismic cone is supplied with an internal inclinometer and a special high-resolution depth recorder has been used. Depth recording has also been checked manually during the penetrations.

At each site, two parallel soundings have been performed with the seismic cone. Seismic tests have been performed at each metre of depth starting 1 or more often 2 m below the ground surface and then continuing to firm bottom. In some very deep profiles, the soundings have been terminated earlier, mainly because of a lack of reference data.

The cone has been pushed into the ground with a drill rig mounted at the rear end of a Unimog truck. When drilling, the truck is lifted hydraulically so that it rests on two legs at the front and the drill rig at the rear end. The drill rig can then be adjusted into a vertical position by using the hydraulics.

In the seismic tests, a steel beam was placed on the ground 0.65-1.4 m behind the point of penetration and perpendicular to the direction of the truck. The steel beam was pressed against the ground by an extension of the drill rig. Following a suggestion by Eidsmoen et al (1985), the beam was supplied with two steel skirts protruding about 0.1 m into the ground in order to prevent sliding of the beam when struck with the sledge-hammer.

Power to the measuring equipment has been provided by a stabilized portable generator and the computer has been installed in either a small van or a caravan.

3.3 TEST SITES AND RESULTS

3.3.1 General

The test sites in the study have been selected among the well documented test fields previously used by the Institute and by Chalmers University of Technology. Also the extensively investigated landslide area of Tuve and the potential landslide area at Munkedal have been used. The various test areas are briefly described in the following section together with the Norwegian test areas that have been investigated by our Norwegian colleagues.

The seismic test results have been very uniform and the results are mainly presented as averages of the two soundings (three at Onsöy). The shear strengths referred to in empirical correlations are shear strengths evaluated from direct simple shear tests and corrected field vane and fall-cone tests supplemented by dilatometer tests. These types of undrained shear strengths are normally determined and used in Sweden and are directly compatible (Larsson et al 1984, Larsson 1989). In order to avoid repetition, the results are presented together with corrected curves and empirical correlations which are explained in detail in Chapter 4.

3.3.2 SGI sites

• **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 kilometres 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, 1986 and 1990, 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 of determining these properties have been introduced (Larsson 1986 and 1990). The soil profile is shown in FIG. 20.

At the top, there is a layer of organic top soil. The dry crust is unusually thin and consists of organic soil. The dessicated 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, at depths between 2.5 and 6.5 metres, amount to 0.5% of the dry weight of the soil.

The natural water content is approximately 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 of depth has been studied in triaxial tests and direct simple shear tests in a special investigation (Larsson 1977).

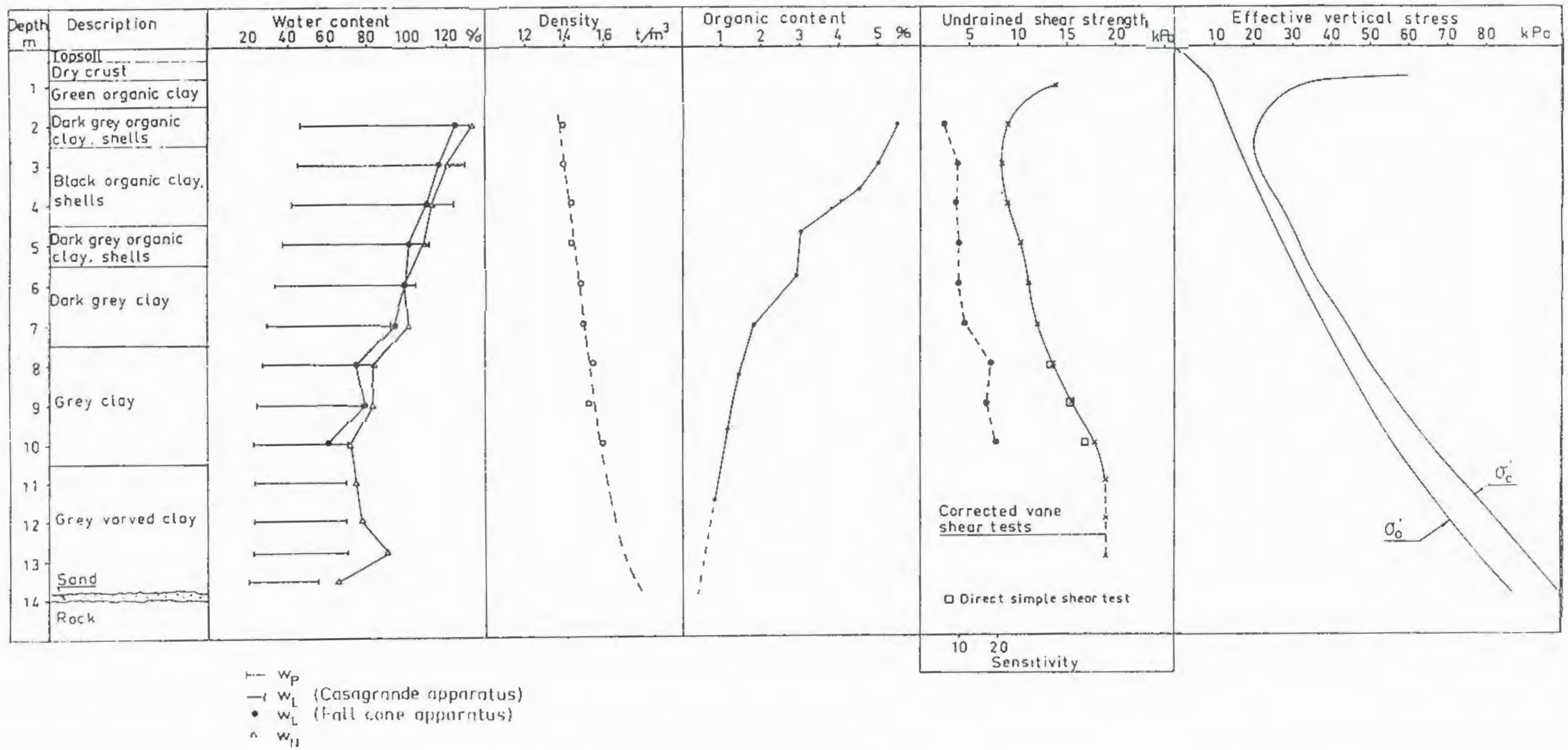


Fig. 20. Soil profile at Lilla Mellösa.

The pore pressure in the ground has been found to be close to the hydrostatic pressure for a ground water level 0.8 m below the ground surface.

Overconsolidation because of the various effects in the dry crust and directly below it occurs down to about 2.5 m depth. Below this depth, the overconsolidation ratio is almost constant 1.2. The coefficient of earth pressure has been estimated from laboratory tests to be around 0.75 in the depth range 8-10 metres.

Dilatometer tests supplementing the previous investigations were performed in November 1988. The results have been reported in detail by Larsson and Eskilson (1989).

The seismic tests were performed in December 1989 during a cold spell when temperatures fell to about -30° C, FIG. 21. The testing programme was nevertheless performed without any problems with the sounding equipment.

Fig. 21. The test site at Lilla Mellösa during the cold spell in December 1989. Cars and trucks broke down, hydraulics jammed and the working conditions for the field crew were very tough. The seismic tests could nevertheless be performed without any problems with this part of the equipment.

The results were very uniform. The measured shear modulus increases continuously with depth and forms a very smooth curve, which could be expected in this uniform profile where all other properties also change smoothly and continuously with depth, FIG. 22.

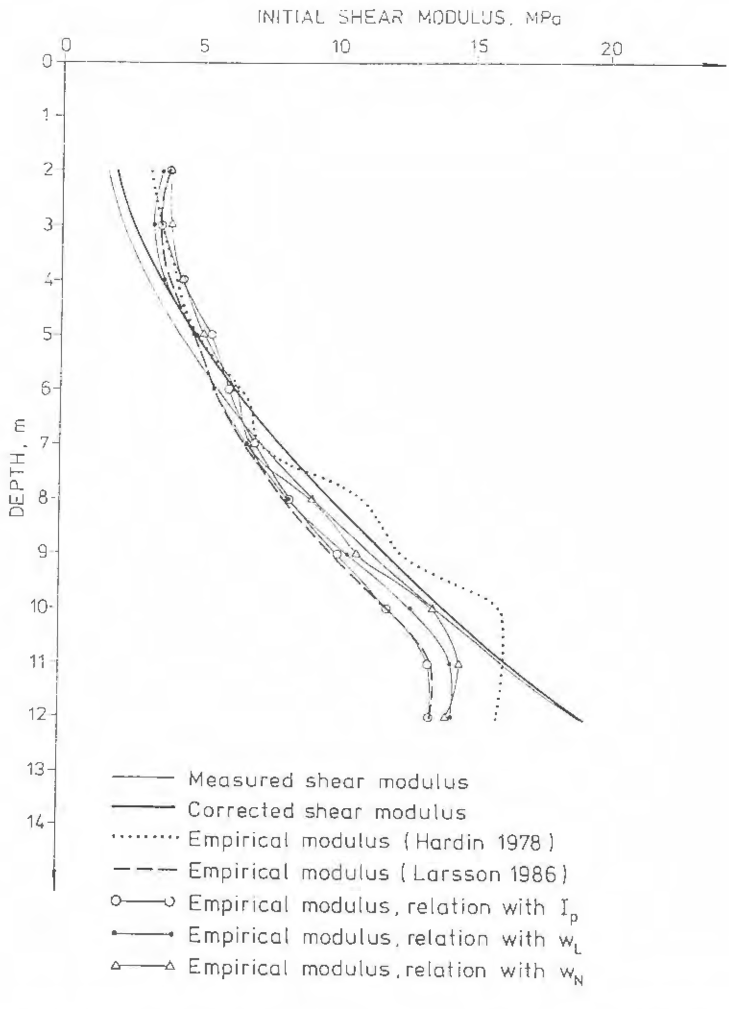


Fig. 22. Results from the seismic tests at Lilla Mellösa.

- Skå-Edeby

Skå-Edeby is the second test field used by SGI for observation of long-term settlements. The field is located on Svartsjölandet, which is an island in Lake Mälaren about 25 kilometres west of Stockholm. The field has been used since 1957, when the first test fills were constructed (Hansbo 1960). A large number of investigations have later been made in the test field at Skå-Edeby, (Kallstenius and Bergau 1961, Kallstenius 1963, Osterman and Lindskog 1963, Pusch 1970, Holtz and Lindskog 1972, Holtz and Broms 1972, Holtz and Holm 1972, Boman and Broms 1975, Wiesel 1975, Massarsch et al 1975, Torstensson 1976, Holm and Holtz 1977, Larsson 1986 and Larsson and Eskilson 1989).

The soil in the test field consists of 10-15 metres of soft clay on rock or till. The area is very flat, but small hills and outcrops of rock can be seen some distance away. The conditions in different parts of the field are not quite homogeneous as the thickness of the clay layers varies somewhat. A detailed description of the test field has been given by Hansbo (1960) and a soil profile which is relevant for the part of the field where the seismic tests were performed is shown in FIG. 23.

The dessicated dry crust is only about half a metre thick. It is followed by a layer of grey-green organic clay, which is slightly overconsolidated due to crust effects . The post-glacial clay beneath reaches to 6 m depth and is high-plastic and slightly organic. This layer, as well as the lower lying glacial clay, is coloured or banded by ferrous sulphide.

The glacial clay is varved, which becomes more pronounced with depth and at the bottom there are also infusions of silt and sand. The clay content between 2.5 and 10 m depth is approximately 70%.

The water content decreases from 100% in the upper clay layers to 60% in the bottom layers and is, except for the uppermost 2 metres, somewhat higher than the liquid limit. The density of the soil increases correspondingly from 1.3 t/m^3 to 1.7 t/m^3.

The ground water level has been found to vary seasonally from the ground surface to 1 metre below. In November 1989, when the seismic tests were performed, the free ground water level was 0.8 m below the ground surface and the pore pressure in the coarser bottom layers corresponded to a hydrostatic pressure from this level.

Corresponding calculated effective vertical stresses for a hydrostatic pressure in the soil and measured preconsolidation pressures yield overconsolidation ratios between 1.1 and 1.2 in the depth range 4 to 10 m, increasing to about 1.3 at a depth of 12 m. Above the 4 m depth level, the overconsolidation ratio increases because of crust effects.

The undrained shear strength has been measured nearby in a comprehensive field vane testing program (Wiesel 1975). The sensitivity is in the order of 15.

The coefficient of earth pressure has been measured by earth pressure cells and hydraulic fracturing (Massarsch et al 1975). The results from earth pressure cells have been found to be most relevant in Swedish soils.

Supplementary dilatometer tests were performed under identical ground water conditions in October 1988. The results have been reported in detail by Larsson and Eskilson (1989).

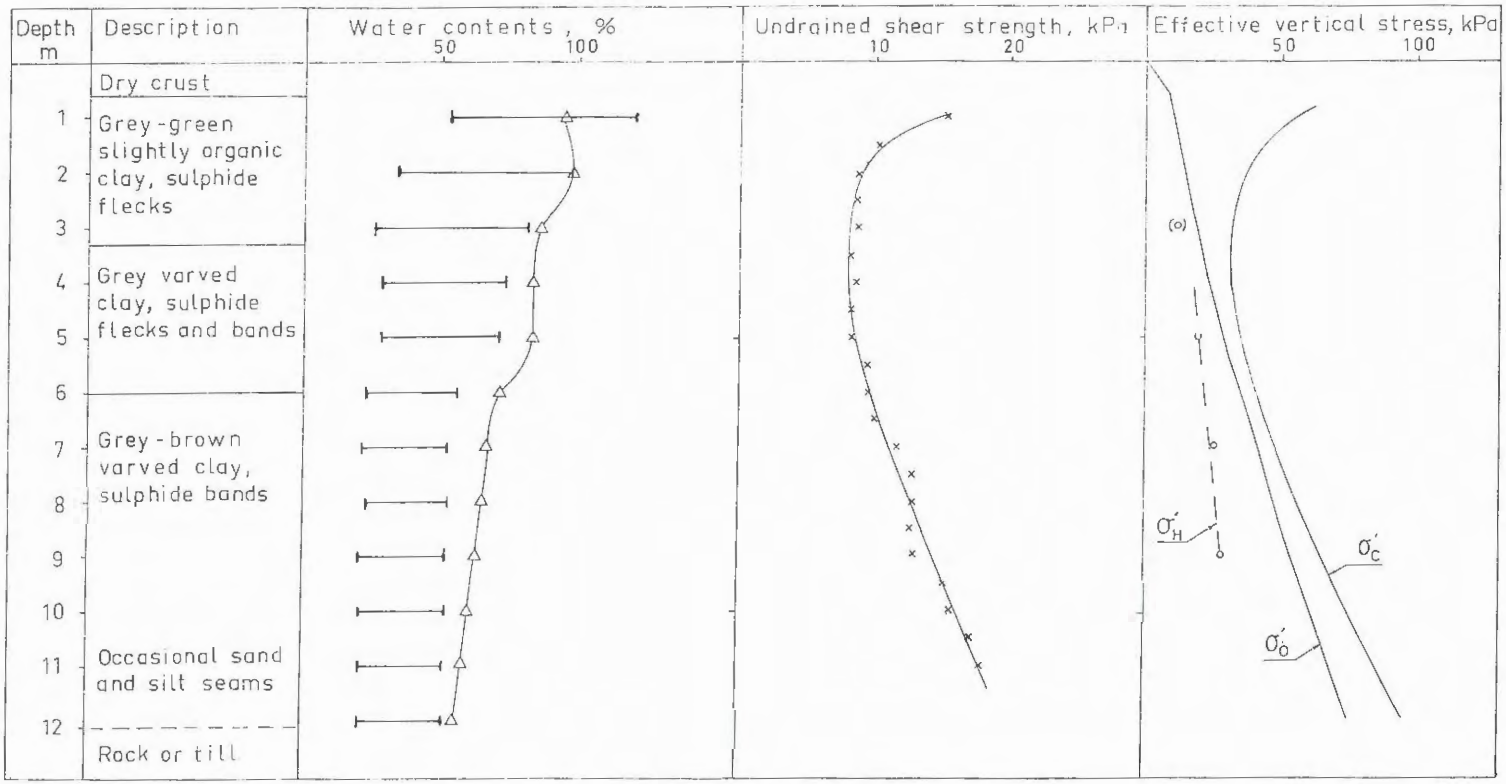


Fig. 23. Soil profile at Skå-Edeby.

The seismic cone tests were performed in early December 1989. The results were very uniform and show a continuous increase in shear modulus with depth without any indication of abrupt changes in soil properties, FIG. 24. This is in direct agreement with what might be expected from the other soil properties in the profile.

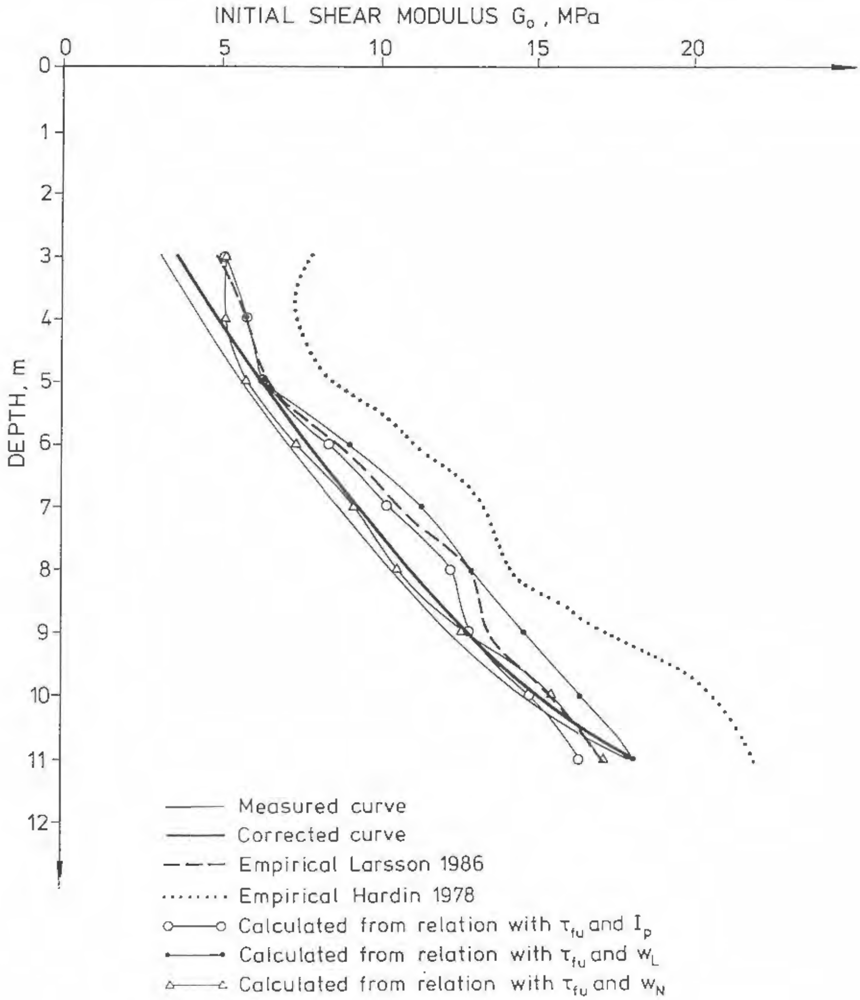


Fig. 24. Results from the seismic tests at Skå-Edeby.

• Norrköping

The test field in Norrköping has been used by SGI in recent years mainly to study developments of stresses and strains in connection with slope failures and to develop permanently installed control and alarm systems for slopes. This project and the associated site investigations have been managed by Möller (1990).

Within the present project, both dilatometer tests and CPTU tests have been performed.

The soil profile in the test field consists of about 14 metres of grey varved clay on top of friction material and rock, FIG. 25. The dry crust is about 1 m thick, but fissures and root threads extend to 2 m depth. Thin layers or seams of silt occur at about 5 m depth and from 7 m depth thin silt layers occur regularly. These layers become thicker with depth. The natural water content is higher than the liquid limit and varies from 130% at 3-4 m depth to about 40% in the silty bottom layers. The variation is relatively large because of the silt layers. The density varies between 1.45 t/m^3 in the upper layers to 1.80 t/m^3 in the bottom layers with frequent thick silt layers.

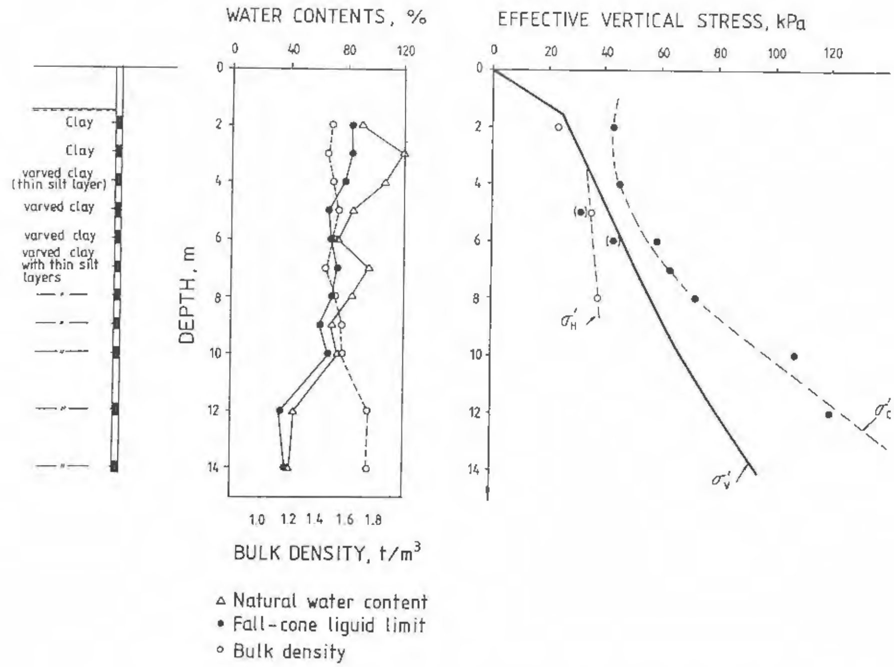


Fig. 25. Soil profile at the test site in Norrköping.

The free ground water level is normally located 1.5 m below the ground surface and the pore water pressures are hydrostatic from this level.

The preconsolidation pressures in the clay have been determined by CRS-oedometer tests. Because of the frequent silt layers, it was difficult to obtain good quality undisturbed samples with the standard piston sampler. This was observed, for instance from the relatively low shear strengths obtained in fall-cone tests. A good picture of the consolidation in the profile was obtained, however, by performing a large number of tests and eliminating all values judged to emanate from disturbed specimens. The overconsolidation ratio is thereby estimated to have a minimum of about 1.2 between 4 and 6 m depth. Above 4 m depth, the overconsolidation ratio increases rapidly because of crust effects and below 6 m depth it gradually increases to 1.5 - 1.6 at 10 m depth.

The undrained shear strength has been determined in direct simple shear tests and in a large number of vane shear tests. The direct simple shear tests have been reconsolidated to in situ stresses but, because of sample disturbance, there is a risk that the shear strength values in some cases are somewhat too low as not all the previous consolidation effects have been recreated. Correspondingly, several of the vane tests give too low values in those parts of the profile where silt layers are frequent and the disturbance at insertion of the vane is unusually large. Both types of tests, however, show a pronounced hump in the shear strength-depth relation at about 8 m depth. The sensitivity of the clay varies between 10 and 20. The estimated shear strength is shown in FIG. 26.

The horizontal stresses in the ground has been measured at three levels by earth pressure cells.

The soil investigations have later been supplemented by dilatometer tests. The results have been reported in detail by Larsson and Eskilson (1989). Also these tests show the pronounced hump in the shear strength-depth relation at about 8 m depth.

The tests with the seismic cone were performed in September 1989. These were the first tests performed with this particular equipment and a number of additional tests were performed. In these tests, the importance of the direction of the seismometer was tested, both by rotating the cone at a couple of levels and performing tests with various directions of the seismometer and by performing a complete sounding with the seismometer turned 90^0 to the applied shear wave direction. The results indicated that the direction of the seismometer was very important in the upper part of the profile, but gradually decreased to become almost insignificant in the bottom layers. This implies that it is very important to start a seismic sounding with the cone properly oriented, but a minor gradual rotation occurring successively during penetration to deeper layers may not affect the results too seriously.

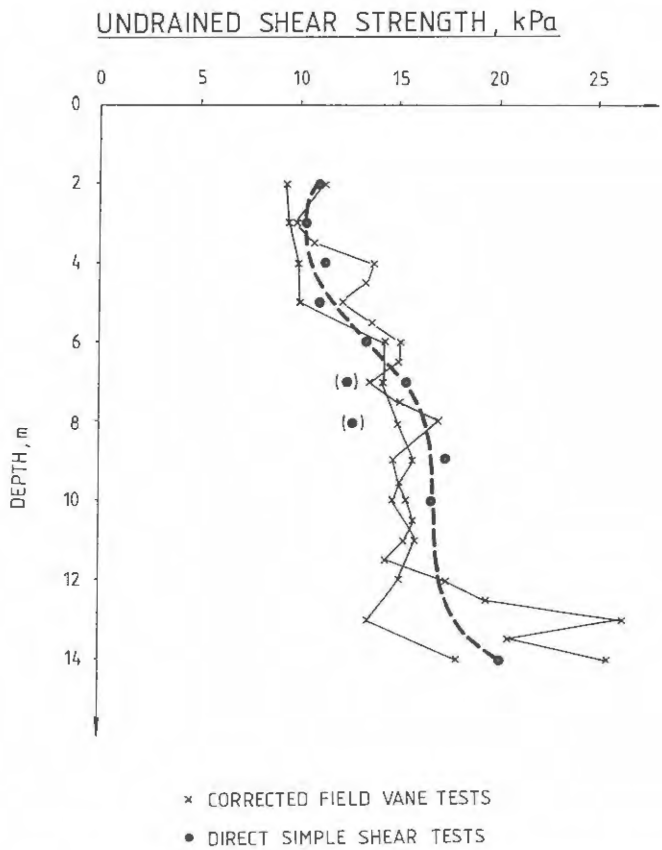


Fig. 26. Undrained shear strength at the test site in Norrköping.

In spite of the silt layers, the results were relatively uniform. The measured values from the two standard soundings, which were slightly separated in terms of testing level, are shown in FIG. 27. The same trend, with a hump in the modulus-depth curve, as for the shear strength around 8 m depth can be observed. The rapid increase in shear modulus at 11-12 m depth is more associated with a drop in water content (void ratio) and consistency limits. The trend in the shear modulus with depth is thus very much as could be expected from the other soil properties.

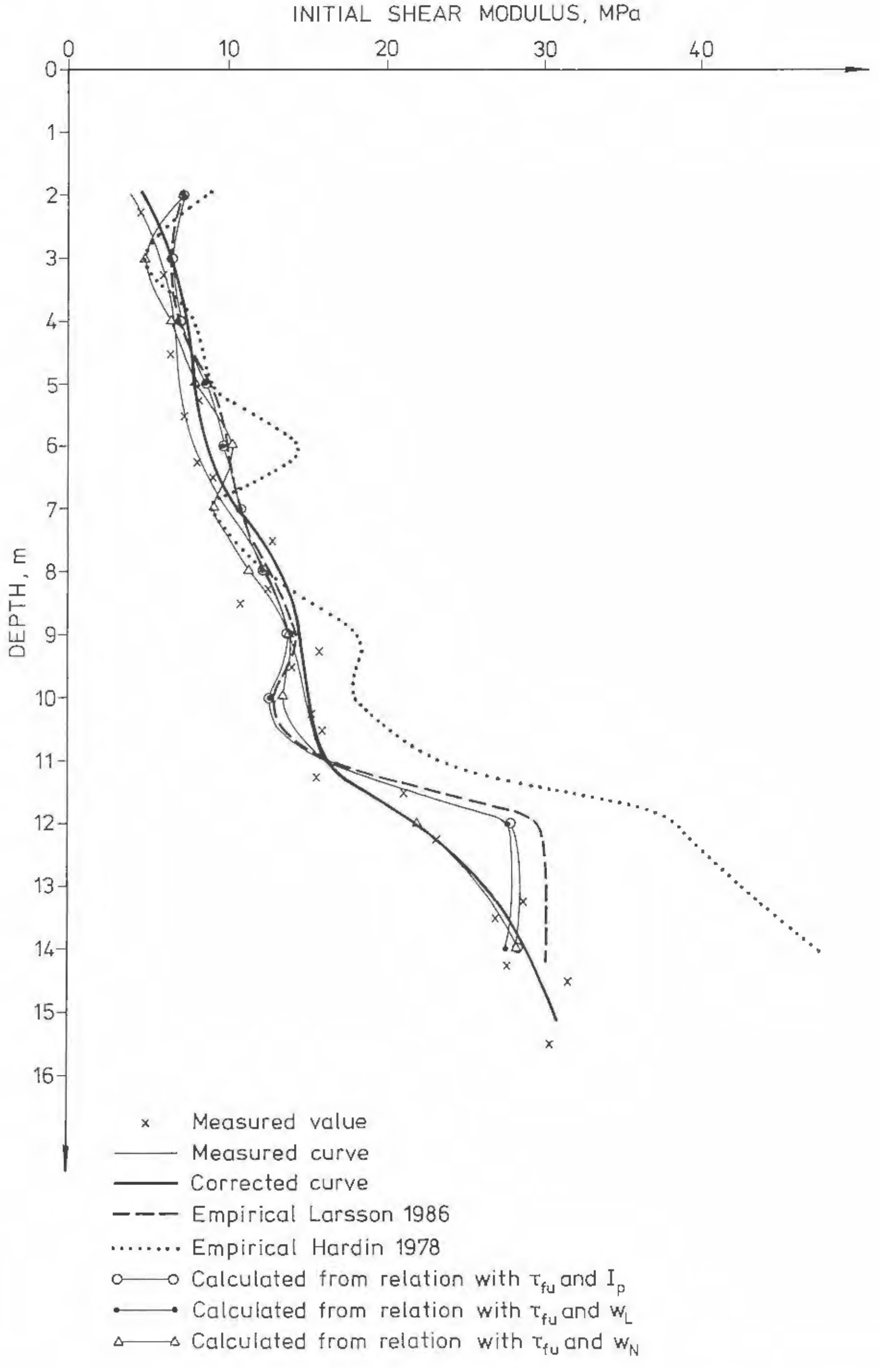


Fig. 27. Results from the seismic tests at the test site in Norrköping.

• Bäckebol

Bäckebol is situated on the island of Hisingen in the northern part of the city of Gothenburg. The test area, which has been used extensively by both SGI and Chalmers University since the end of the 60s, is located near the river Göta Älv. The layers of clay in the area are about 40 m deep.

The first investigations in the area were made by SGI and mainly concerned the behaviour of long driven piles in clay (Fellenius 1971, Bjerin 1977), FIG. 28.

In the main test area, mainly used by Chalmers University, a large number of projects have been carried out concerning most aspects of clay behaviour in different types of loading. Also seismic tests have been performed (Andréasson 1979).

The upper 10 m of clay has been extensively investigated from both geotechnical and geological aspects (Push 1970, Torstensson 1973, Sällfors 1975 and Larsson 1975 and 1981). The clay here consists of marine post-glacial clay, with illite as the dominating clay mineral and quartz and feldspar in the silt particles. The clay content is around 60% and the organic content is less than 1%. The upper metre of soil consists of dry crust, followed by grey clay with infusions of shells. Root channels from vegetation occur down to 3 m depth.

The water content down to 10 m depth is 70 to 90% and is slightly above the liquid limit, while the plasticity index is around 50%. The clay is overconsolidated down to 10 m depth and is thereafter largely normally consolidated.

The ground water level is about 0.4 m below the ground level and the pore pressures below are mainly hydrostatic from this level.

The coefficient of earth pressure at rest has been thoroughly investigated in the field as well as in the laboratory, FIG. 29.

The undrained shear strength has been determined in a comprehensive testing program involving both field vane tests and model pile tests, whereby also the correction factors for the field vane test have been checked. In the laboratory, comprehensive testing has been performed with both triaxial and plane-strain tests. The sensitivity of the clay is around 17.

Dilatometer tests have also been performed and the results are included in the basis for the SGI-evaluation of this type of test (Larsson and Eskilsson 1989).

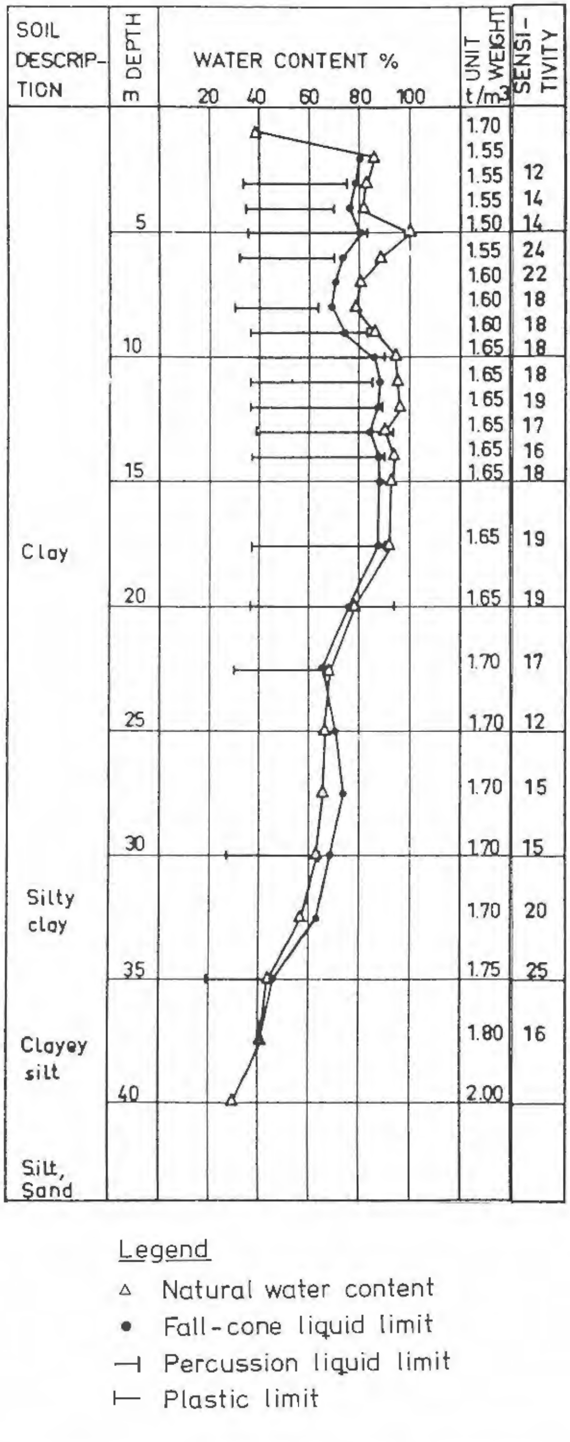
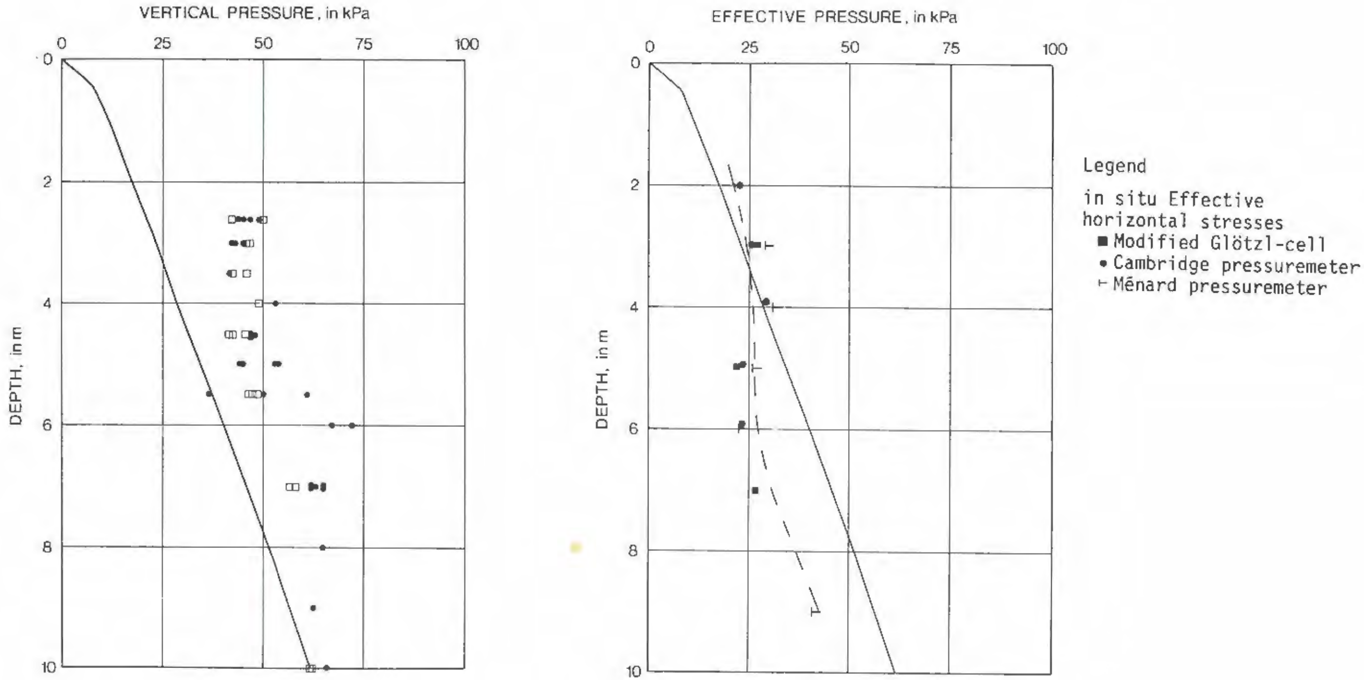


Fig. 28. Soil profile at Bäckebol, after Fellenius (1971).



• σ_c' determined from CRS-tests; □ σ_c' determined from full-scale field tests.

Solid curve: in situ vertical effective stress.

Fig. 29. Preconsolidation pressures and horizontal stresses in the main test area at Bäckebol, after Sällfors (1975) and Larsson (1975). The broken line in the second diagram represents horizontal stresses estimated from laboratory tests.

The underlying clay has not been investigated in such detail. The results from three nearby investigations; test area II located 500 m north of the main test area (Andréasson 1979), the pile test area 100 m to the east (Fellenius 1971 and Bjerin 1977) and the test area used in the present investigation 150 m to the south, all indicate that there is a rather abrupt change in the soil properties at 10 m depth. The undrained shear strength rises with a jump in the strength-depth relation and the water contents and consistency limits, which decrease slightly with depth in the upper 10 metres, suddenly increase at the 10 m level, FIG. 30.

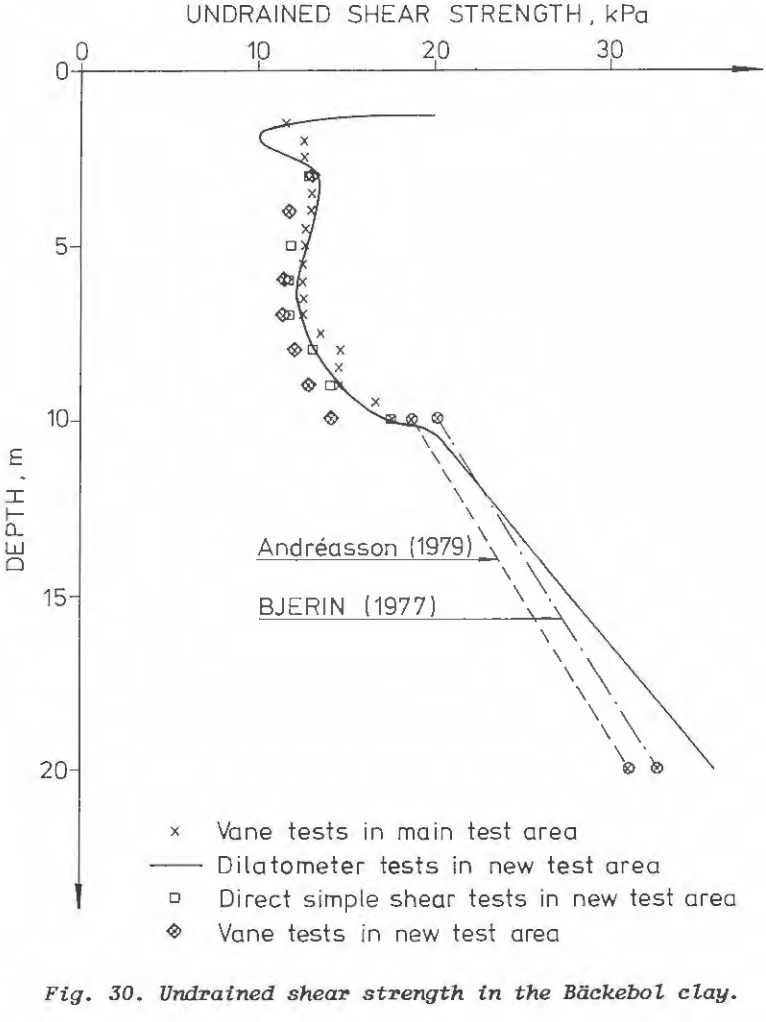


Fig. 30. Undrained shear strength in the Bäckebol clay.

The same type of sudden increase at a corresponding depth can also be observed at the test field in Tuve, which is located in a parallel valley 3 km to the west on the same island, and has there been explained in detail by the geological investigation made after the landslide in 1977 (Cato et al 1981).

The main test field has now been built upon and the present investigation was made on an undeveloped site located about 150 m to the south of the main test area. A supplementary soil investigation was made at the new site with sampling and field vane testing down to 10 m depth and dilatometer tests down to 20 m depth in order to check that the soil conditions were similar. Only minor differences were observed. A comprehensive programme of CPTU-testing and two seismic cone tests was also carried out down to 20 m depth.

Seismic tests had previously been conducted by Andréasson (1979). In test area II (located 500 m to the north of the main test area) cross-hole tests and down-hole tests were compared. As stated by Andréasson, the results are quite consistent, FIG. 31.

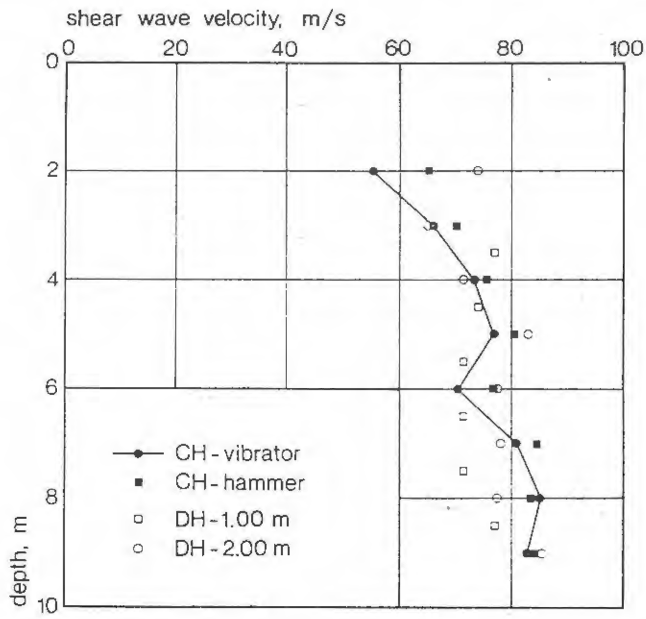


Fig. 31. Comparison between cross-hole and down-hole test results from test area II (Andréasson 1979).

In the main test area, only cross-hole tests were performed, but with slightly different techniques. The results that, according to Andréasson, are most realistic are compared to the results from the seismic cone tests in FIG. 32. As can be seen from the figure, also these two sets of results are compatible.

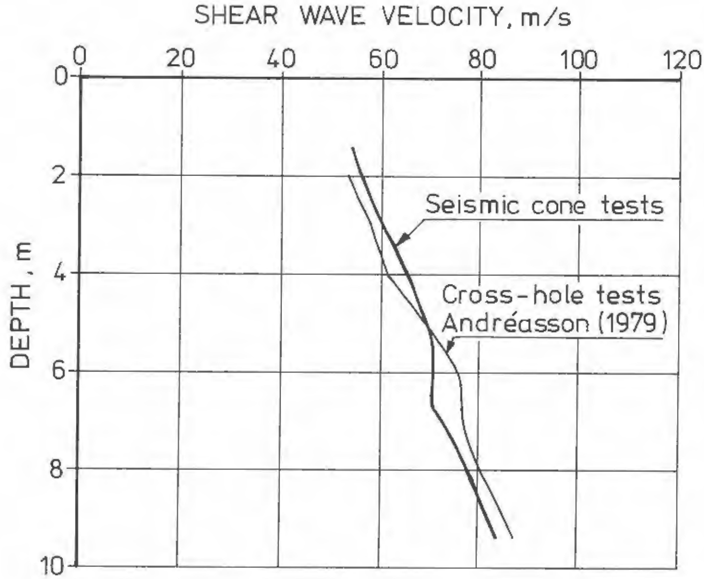
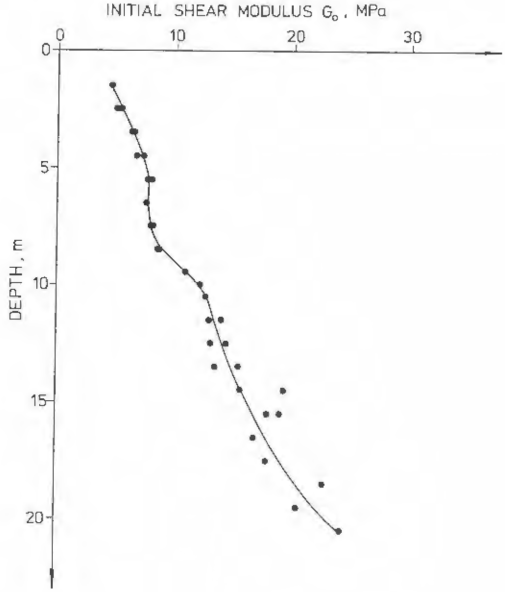
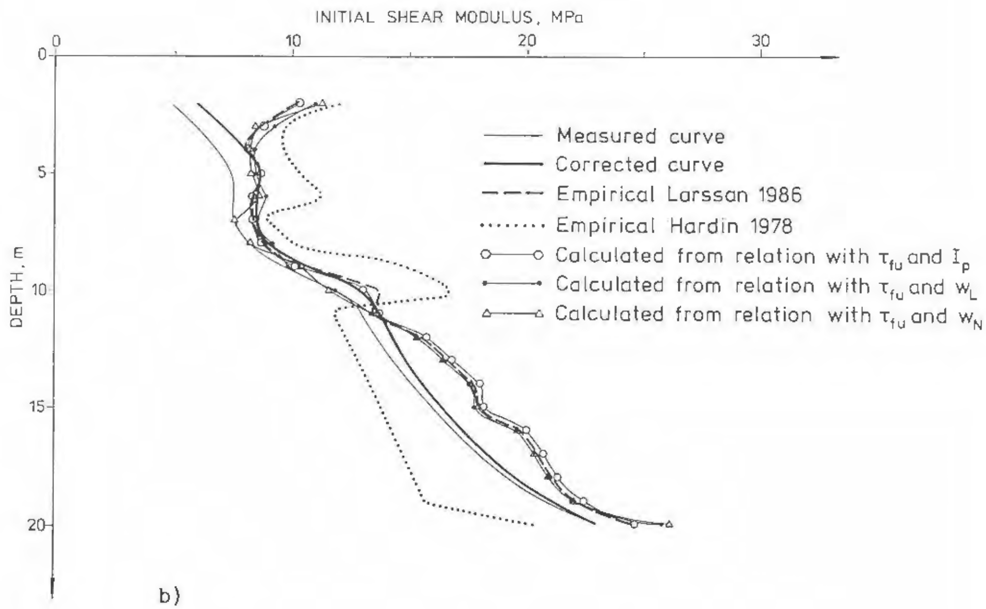


Fig. 32. Comparison between test results obtained in cross-hole tests in the main test area at Bäckebol (Andréasson 1979) and results from the seismic cone tests about 150 m away.

The results from the seismic cone tests at Bäckebol are very uniform in the upper 11 metres, FIG. 33. Below this depth, there is some spread, especially around 15 m depth, but in general the results are consistent. Because of the rather special stratigraphy, there is relatively little variation both in water contents and consistency limits for the entire 20 m thick upper clay profile which was investigated. The measured shear modulus is also found to mirror the relation between undrained shear strength and depth with the same sudden increase in the modulus-depth curve at about 10 m depth.



a)



b)

Fig. 33. a) Results from the two seismic soundings at Bäckebol.

b) Comparison between average results from the seismic cone tests at Bäckebol and empirical relations for the initial shear modulus.

• Tuve

The test area in Tuve is located in the Kvilledalen valley on the island of Hisingen in north Gothenburg. A major landslide occurred in the area in 1977 and was followed by extensive investigations concerning geology, geo-hydrological conditions and soil properties (Cato et al 1981, Blomqvist and Gustavsson 1981 and Larsson and Jansson 1982). Investigations were also made outside the actual slide area and especially around Point 18, which is located at the bottom of the valley about 150 metres to the north of the slide area and about 100 metres away from the small brook at the centre of the valley.

The soil here consists of about 27 metres of marine clay on top of sand and bedrock. The distribution of certain chemical and physical parameters in the clay can be seen in FIG. 34.

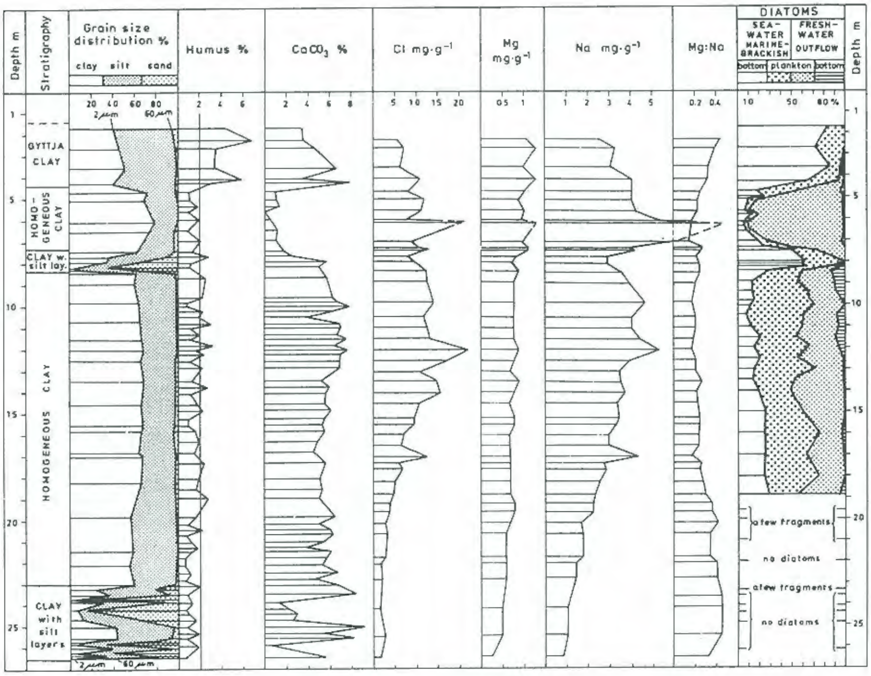


Fig. 34. Distribution of certain chemical and physical parameters in the soil profile at Point 18 in Tuve, after Cato (1981).

The clay profile can roughly be divided into 5 main layers, FIG. 35. At the bottom, there is a varved glacial clay with sand and silt layers resting on sand and bedrock (E). Above the glacial clay is a thick homogeneous clay layer (D). Zone C, which is about one metre thick, marks a change in the previous sedimentation history. The layer was deposited after a rapid drainage of 10,000 km^3 of previously ice-dammed water from the Baltic into the Skagerrack about 10,000 years ago. This drainage caused erosion in the inland and redeposition in the coastal areas. On top of Zone C, which contains several thin silt layers, there is a new zone of homogeneous clay characterized by a very high clay content(B). The uppermost clay layer (A) has a fairly high organic content and is classified as organic clay.

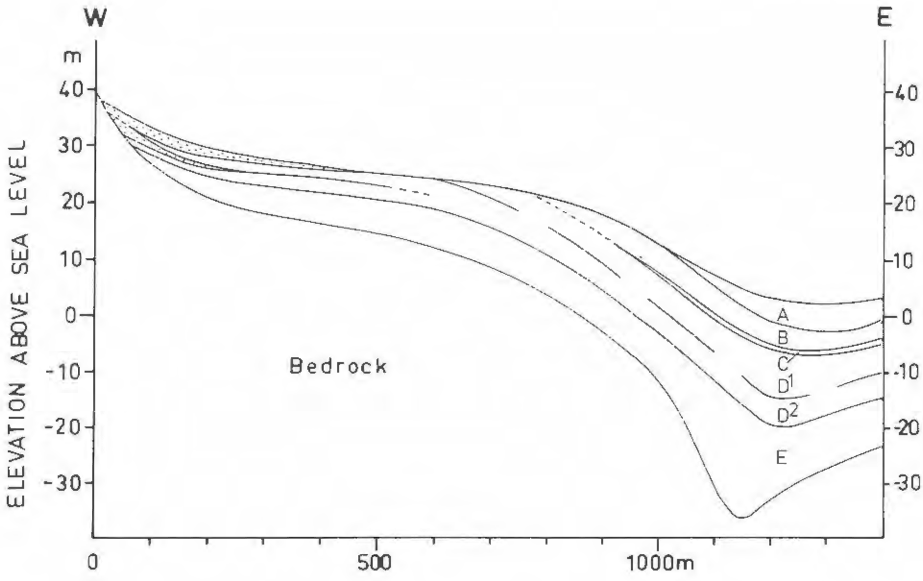


Fig. 35. Generalized soil profile at the slide area in Tuve, after Cato et al (1981).

Because of the process of land-heave, which occurred simultaneously with the sedimentation process, the upper three layers are found only in the centre of the valley and the thickness of the organic clay layer increases towards the brook.

Point 18 was located inside a cultivated area. For reasons of easy access, the present investigation was made about halfway between Point 18 and the brook. Supplementary investigations were made with piston sampling to 10 m depth, two soundings with dilatometer tests and a large number of CPTU soundings.

The soil data have then been estimated by comparison of the soil profiles and the data obtained in the present investigation and corresponding data obtained in Points 18 and 2 in the previous investigation (Larsson and Jansson 1982), FIG. 36. Point 2 is located inside the slide area at about the same distance from the brook as the present investigation and about 200 m downstream.

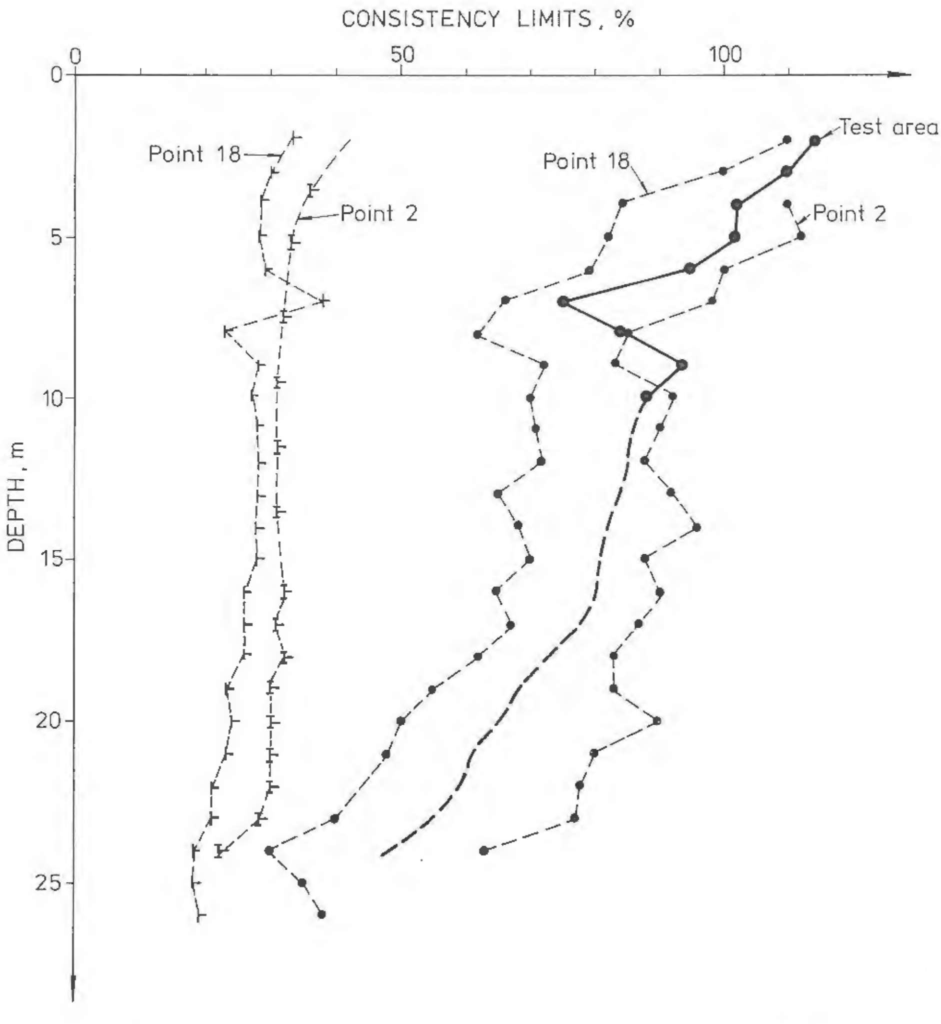


Fig. 36. Consistency limits in the area for seismic tests in Tuve.

The upper 6 m of soft soil consist of organic clay with infusions of plant remnants and shells. The crust is only about half a metre thick. At 7 metres depth, the organic content is insignificant, but the content of shells increases. The transition layer with thin silt layers occurs at around 8 metres depth and below this level there is homogeneous clay. The dilatometer tests and the CPTU tests indicate that there are infusions or layers of some kind, probably stones or shells, at a depth of 14 to 16 metres, but this was not revealed by the previous investigation.

The liquid limit in the organic clay decreases from over 110% below the crust to about 95% at 6 m depth. In the transition layer with shells and silt layers, it drops to around 80% and then increases to again become about 95% at 9 m depth. It then gradually decreases and is estimated to be 50% at 24 m depth. The plastic limit is up to 40% in the organic clay and around 30% in the major part of the thick layer of homogeneous clay. The water content is consistently around 10% higher than the liquid limit for the whole profile.

The bulk density is about 1.4 t/m^3 in the organic clay, about 1.5 t/m^3 between 7 and 10 m depth, and then gradually increases to become 1.7 t/m^3 at 24 m depth.

The free ground water level is very high. At the time of testing in May 1990, it was about 0.2 m below the ground surface but, since the landslide caused a slight damming of the brook, the area is flooded both seasonally and also after heavy rains. The pore pressures are not hydrostatic. Water infiltrates into the more pervious bottom layers at the higher valley sides and there is an artesian water pressure in the sand layers between the clay and bedrock. At the time of testing, this artesian water pressure in the test area corresponded to a water head 3.25 m above the ground level.

The area is barely accessible for truck-mounted equipment and then only under relatively dry conditions.

The effective stresses in the ground are very low because of the ground water conditions. The preconsolidation pressures are also very low, 15-30 kPa, in the upper 7 metres of organic clay. Yet, there is an overconsolidation ratio of 2-1.5 in this layer in relation to the effective stresses at the time of testing. At 7-8 m depth, where the transition layer occurs, there is a jump in the preconsolidation pressures, which increase significantly, FIG. 37.

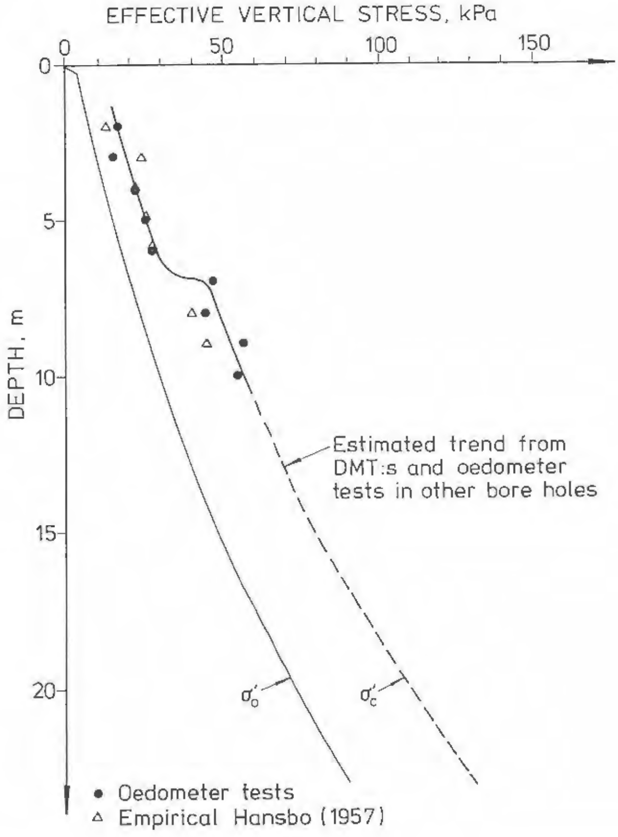


Fig. 37. Preconsolidation pressures at the area for seismic tests in Tuve.

The previous investigations in the area had shown that the preconsolidation pressures at greater depth could roughly be expressed as indicating that the soil there is normally consolidated for a ground water level 1 m below the ground surface and hydrostatic ground water conditions. This is supported by the oedometer tests below 7 metres depth in the test area and also by the results from the two dilatometer tests, which gave very consistent results.

The undrained shear strength has been measured by fall-cone tests and direct simple shear tests in the laboratory and the dilatometer tests in the field. The fall-cone tests have been checked against Hansbo's relation and can be considered relevant, especially for the organic clay (see Larsson et al 1984). The results from the dilatometer largely follow what might be expected from empirical correlations between undrained shear strength, preconsolidation pressure and consistency limits. They are further supported by the results from the CPTU tests, which show the same trend and give about the same values if a cone factor of 17 is used ($\tau_{fu} = (q_T-\sigma_v)/17$). The same hump is found in the shear strength-depth curve as for the preconsolidation pressure-depth curve (and which is also found at the nearby Bäckebol site), FIG. 38.

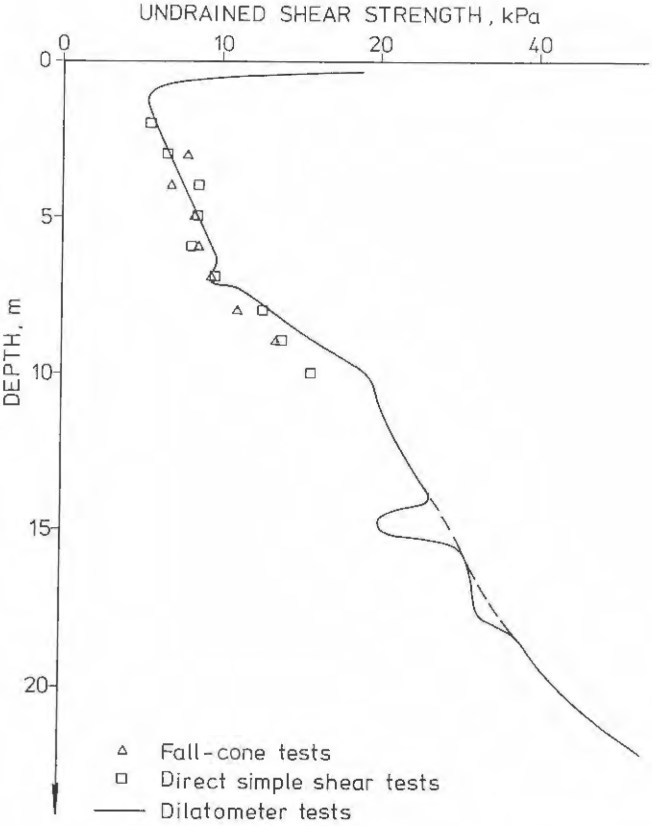


Fig. 38. Undrained shear strength at the area for seismic tests in Tuve.

The results from the seismic tests are shown in FIG. 39. The two soundings yielded very similar results, forming a smooth curve with depth and with a hump at 8 m depth. Some scatter and odd values were obtained between 14 and 17 m depth, similar to the results from the dilatometer tests and the CPTU tests. Apart from this, the results are very consistent and in accordance with what can be expected from the other soil properties at the site.

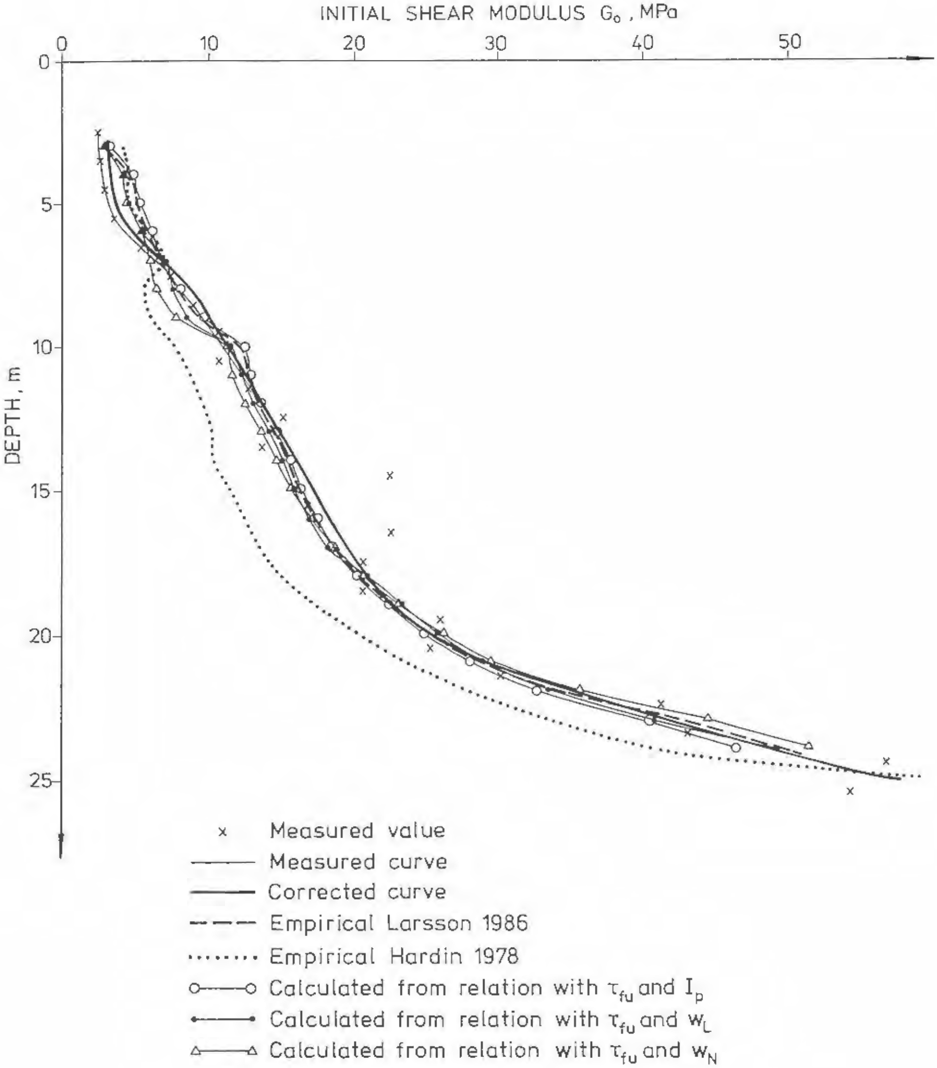


Fig. 39. Results from the seismic tests at Tuve.

• Välen

Välen is a test field in south 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 northern 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.

Seismic tests have previously been performed by Andréasson (1979) and the field has also been used by SGI for dilatometer tests (Larsson and Eskilson 1989) and in a larger investigation on the behaviour of organic clay and gyttja (Larsson 1990).

The soft soils at the site have a thickness of about 11 metres. The conditions in the area are fairly uniform, but the field is large and 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 previous field vane tests which were comprehensive at two levels, but for which a continuous profile was missing, and also to some degree the previous seismic tests. New field vane tests were performed in connection with the present investigation. A typical soil profile for the site is shown in FIG. 40.

The dry crust in the area is about 1 metre thick, but the effects of proximity 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 has been the focus of all investigations in the field. It 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 and are not investigated in detail.

Välen

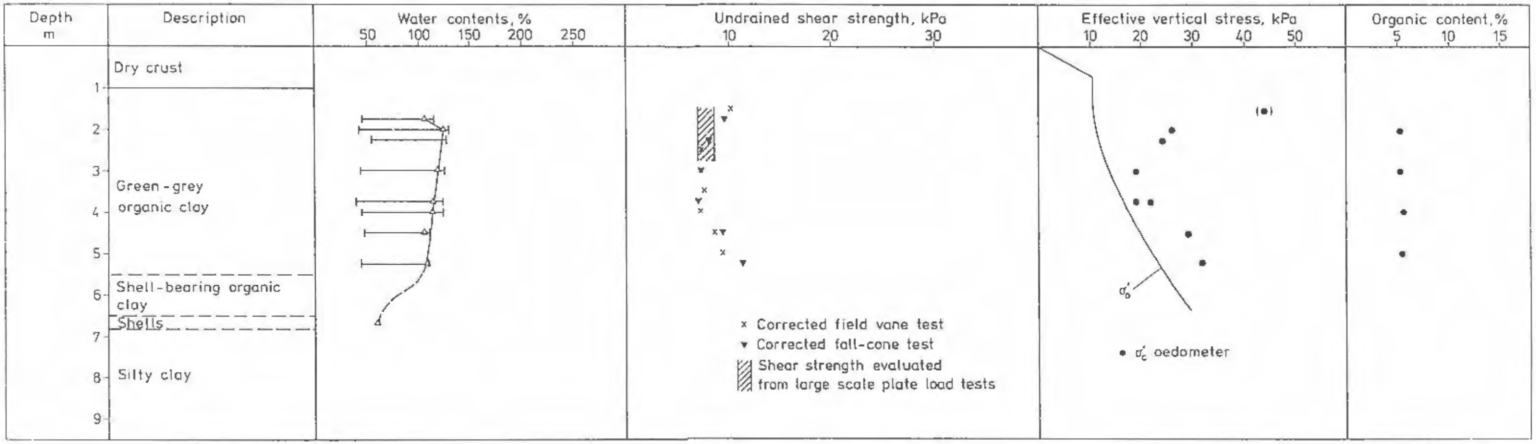


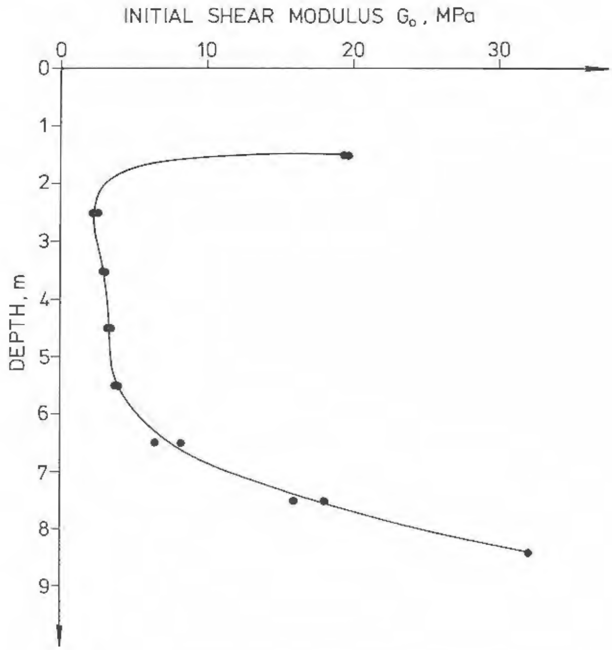
Fig. 40. Soil profile at Välen.

The natural water content in the organic clay between 2.5 and 5.5 metres of depth is approximately 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, but no such effects have been found in the part of the field used for the present investigation.

The results from oedometer tests and large-scale field loading tests show that the soil is overconsolidated at the top because of dry crust effects. The overconsolidation has a minimum of about 1.2 (at lowest ground water level) at a depth of 3.5 m and then increases further down. At the time of the present investigation in May 1990, the free ground water level was located 0.7 m below the ground surface.

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

The results of the seismic tests are shown in FIG. 41. In the organic clay, the results from the two soundings are almost identical. The results are also very much in the same order as can be expected with regard to the other soil properties. The somewhat low values just below the crust may to some degree depend on disturbance effects when penetrating through the relatively stiff crust into the underlying very soft soil.



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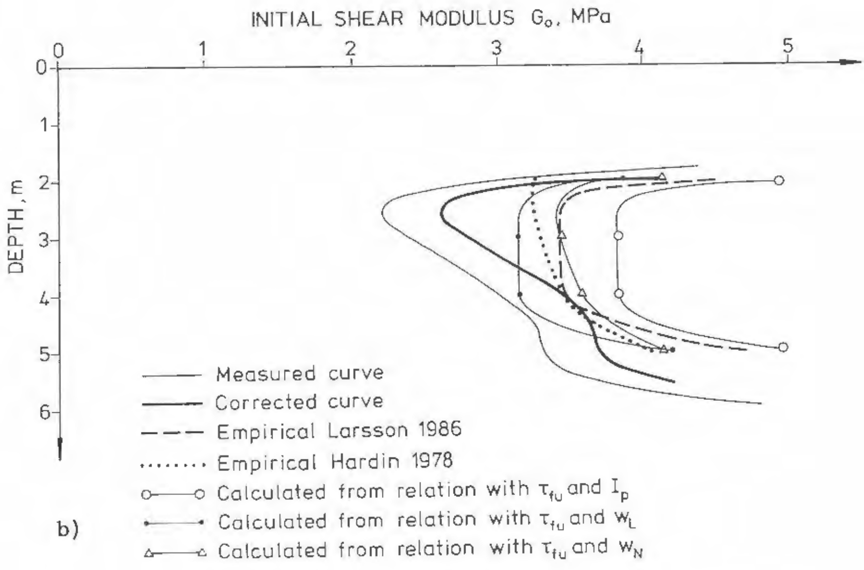


b)

Fig. 41. a) Results from the two seismic soundings at Välen.

b) Comparison between average results from the seismic cone tests at Välen and empirical relations for the initial shear modulus.

Seismic tests had previously been performed in another part of the field by Andréasson (1979). A comparison between the results from the two investigations shows that the results are very similar, FIG. 42, even if the results from the seismic cone are slightly on the low side for part of the profile. However, only minor changes in depth levels are required for the results to coincide.

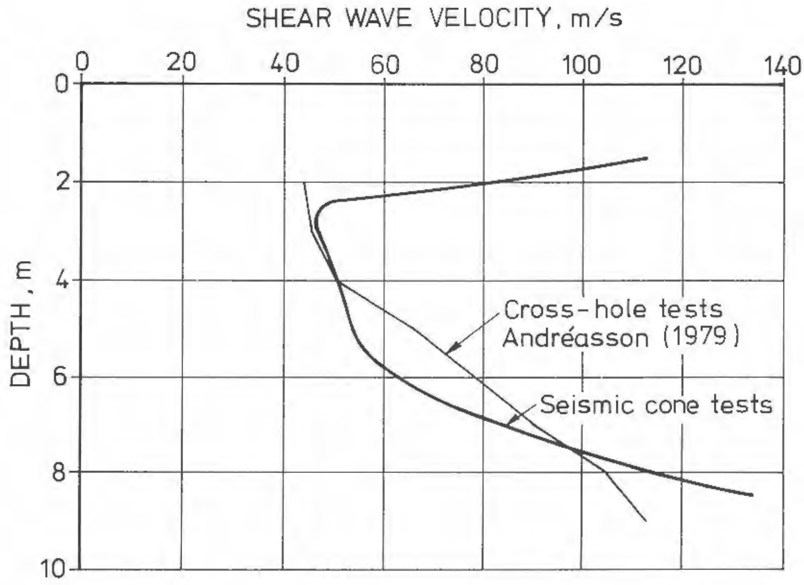


Fig. 42. Comparison between test results obtained in cross-hole tests in the test field at Välen (Andréasson 1979) and results from seismic cone tests performed some distance away.

• Munkedal

Munkedal is a community located close to the Swedish west coast about 100 km north of Gothenburg. Both the main road along the west coast (E4 to Oslo) and the railway line "Bohusbanan" run through the community. The central parts of Munkedal, including the railway station, are located on a long and fairly steep slope towards the river "Örekilsälven". The soil in the slope consists of over 30 metres of clay, which in some parts is highly sensitive or "quick". Directly below the railway station, the meandering course of the river had formed a U-shaped bend and, until recently, considerable erosion was in progress. A number of landslides also occurred here and in the vicinity along the river. During the 80s, it was therefore decided to thoroughly investigate the stability of the area. As a result of these investigations, it was decided to move the whole river away from the community and to build a large stabilizing berm in the former location of the river-bed (Bergdahl and Tremblay 1987).

During the soil investigations, areas with quick clay were found and in the present project it was decided to test the equipments in this type of soil. The investigations were made close to the former bore hole No. 8 where the thickness of the clay layer is around 30 metres and the sensitivity of large parts of the profile is in the order of 300. The area is located in the upper part of the slope just below the railway tracks. The previous investigations have been supplemented by field vane tests, dilatometer tests and CPTU tests in connection with the seismic tests.

The upper 1.5 m of the soil profile consists of varying fill material and dry crust. Below the crust, there is about 30 metres of medium stiff clay. The thickness of the clay layer varies somewhat because of a slight inclination of the ground surface and a steeper inclination of the underlying bedrock. Between the clay and the bedrock there is sand.

The clay layer is fairly homogeneous, but there are infusions of shells and also other harder objects have been encountered in the soundings at depths greater than 13 metres.

Thin silt layers have been found in the old samples at about 20 m depth. Samples taken just below this level are obviously disturbed and there are also large drops in the shear strengths measured by field vane tests and in tip resistances measured by CPTU tests. A similar but much smaller drop is observed in the dilatometer results and the excess pore pressures generated at soundings. Part of the large drops in measured shear strength at this level is believed to be disturbance effects when passing from a stiffer layer into a somewhat softer layer below. Samples have only been taken down to 23 m depth, but the results from the CPTU tests indicate that the clay below this level is varved and probably contains thin silt layers.

The liquid limit of the clay gradually decreases from about 65% at the top to 43% at 23 m depth. The water content is significantly higher and, except for a few metres just below the fill, the liquidity index is in the order of 2. The sensitivity here is around 300. Below 16 m depth, both the liquidity index and the sensitivity gradually decrease, but remain high. The plastic limit has not been measured at this site, but is estimated by comparisons with results from other clays in the same region. The bulk density of the clay increases from 1.65 t/m^3 at the top to 1.8 t/m^3 at 23 m depth.

The free ground water level is located about 1 metre below the ground level. The test area is located in the upper part of the slope and there is a downward gradient in the pore pressure. The water pressure measured in the sand layer below the clay corresponded to a water head 9 metres below the ground surface.

The upper part of the clay profile is overconsolidated. Results from oedometer tests and the results from the field vane tests and the dilatometer tests unanimously indicate that the overconsolidation ratio decreases to about 1.2 at 10 m depth. Below 20 m depth, the overconsolidation ratio estimated from the field tests drops to 1.0, but this is probably a result of the aforementioned disturbance effects and the overconsolidation ratio can be assumed to remain at about 1.2 with depth, FIG. 43.

The undrained shear strength has been measured by fall-cone tests in the laboratory and by field vane tests and dilatometer tests in the field, FIG. 44. Also the results from the CPTU tests have been used for support. The shear strength to be used in design and for correlations has then been estimated with consideration to reliability of the various tests and known disturbance effects in various soil conditions (Larsson et al 1984). Down to 14 m depth, the estimated shear strength is mainly based on the results from field vane tests and fall-cone tests. At greater depths, the fall-cone tests normally yield too low values and the estimated strength between 14 and 19 m depth is mainly based on the field vane tests. Below this level, the estimate is based on the dilatometer tests down to 23 m. At even greater depths, there is a significant spread also in the results from the two dilatometer soundings, but this does not affect the present investigation which stops at 23 m depth.

The clay at Munkedal is considerably stiffer than the clays in the previously investigated sites. Because of a pattern that had emerged from these previous investigations, and which indicated strain effects on the results, special care was taken to restrict the hammer blows in the seismic tests at the upper levels. The combination of these two facts made it possible to keep the shear strains in this part of the profile up to one magnitude lower than in the previous tests at the other sites.

The results from the two seismic soundings are very uniform and form a relatively even trend with depth, except for a small hump at 19 m depth, FIG. 45. The results thus mirror the trend in shear strength with depth. In general, however, the results are somewhat on the low side in relation to what might be expected down to about 16 m depth. The discrepancy, which is moderate, may possibly be related to the very high sensitivity in this part of the profile.

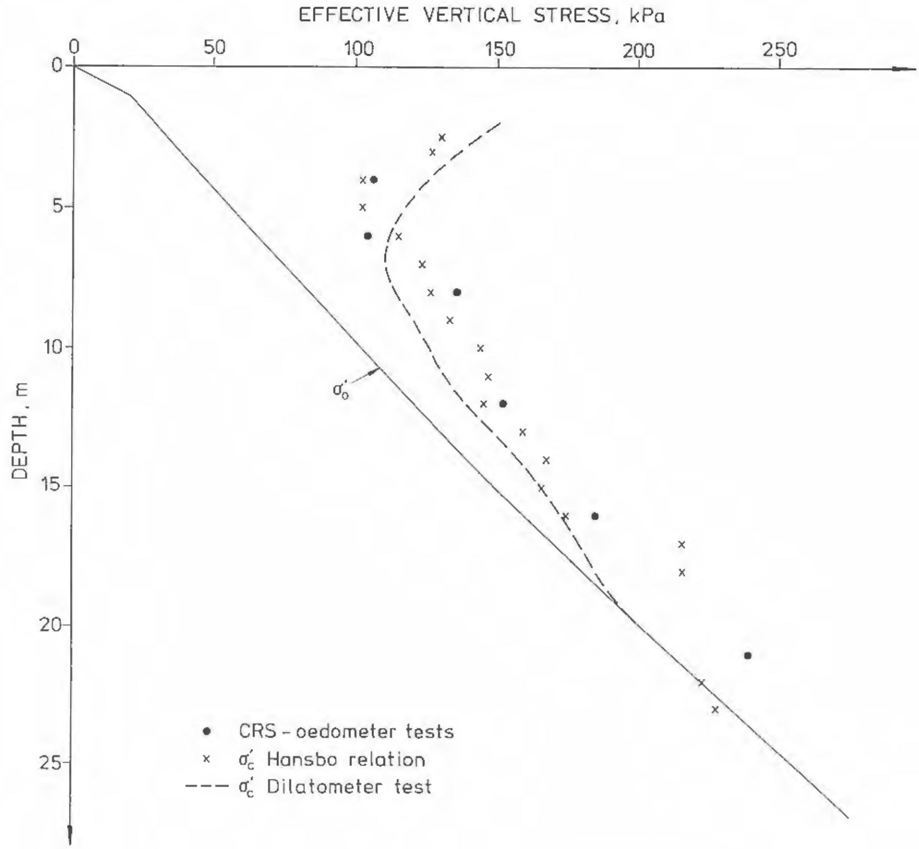


Fig. 43. Measured and estimated preconsolidation pressures at the area for seismic tests in Munkedal.

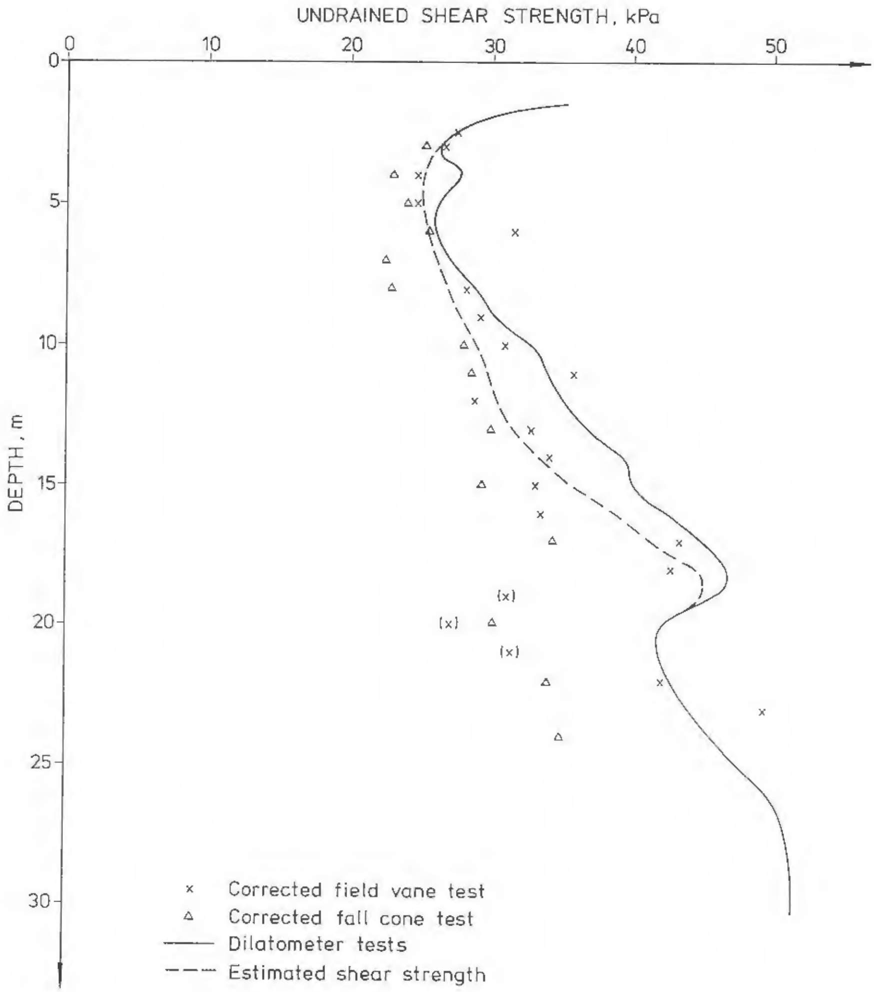


Fig. 44. Undrained shear strength at the area for seismic tests in Munkedal.

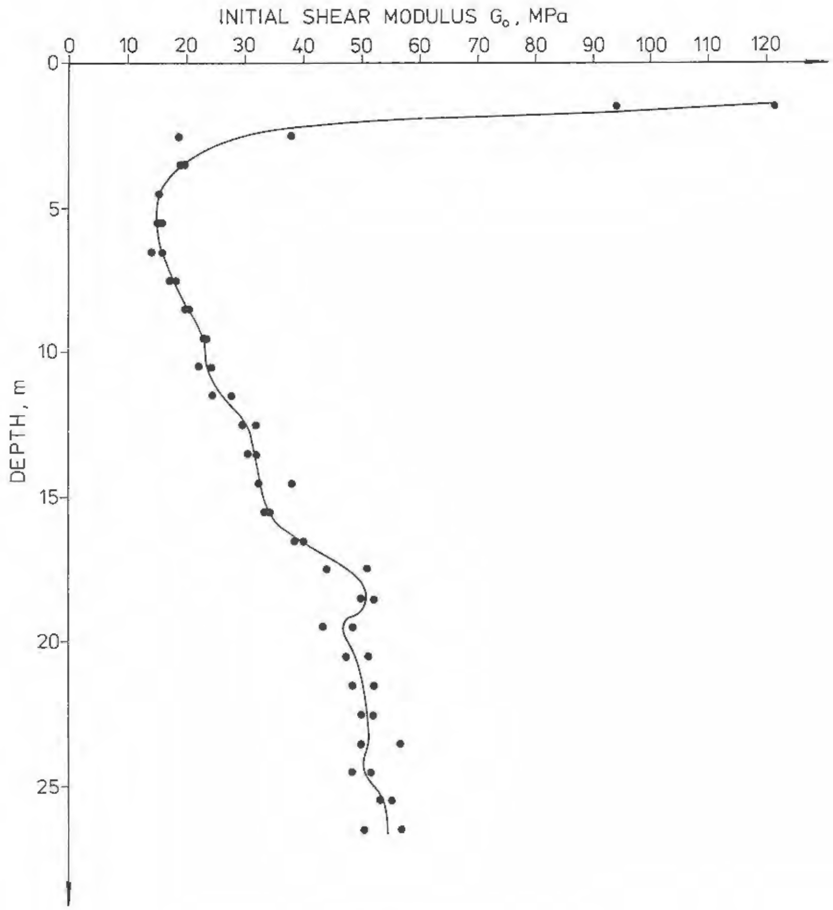


Fig. 45. a) Results from the two seismic soundings at Munkedal.

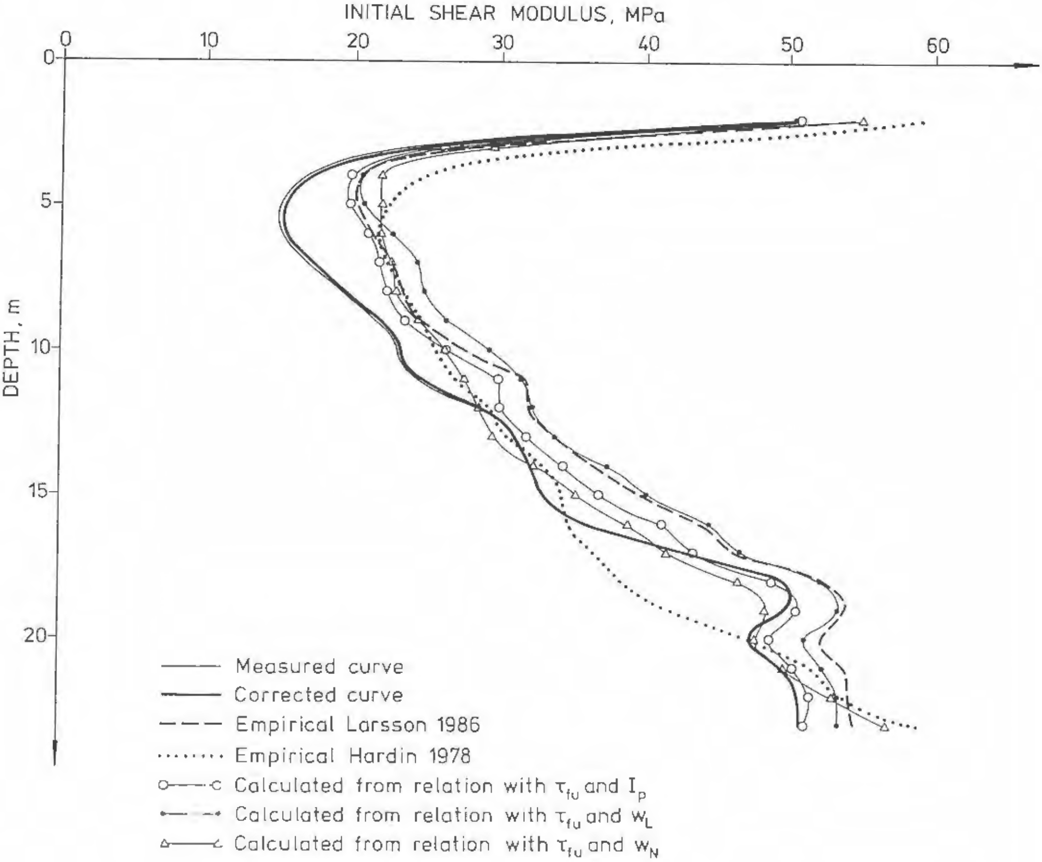


Fig. 45. b) Comparison between the results from the seismic cone tests at Munkedal and empirical relations for the initial shear modulus.

• 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 performed at the site in connection with a large investigation on properties of organic soils (Larsson 1990). The pore water pressures have also been measured.

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 m 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. 46.

The organic content in the clayey gyttja is between 10 and 11 %. The natural water content is slightly lower than the liquid limit, which varies between 150 to 175 % and 180 to 205 % respectively. The natural water content in the lower layers is only 30 to 50 %. The bulk 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 several 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.

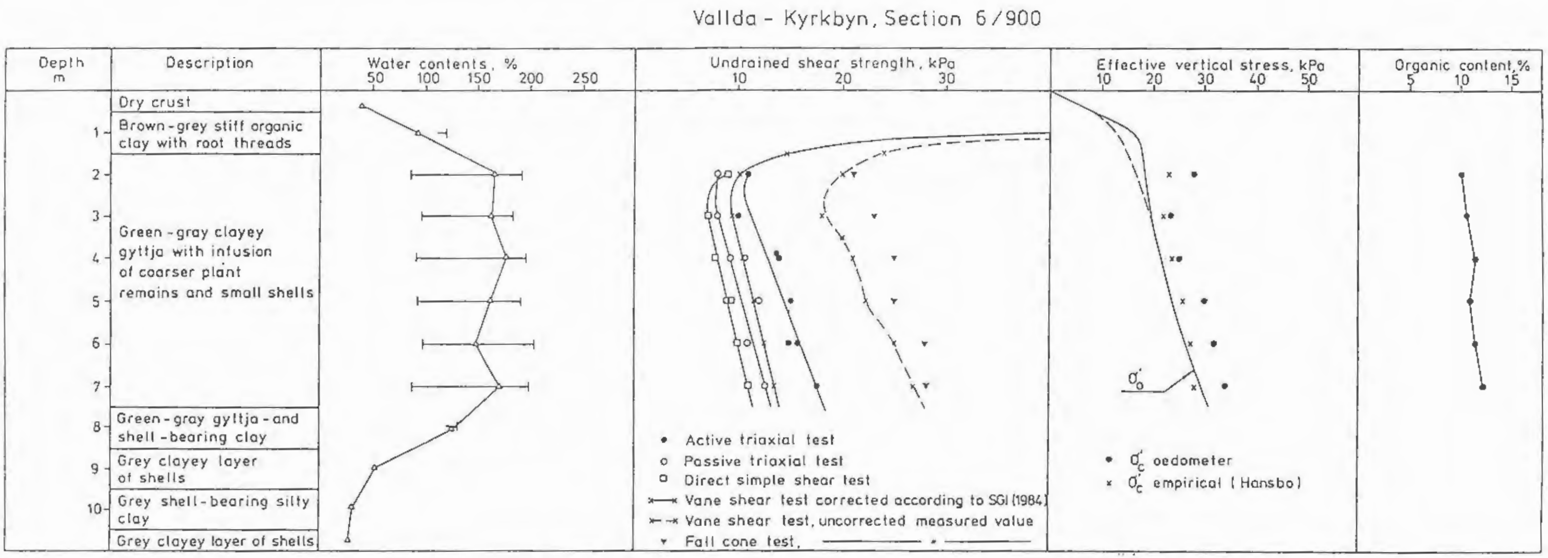


Fig. 46. Soil profile at Section 6/900.

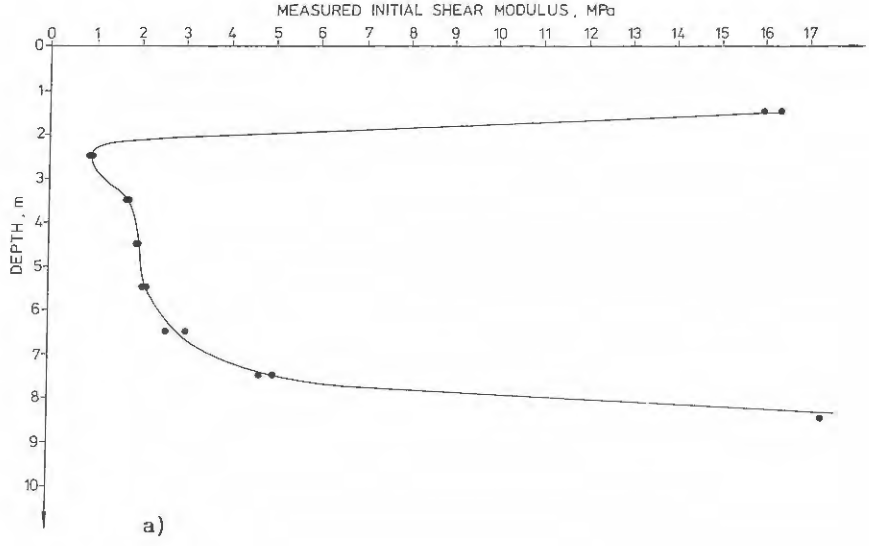
The consolidation parameters have been determined in several series of CRS tests with different rates of deformation, in several series of standard incremental tests and in tests with other loading procedures (Larsson and Sällfors 1975). Overconsolidation effects in and just below the dry crust extend 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.

Dilatometer tests have also been performed in the field. The results have been reported in detail by Larsson and Eskilsson (1989).

The seismic tests at Section 6/900 were performed with great restrictions on the hammer blows. The impact energy was only high enough to trigger the oscilloscope and in this way the shear strains in the tests were kept at a minimum. The results from the two nearby soundings were very uniform, FIG. 47a.

A comparison between the evaluated shear moduli and empirical relations used for clays shows a good correlation for the relations based on stress history and void ratio (Hardin 1978) and on undrained shear strength and liquid limit. The void ratios have been calculated from the water contents and the organic contents and with an assumed density of the organic matter of 1.4 t/m^3.

The relations based on plasticity index, on the other hand, yield initial shear moduli that are up to twice the measured values, FIG. 47b.



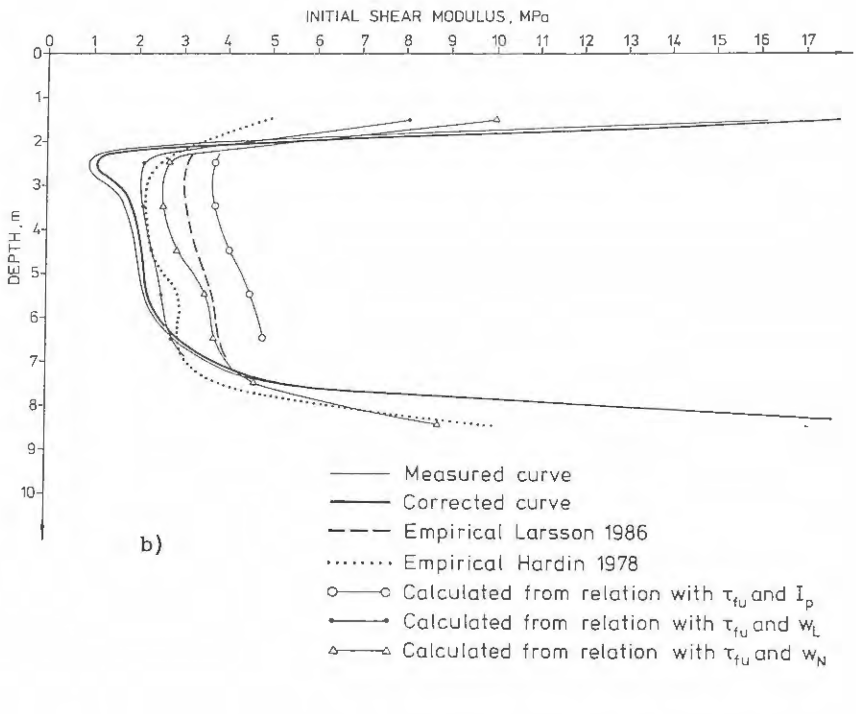


Fig. 47. a) Results from the two seismic soundings at Section 6/900.

b) Comparison between average results from the seismic cone tests at Section 6/900 and empirical relations for the initial shear modulus.

• 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. 48.

The same type of investigations as at the previous site have also been made here (Larsson 1990, Larsson and Eskilsson 1989). The natural water content and the liquid limit in the clayey gyttja vary from 135 to 155 % and 155 to 170 % respectively. The bulk 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 at 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 seismic tests were performed with the same restrictions on the hammer blows as at the previous site. The results from the two seismic soundings were very uniform, Fig. 49a.

Also here the results showed reasonable correlations with the empirical relations based on stress history and void ratio and on undrained shear strength and liquid limit, whereas the relations based on plasticity index yielded initial shear moduli about twice the measured values, FIG. 49b.

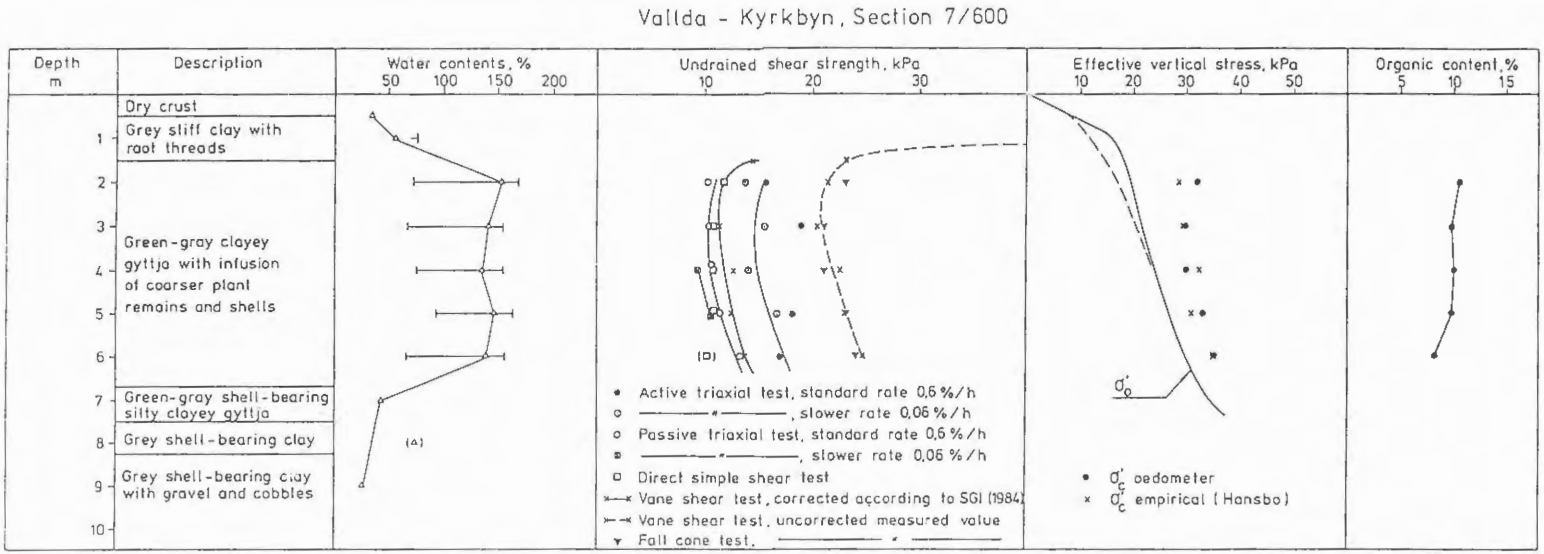


Fig. 48. Soil profile at Section 7/600.

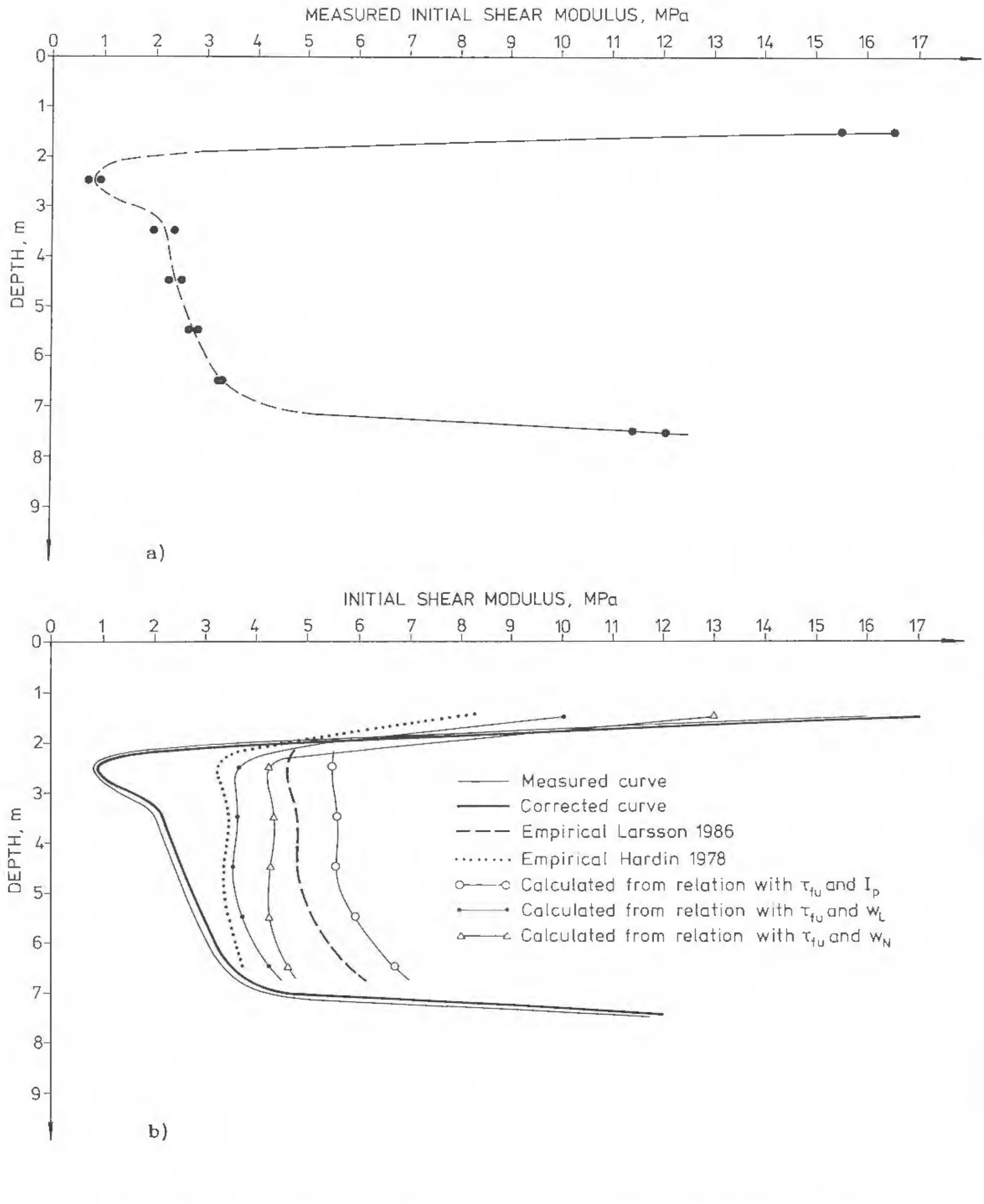


Fig. 49. a) Results from the two seismic soundings at Section 7/600.

b) Comparison between average results from the seismic cone tests at Section 7/600 and empirical relations for the initial shear modulus.

3.3.3 NGI Sites

- Onsöy

The test site at Onsöy has been described in detail by e.g. Lunne et al (1976) and Lacasse and Lunne (1983). The soil deposit consists of a weathered crust less than one metre thick underlain by two different layers of clay over bedrock. The upper clay layer is 8 m thick. It contains iron spots, organic matter and shell fragments and has an average plasticity index of 23%. The lower clay layer is more plastic with a plasticity index of about 36% and has a thickness of about 36 m. The soil investigations mainly stop at about 20 m depth. The natural water content is fairly constant down to this depth, but an increase from about 60 to 65% can be observed at the intersection between the two clay layers. FIG, 50.

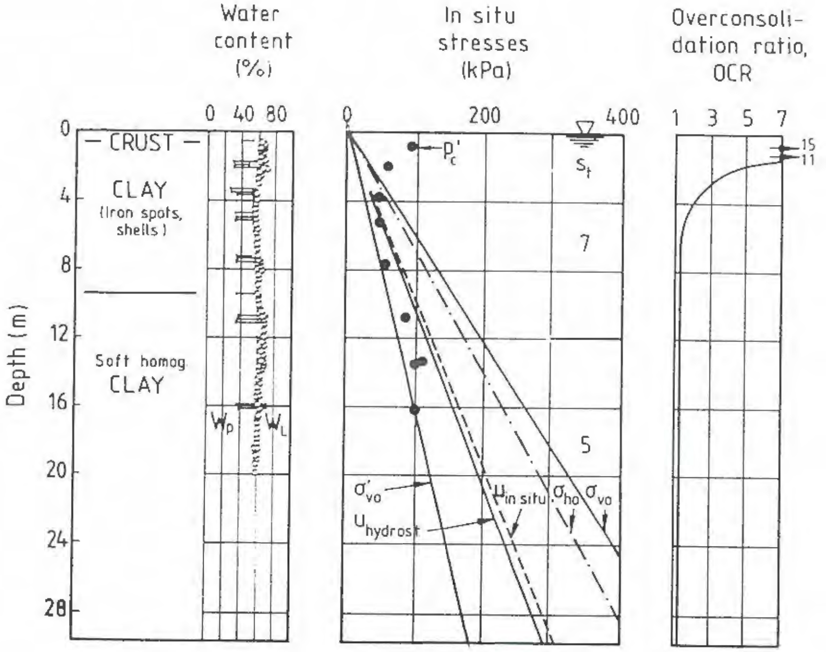


Fig. 50. Soil profile at the Onsöy test site (Lacasse and Lunne 1983).

The undrained shear strength has been determined in a number of tests. In this report, the results from direct simple shear tests and field vane tests have been used, FIG. 51.

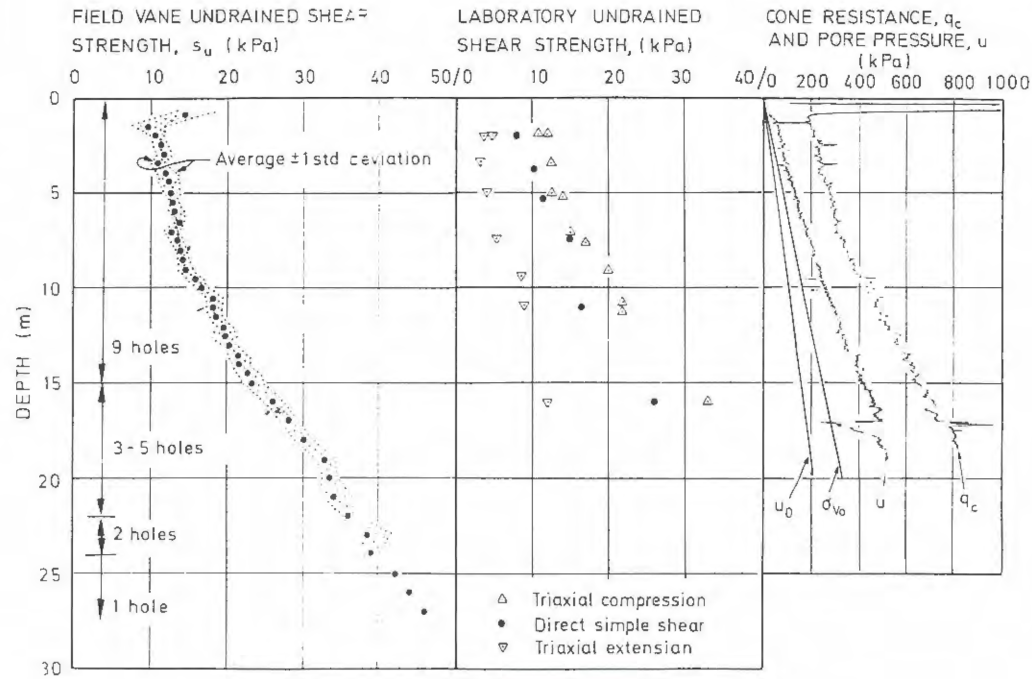


Fig. 51. Undrained shear strength at the Onsöy test site (Lacasse and Lunne 1983).

Seismic tests have been performed with the UBC seismic cone, which is very similar to the Hogentogler cone, by Eidsmoen et al (1985 a and b). Three parallel soundings were made and the results are very uniform, FIG. 52.

The measured shear modulus increases very smoothly with depth and no change in the curve can be observed at the intersection between the two main clay layers. This is in contrast to what might be expected with consideration to the changes in both water content (void ratio) and plasticity index at this point. The measured shear modulus - depth curve, however, is parallel to the expected trends in both clay layers and is situated about half-way between them. The discrepancy is therefore not very large, FIG. 53.

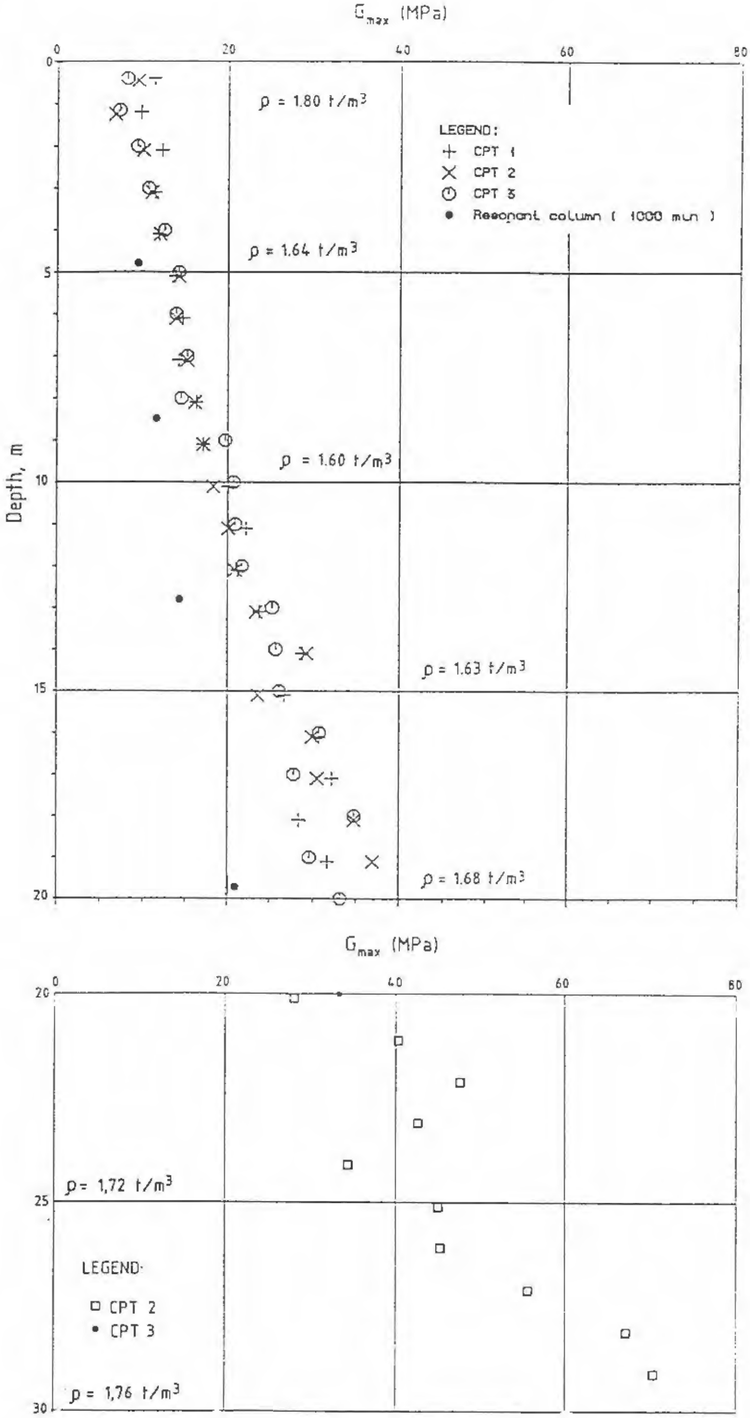


Fig. 52. Results from the seismic cone tests at the Onsöy test site (Eidsmoen et al 1985).

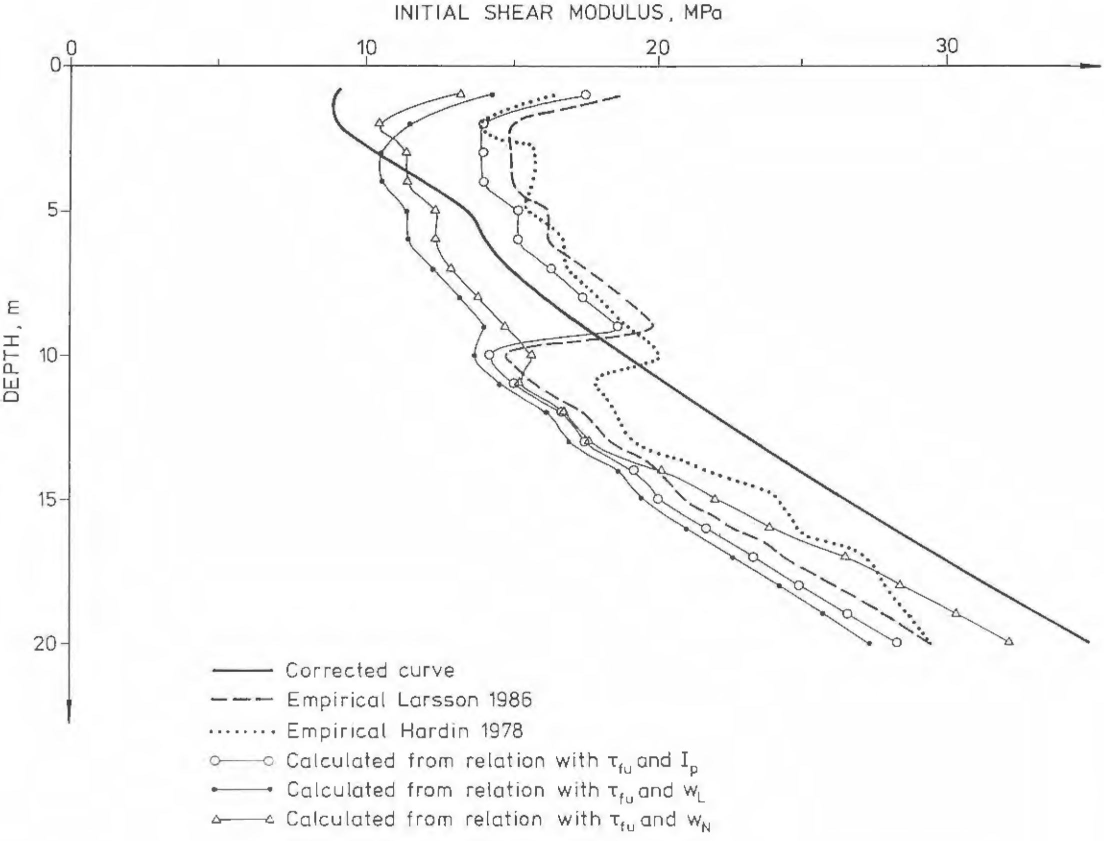


Fig. 53. Comparison between the results from the seismic cone tests at Onsöy and empirical correlations for the initial shear modulus.

• Drammen

The soil conditions in the city of Drammen and the local geology have been described by Bjerrum (1967). Many investigations have been performed since then and more detailed information on the geotechnical properties of special interest for this report can be found in e.g. Lacasse and Lunne (1983).

The actual test site is located in Museumsparken in the centre of the city. The very detailed soil investigation mentioned above was made about 50 m to the west of the present test location and another less detailed investigation was made about 20 m to the north (Eidsmoen et al 1985). The results differ slightly in terms of thicknesses of layers and undrained shear strengths and an interpolation has been made in this report.

The soil profile consists of about 2.5 m of fine sand on top of silty clay extending to 5 m depth. Below the silty clay, there is about 40 metres of marine clay; first a 5 to 6 m thick layer of plastic clay and then low-plastic (or lean) clay. The natural water contents are around 50% in the plastic clay and the plasticity index is around 28%. In the underlying low-plastic clay, the water content and plasticity index decrease to 30% and 10% respectively from 15 m depth and downwards. FIG. 54.

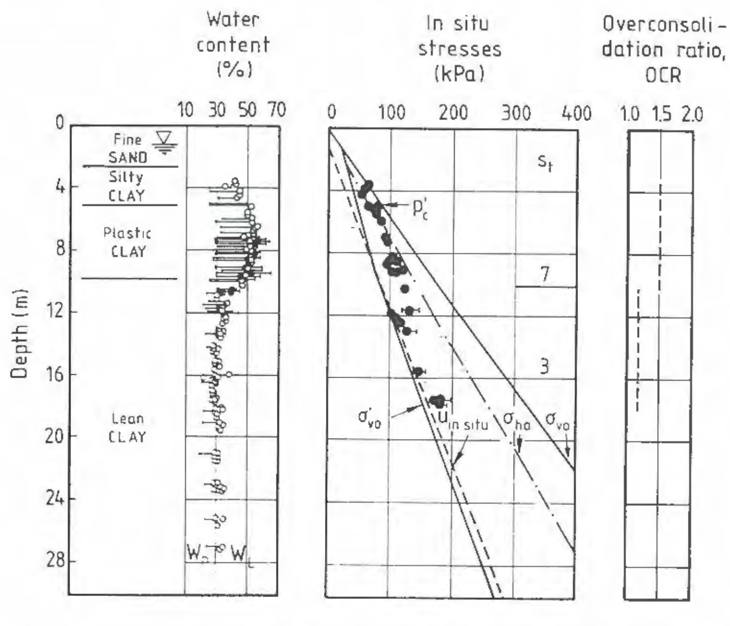


Fig. 54. Soil properties at the test site in Drammen, Danviksgate location (Lacasse and Lunne 1983).

Also in Drammen, the undrained shear strength has been determined in a large number of ways, but in this report only the results from direct simple shear tests and field vane tests are considered. FIG. 55.

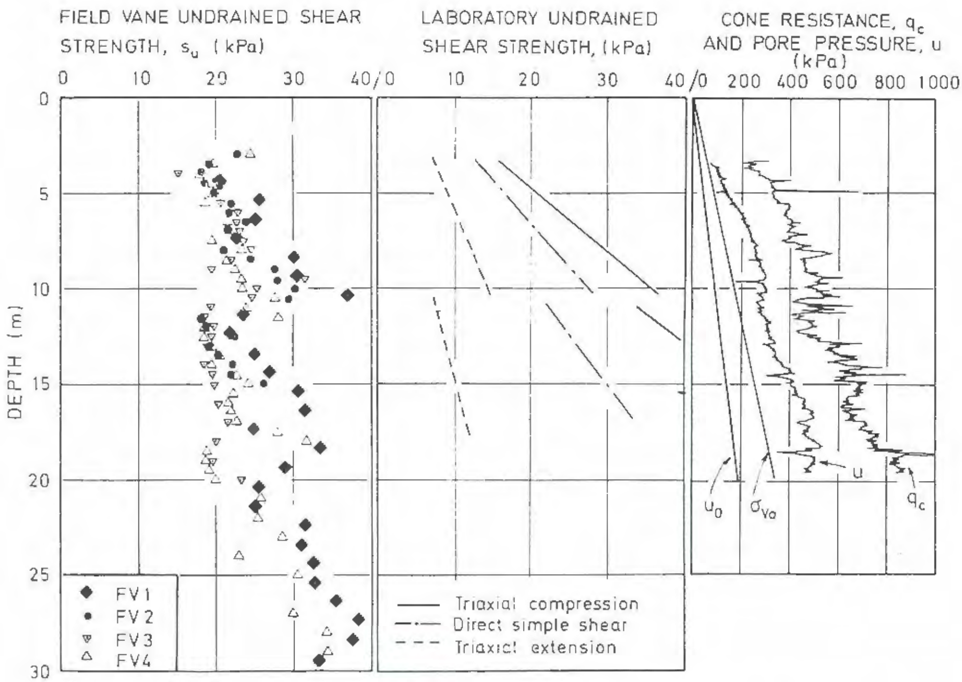


Fig. 55. Undrained shear strength at the test site in Drammen, Danviksgate location (Lacasse and Lunne 1983).

Seismic tests have been performed both by cross-hole tests and by the UBC seismic cone. The results are reported to be compatible (Eidsmoen et al 1985 a and b). The tests with the seismic cone were performed in two parallel soundings and the results were uniform, FIG. 56. For the seismic tests in Drammen, also the shear strain amplitudes in the tests were reported.

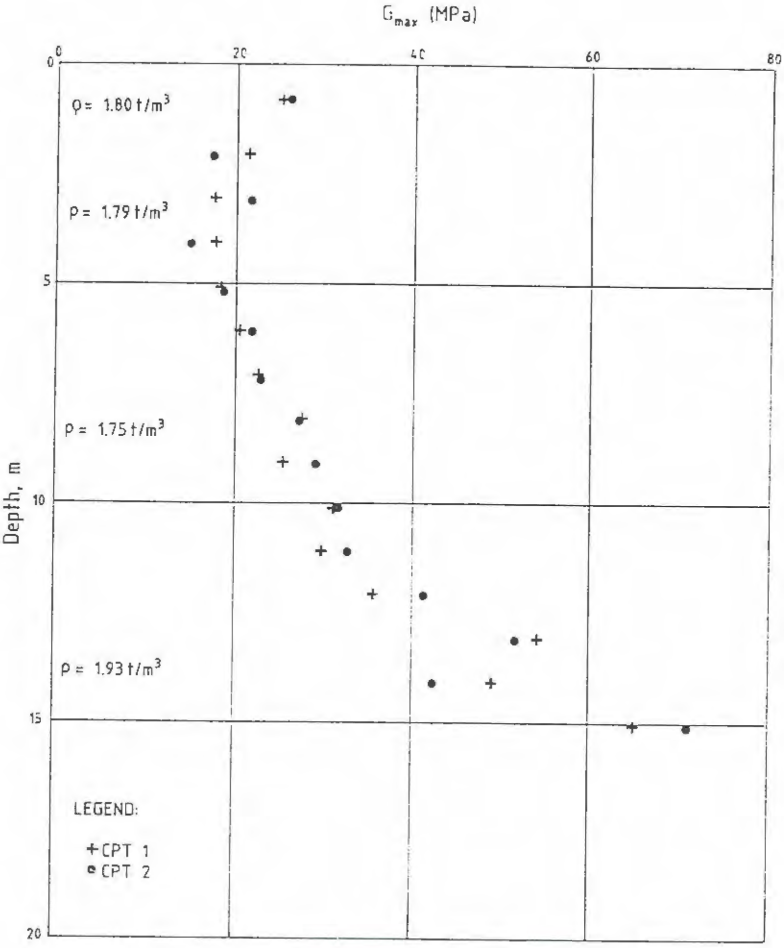


Fig. 56. Results from the seismic cone tests in Drammen (Eidsmoen et al 1985).

The measured initial modulus forms a smooth curve with depth and shows a distinct change in trend at 11-12 m depth. This change coincides with the intersection between the two main clay layers. Also in other aspects, the results are very much as could be expected from the other properties, except for the upper part of the plastic clay layer and the silty clay. The somewhat lower water content and plasticity index in this part, which empirically should give slightly higher shear moduli, cannot be observed in the measured values, FIG. 57.

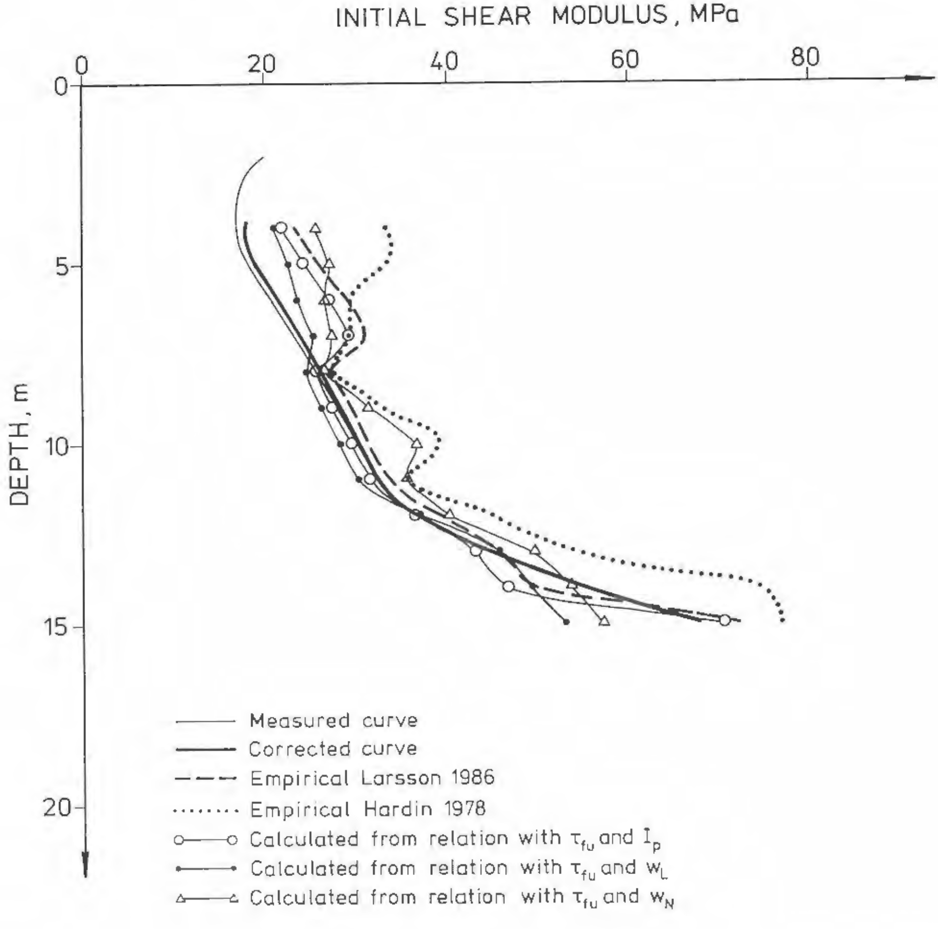


Fig. 57. Comparison between the results from the seismic cone tests in Drammen and empirical relations for the initial shear modulus.

3.3.4 NTH sites

• **Lade, Utler and Barnehagen**

The three NTH sites at Barnehagen, Lade and Utler and the seismic tests performed there are described in detail by Westerlund (1978). The investigated soils are all low-plastic silty clays with plasticity indices of about 10% and clay contents around 30%.

The investigations differ from the other sites in that only cross-hole tests have been performed in the field and no special tests have been performed to accurately determine the undrained shear strength, except for the routine investigations with the fall-cone in the laboratory.

The preconsolidation pressures, however, have been determined by oedometer tests in the laboratory supplemented by empirical estimations.

A comparison between the preconsolidation pressures, the consistency limits and the measured shear strengths has been made. It shows that some of the strength values fall outside what, from empirical experience, might be considered as possible limits. The strength values from the Lade site and one exceptionally low value at 4 m depth in the Utler site have therefore here been replaced by empirical values of $\tau_{fu} \approx 0.18 \cdot \sigma'_c$.

An accurate determination of the undrained shear strength is essential for most of the correlations used in this report. The results from the NTH sites have therefore been treated with caution and are not included in many of the evaluated correlations. They are, however, of great value also for this report, as they illustrate the point made above and also the sensitivity for some of the relations to the consistency limits in low-plastic clays.

The results from Lade are shown in FIG. 58. In this case, empirical values for the undrained shear strengths are used in the correlations based on this parameter. The results from the cross-hole tests are then in the same order as the estimates both from correlations based on shear strength and plasticity index and the correlation based on effective stress history and void ratio.

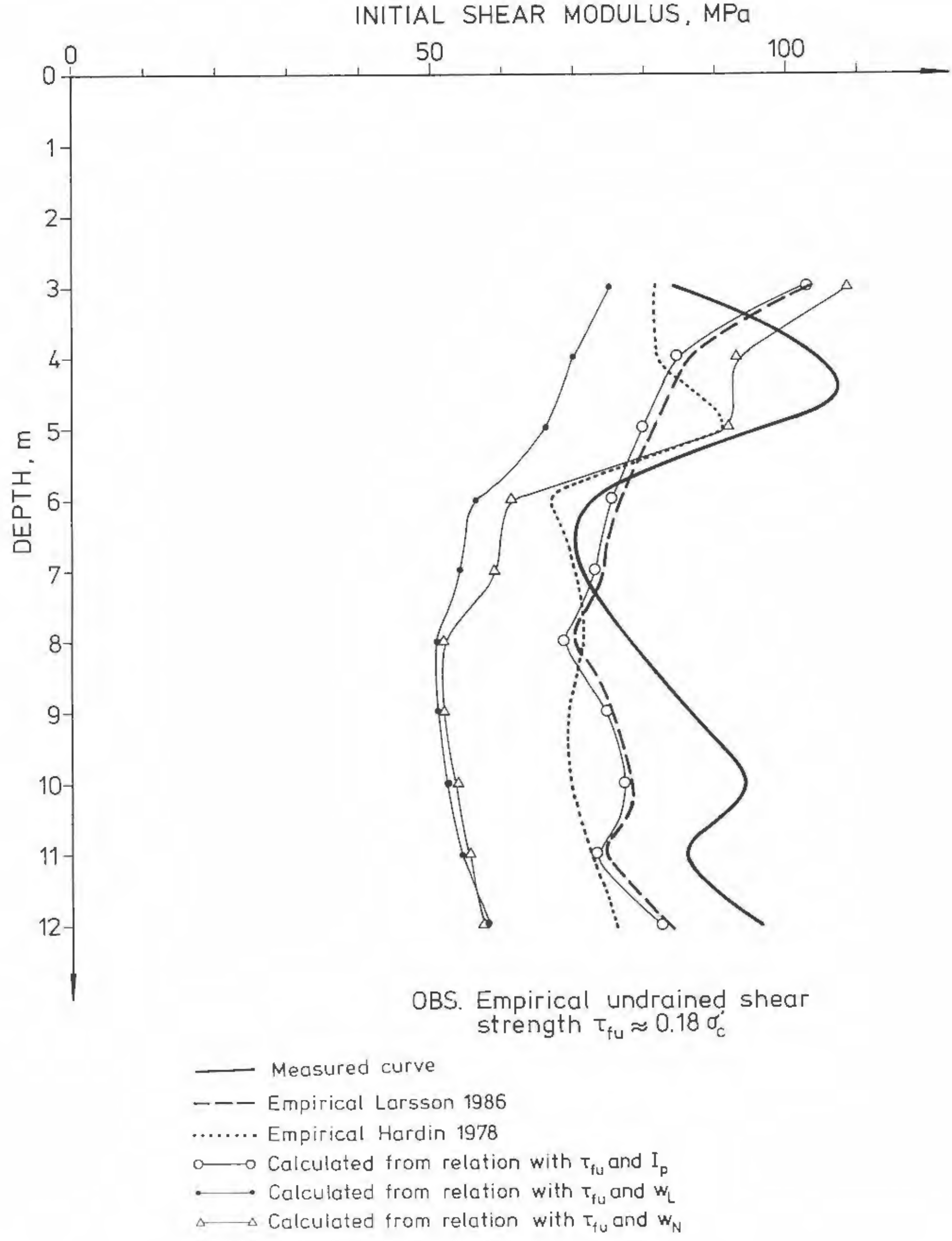


Fig. 58. Results from seismic cross-hole tests at Lade (Westerlund 1978) compared to empirical relations for the initial shear modulus.

The results from the Utler site are shown in FIG. 59. In this case, the measured shear modulus falls about half-way between the values estimated by the two main types of relations, which differ considerably from each other.

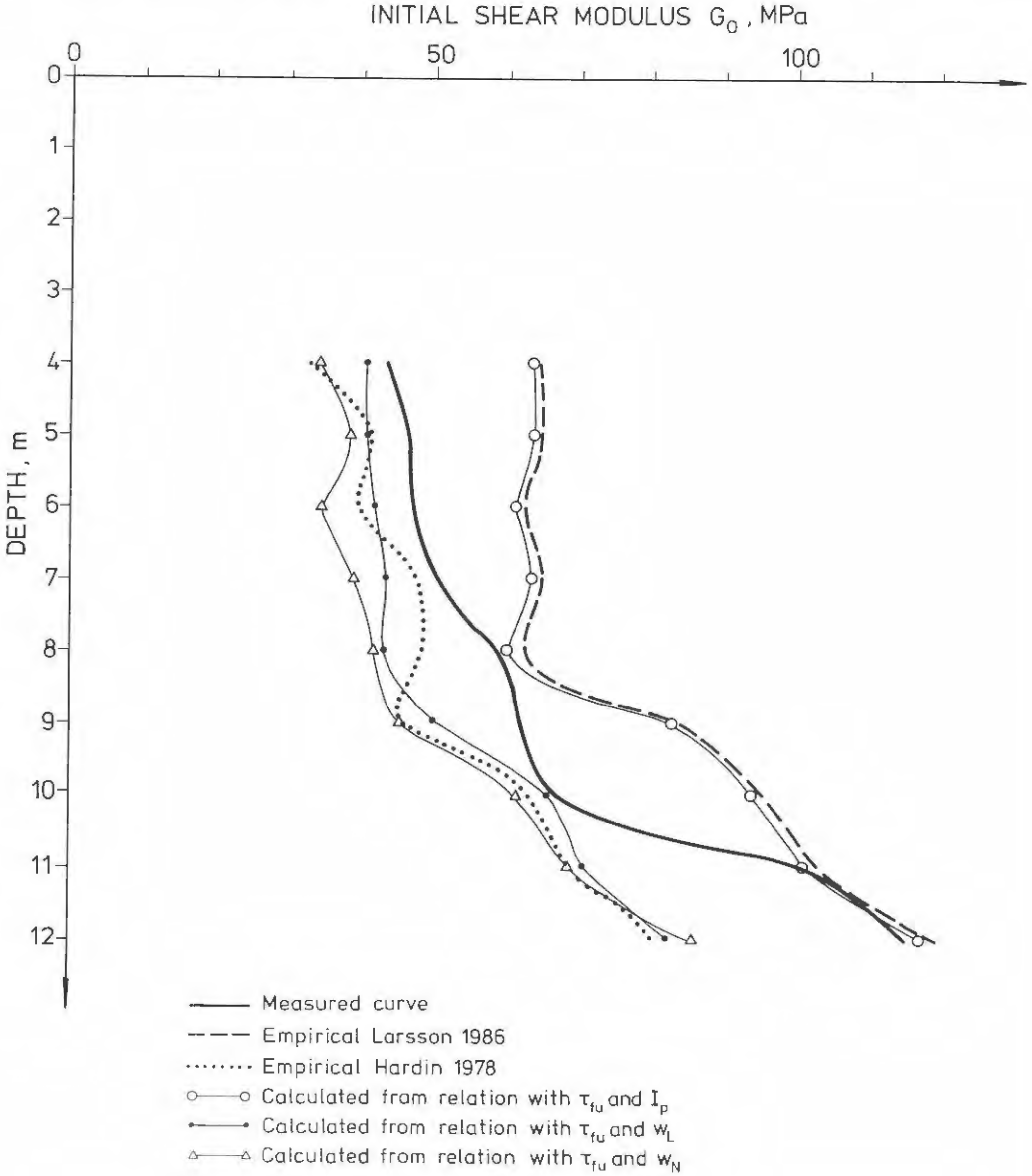


Fig. 59. Results from seismic cross-hole tests at Utler (Westerlund 1978) compared to empirical relations for the initial shear modulus.

The results from Barnehagen, on the other hand, clearly illustrate the danger in using an empirical relation based on plasticity index in a low-plastic clay. The trend of the measured shear moduli with depth directly mirrors the measured variation in undrained shear strength, but the values are only half those estimated by the empirical correlation, FIG. 60. In this case, the less sensitive correlation with the liquid limit yielded far better results. The correlation with the stress history and the void ratio yielded the best results, but the differences are significant also for this relation.

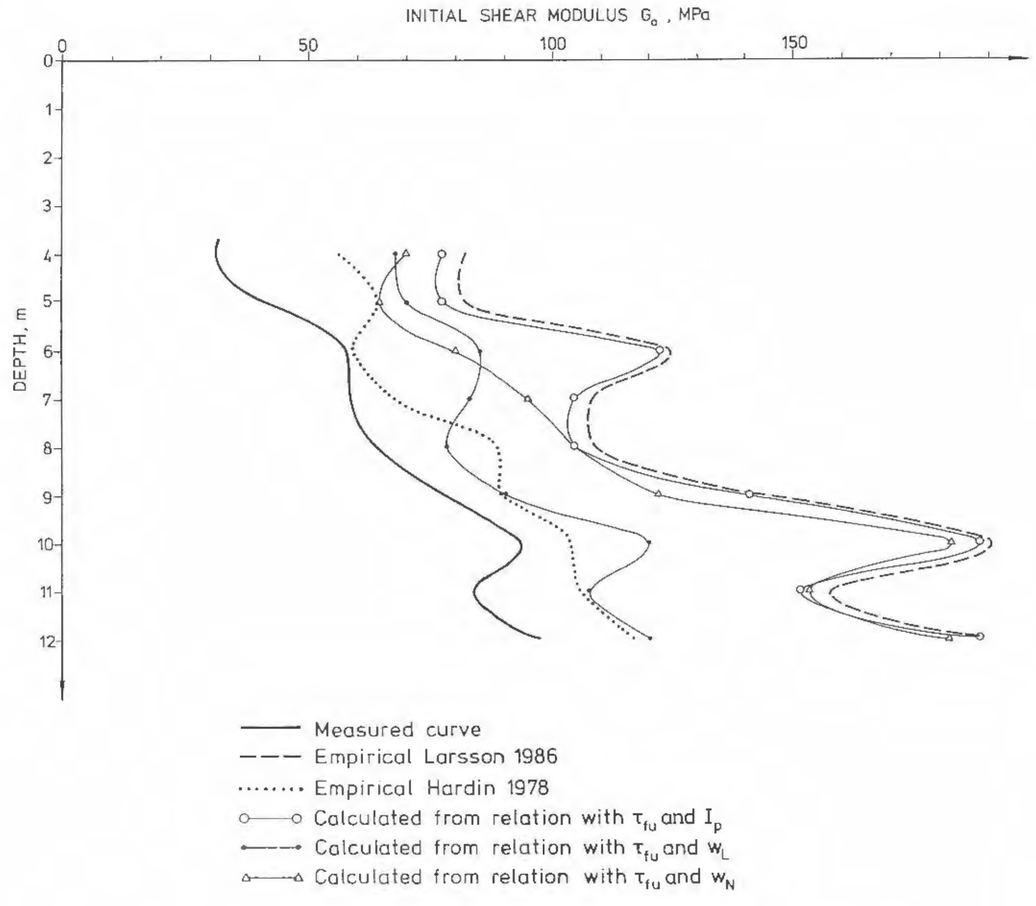


Fig. 60. Results from seismic cross-hole tests at Barnehagen (Westerlund 1978) compared to empirical relations for the initial shear modulus.

4. STUDY OF THE TEST RESULTS AND EMPIRICAL CORRELATIONS FOR THE INITIAL SHEAR MODULUS

4.1 SHEAR STRAINS IN THE TESTS

The test results are obtained by measuring the travel time for a shear wave in the soil, which is created by a manually applied hammer blow on a beam. There is normally no direct control of the energy in the blow. Measurements of the shear strain are not always made, but it is often simply assumed that the resulting shear strain in the soil is so small that it is the initial shear modulus at small strains that is calculated from the shear wave velocity.

In most clay profiles, the stiffness of the soil increases with depth. The amplitude of the shear wave gradually decreases with depth if the same energy is applied at the ground surface. These two facts normally result in a large variation of the shear strains at which the shear modulus is measured at various levels in the profiles.

In the present investigation and also in the investigation at Drammen (Eidsmoen et al 1985) the shear strains in the tests have been evaluated, FIG. 61. The shear strains can be observed to vary by several orders of magnitude in deep profiles. The curves for shear strains versus depth are mainly parallel with the largest shear strains in the profiles with the softest soils (except for the organic soils in sections 6/900 and 7/600, where the shear strains were kept as small as possible). The smallest variation was obtained for the profile at Munkedal, where the variation in shear strength with depth is moderate and the hammer blows were intentionally regulated to give a low impact energy at the higher test levels which was gradually increased with depth.

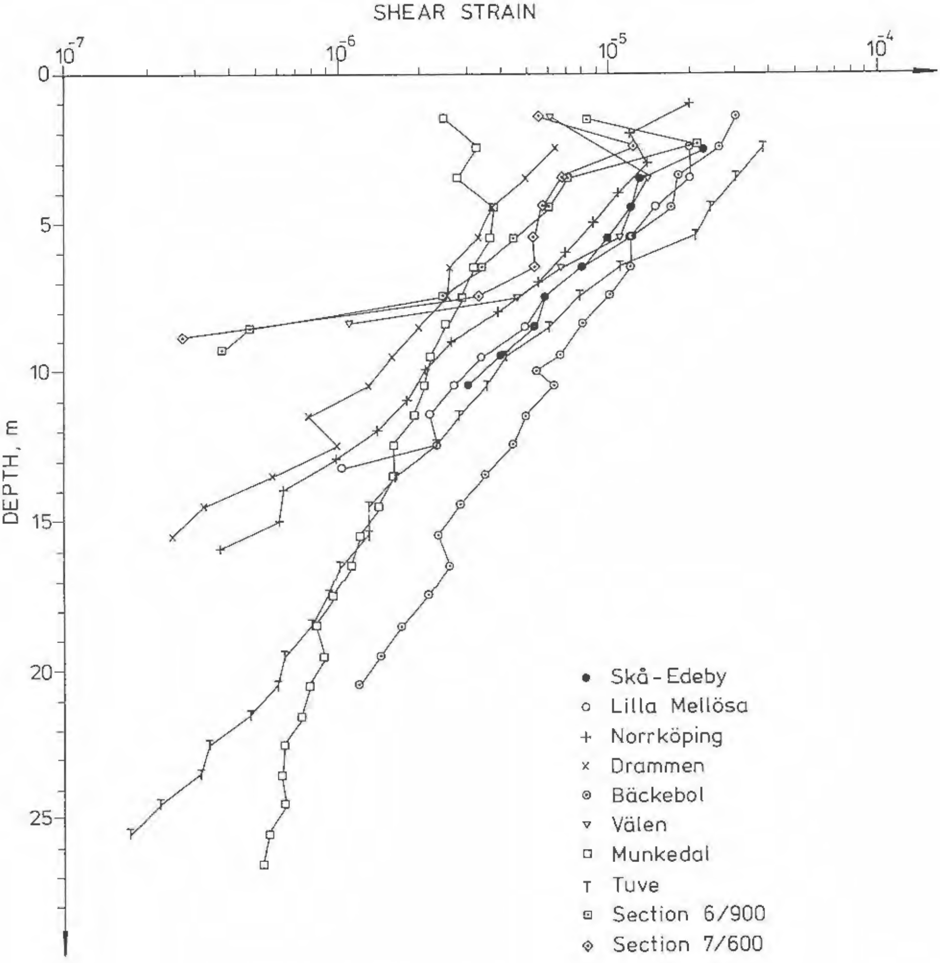


Fig. 61. Evaluated shear strains in the seismic tests at various locations.

The shear strains in the tests are in the range of $4 \cdot 10^{-5}$-$1 \cdot 10^{-7}$ and thus fall into the range of strains where the strain dependency of the shear modulus is debated. Fig. 62 shows a schematic relation between different suggested stress-strain relations obtained on the assumption that they all yield the same moduli at large strains. (It should be observed that the figure is only <u>schematic</u>. The modulus-strain relation obtained by the Hardin-Drnevich (1972) equations depends on the reference strain in the soil ($\gamma_r \approx \tau_{fu}/G_0$), and both the formula presented by Larsson (1986) and results compiled by Massarch (1981 and 1985) and Dobry and Vucetic (1987), (and Vucetic and Dobry (1990), indicate that the relation between G/G_0 and the shear strain is significantly affected by the plasticity of the soil). In the figure are sketched what might be considered the extreme limits in terms of linear-elastic ranges; the Hardin-Drnewich relation (1972) with the assumption that there is no effect on the shear modulus at strains below $1 \cdot 10^{-5}$ and the Seed and Idriss relation (1970) which assumes a continuous effect of the strain down to at least $3 \cdot 10^{-6}$. Curves similar to Seed and Idriss relation are obtained if the stress-strain relation suggested by Larsson (1986) is used. In the figure an average curve is also inserted for the span of modulus reduction with strain suggested by Det Norske Veritas (1977). This curve forms some kind of an average between the two limits. It could also be observed that this curve is very close to that obtained when the Larkin and Taylor (1979) correction is applied to the laboratory determined Hardin and Drnevich relation in order to make it compatible to field conditions.

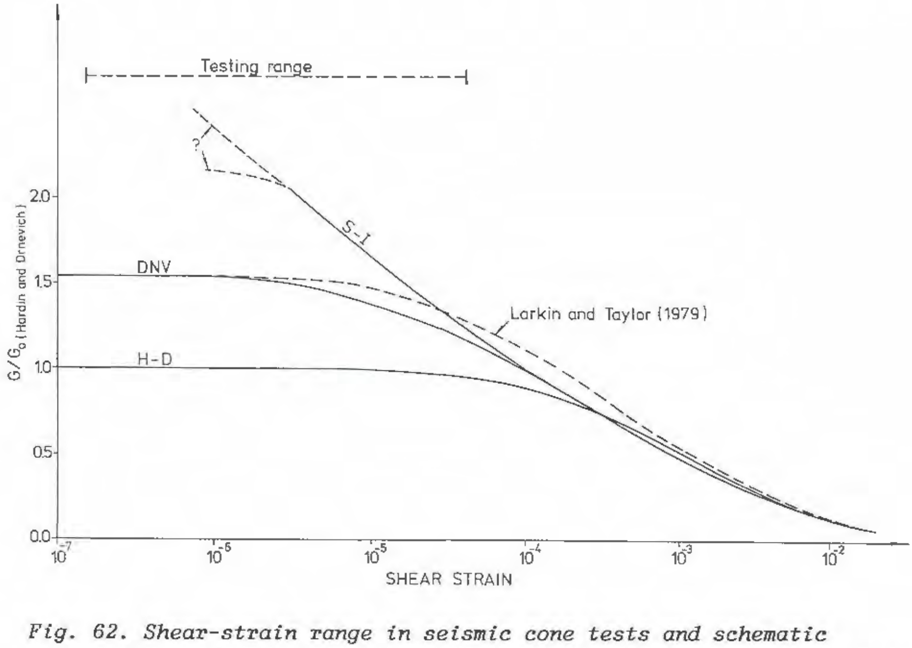


Fig. 62. Shear-strain range in seismic cone tests and schematic relation between suggested stress-strain relations under the assumption that they yield equal moduli at large strains.

Even with the Hardin and Drnewich relation, there should be some minor effects of the very largest strains in the testing range on the measured shear modulus and the Seed and Idriss curve indicates that there may be large effects of the strains for the entire testing range. If the average curve is assumed, there should be a significant effect of the strains for about half the testing range.

The measured shear moduli have been normalized versus the undrained shear strength and plotted versus the shear strain in FIG. 63 using the same method as that of Westerlund (1978) and Andréasson (1979) for the data collected by Seed and Idriss (1970) and themselves (see FIG. 8).

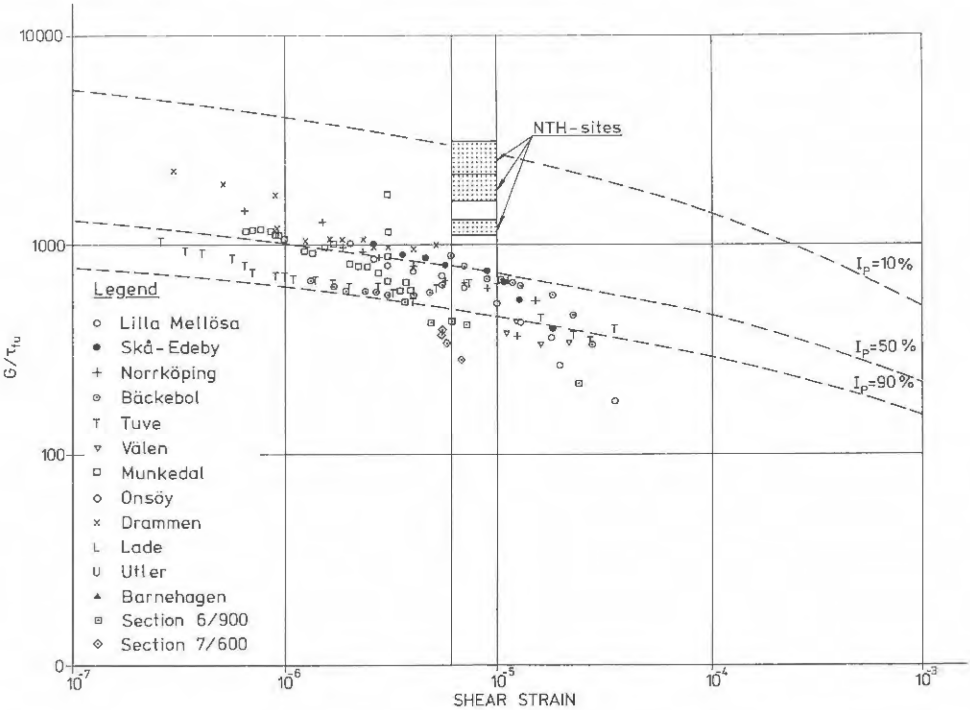


Fig. 63. Normalized values of the initial shear modulus evaluated from the seismic cone tests (and from the cross-hole tests included in the report) versus shear strains in the tests.

The plot indicates that the results are in the right order of magnitude and that there are strain effects on the measured shear moduli. However, as illustrated by the results from the three NTH sites with low-plastic clays and also from the high-plastic clayey gyttjas, this plot can be misleading. There is a significant effect of the plasticity index of the soil on the ratio between G and τ_{fu} and beside the decreasing shear strains with depth there is also normally a gradual decrease in plasticity index with depth in the clay profiles. These two effects have to be separated before any conclusions can be drawn about the influence of the shear strains on the test results.

4.2 The initial shear modulus as a function of the undrained shear strength

The initial shear modulus, as well as moduli at larger deformations, has often been expressed by a constant relation to the undrained shear strength. As shown (e.g. by Larsson 1986), this is an oversimplification because the ratio between these two parameters is strongly dependent on the plasticity index of the soil. The measured values of the initial modulus have been divided by the corresponding undrained shear strengths and are plotted in FIG. 64. The undrained shear strength is the strength measured in direct simple shear tests, corrected field vane or fall-cone tests or dilatometer tests.

As can be seen in the figure, there is a very large spread in the ratio between G_0 and τ_{fu}, from about 300 to 3000 (or an even greater spread if the organic soils are included). Furthermore, it can be observed that

- the ratio is higher the more low-plastic the clays in the profiles are.
- In most of the profiles, the ratio increases with depth as the plasticity index decreases.
- For the parts of the clay profiles at Onsöy and Drammen, where the plasticity indices are almost constant, there is very little variation in the ratio.
- At both Bäckebol and Tuve, the trend of an increasing ratio with depth is broken at the levels where there is a change in type of clay and the plasticity indices increase.

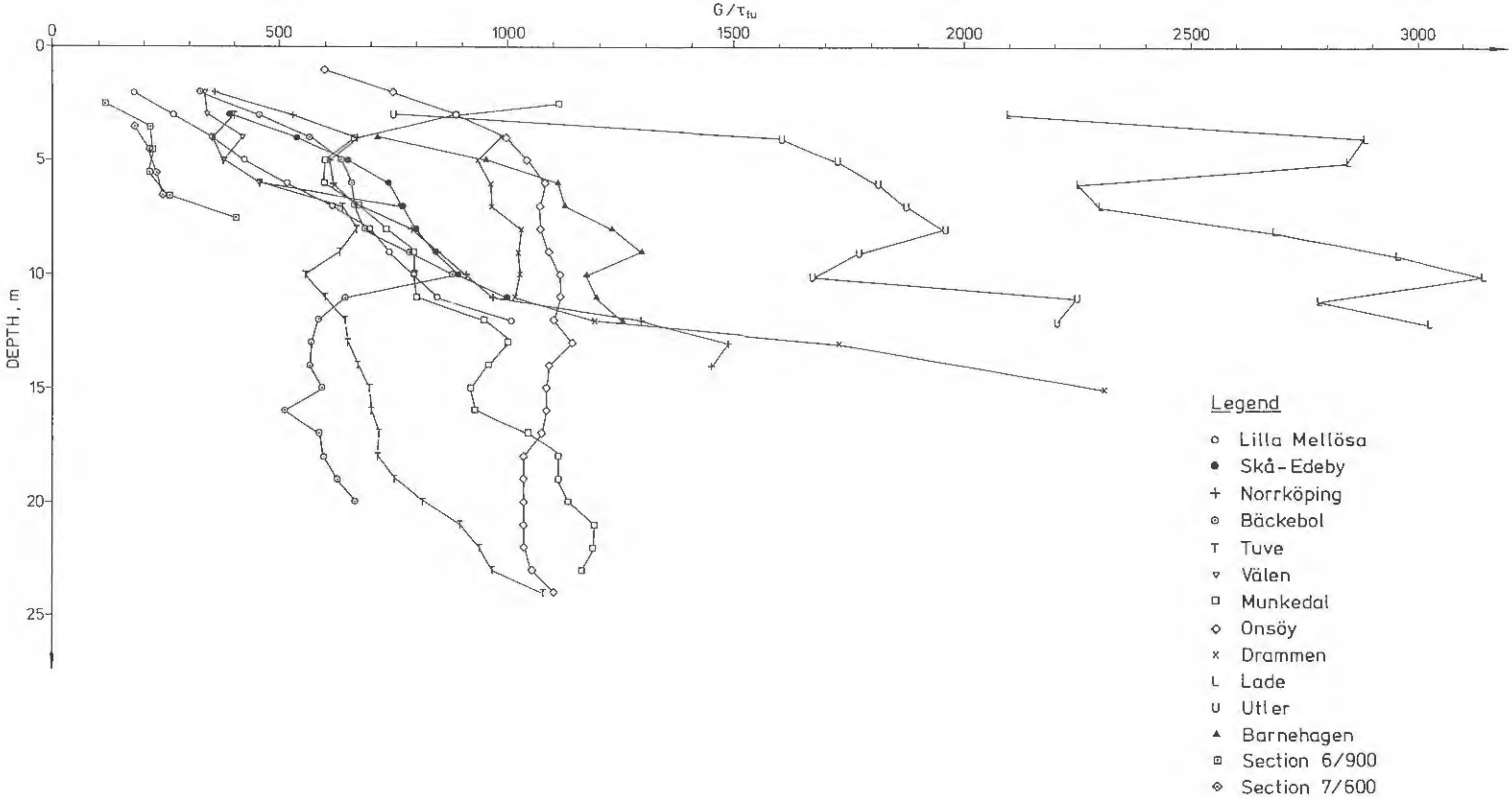


Fig. 64. Relation between the measured values of initial shear modulus and the undrained shear strength in the various soil profiles.

It thus becomes fairly obvious that the ratio between the initial shear modulus and the undrained shear strength is strongly related to the plasticity index of the soil.

(In the plot, it can also be observed that the results from the Barnehagen site are far below the results from the sites at Utler and Lade, in spite of the fact that the clays are reported to have very similar properties.)

The ratios between G_0 and τ_{fu} for the clays were then plotted versus the corresponding inverses of the plasticity indices, the liquid limits and the water contents. Relatively good and linear correlations were obtained for all the reference parameters and the simplest tentative relation was

- $$\frac{G \cdot w_L}{\tau_{fu}} \approx 500$$

The measured moduli were then normalized versus both undrained shear strength and plasticity by dividing them by the corresponding value of τ_{fu} and multiplying by w_L. The normalized moduli are plotted in FIG. 65.

As can be seen in the figure, the normalization brings the results together, even if there is still a considerable spread. Furthermore, it can be observed that, even if the normalized moduli become almost constant for the various profiles at approximately 10 m depth and below, there is still a significant trend for the normalized moduli to decrease towards the ground surface in the upper 10 metres. This trend is not very pronounced in the results from Munkedal.

The latter observations indicate two things; that there is a certain effect of the relatively large shear strains in the upper parts of the profile and also that there is no such effect in the lower parts of the profile where the shear strains are low. A comparison with the estimated shear strains in the tests indicates that the critical deformation should be in the order of $2 \cdot 10^{-6}$-$5 \cdot 10^{-6}$, which is very close to the suggestion of Larkin and Taylor (1979) and what emerges from the average of Det Norske Veritas (1977) curves (see FIG. 58).

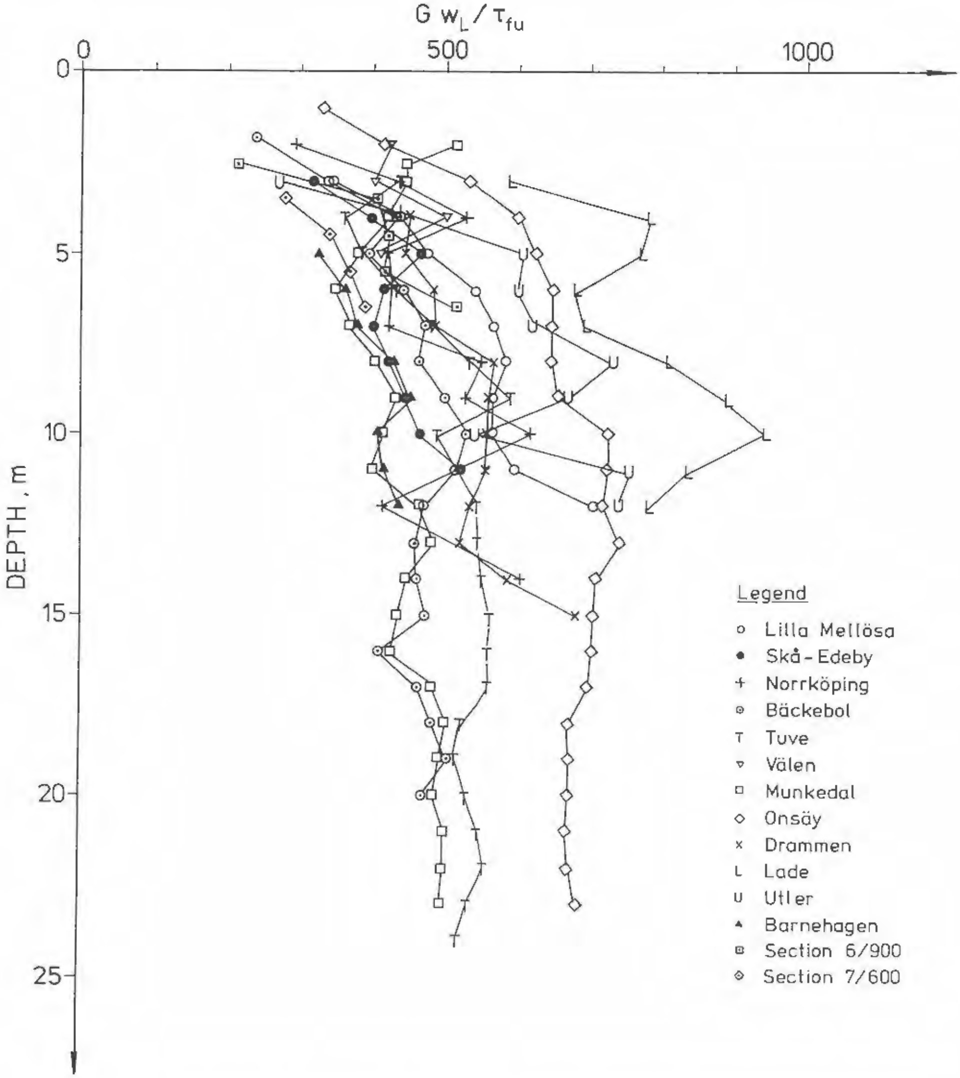


Fig. 65. Measured values of initial shear modulus normalized against undrained shear strength and liquid limit.

A check using the formula

$$\bullet \qquad A = \frac{G \cdot w_L}{\tau_{fu} \cdot \ln \dfrac{\tau_{fu}}{G \cdot \gamma}}$$

by which a correction is applied for all strains also showed that the shear modulus thereby became significantly overcorrected for strain effects at depths below 10 metres, FIG. 66.

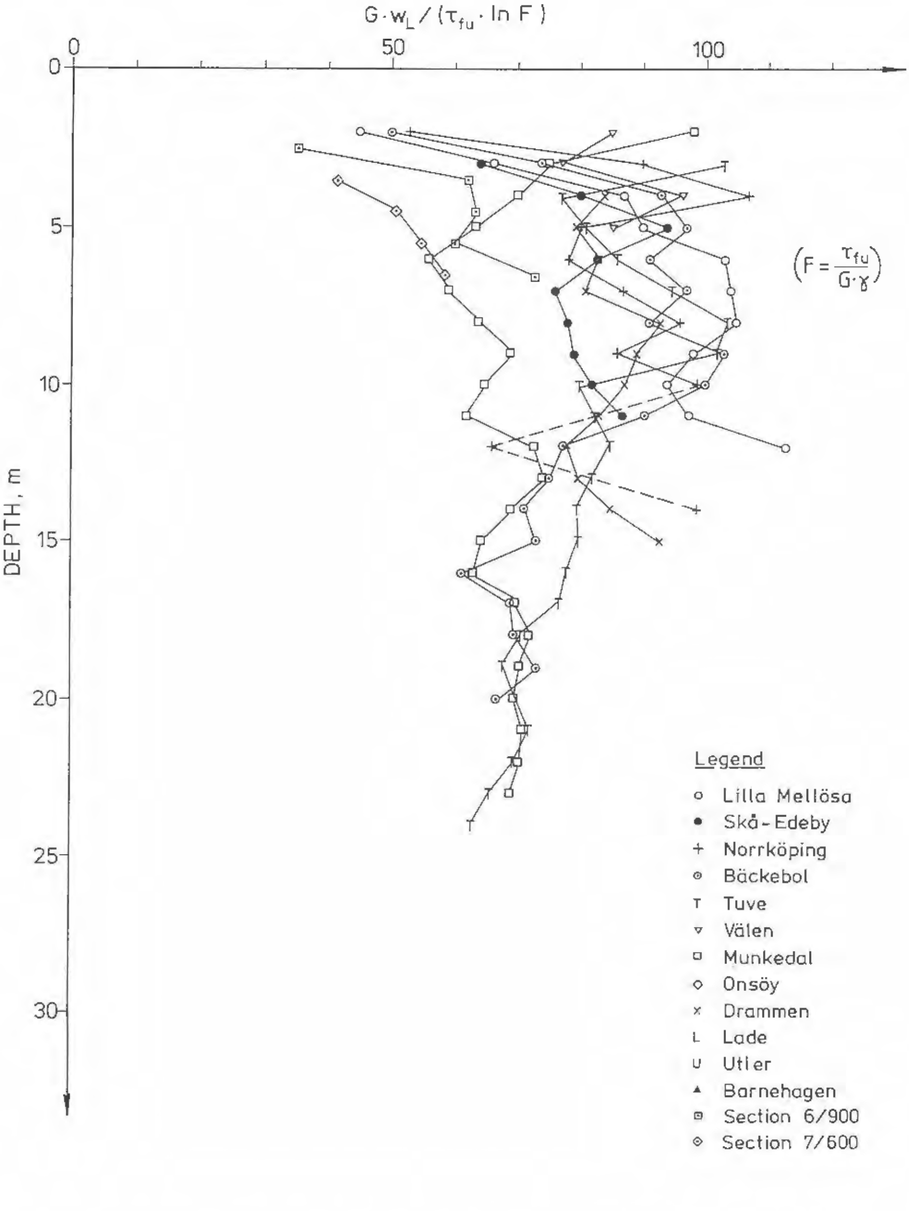


Fig. 66. Normalized measured values of initial shear modulus continuously corrected for shear strains.

The strain dependency was therefore assumed to correspond to the average of Det Norske Veritas curves. For all tests where the shear strains had been estimated to be larger than $1 \cdot 10^{-6}$, a corresponding correction factor according to FIG. 67 was applied

- $G_0 = G_{meas} \cdot$ correction factor

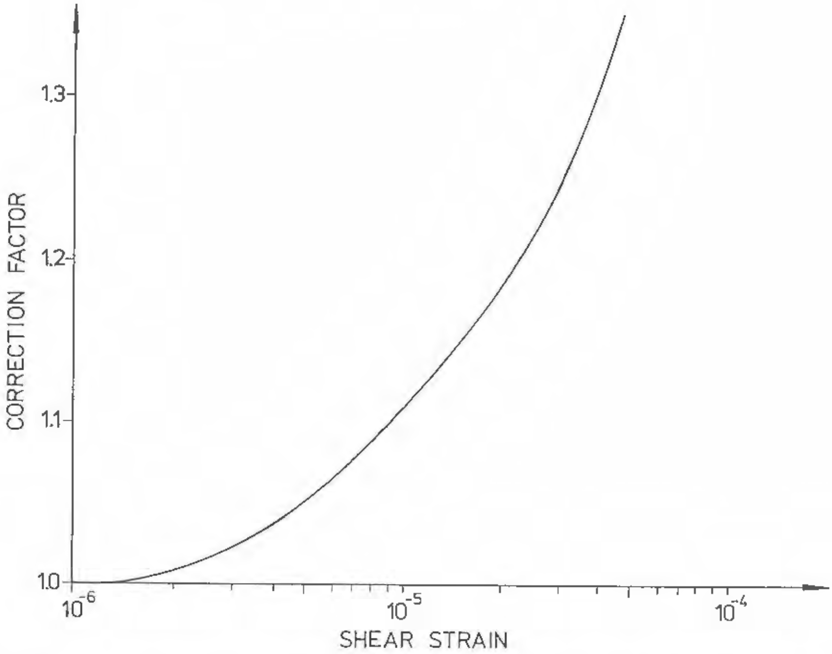


Fig. 67. Tentative correction factor for shear moduli measured at strains larger than 10^{-6}.

The application of this correction together with normalization versus undrained shear strength and liquid limit brings the results further together and the trend for changes with depth disappears, FIG. 68. A tendency for lower values above 4 metres depth can be observed in some of the profiles, but this may be associated with other factors related to the closeness to the ground surface (see Chapter 5).

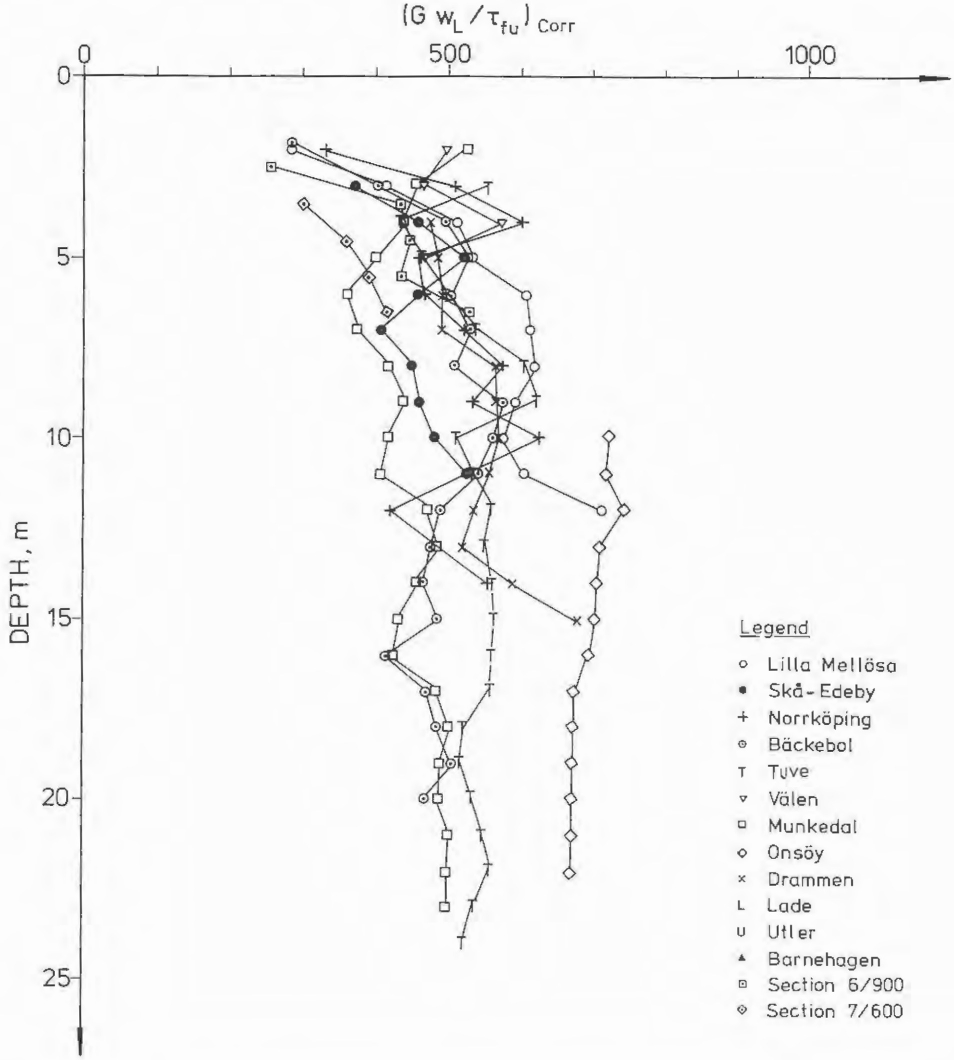


Fig. 68. Normalized measured values of initial shear modulus corrected for shear strains according to Fig. 67.

It can thus be concluded that the measured initial moduli in soft clays often have to be corrected for excessive shear strains in the tests and that this can be done with the tentatively suggested correction factors for shear strains larger than $1 \cdot 10^{-6}$. It can also be concluded that these corrected shear moduli can be normalized versus undrained shear strength and consistency limits to yield useful empirical relations.

The results obtained above also entail that, if the relation for the shear modulus suggested by Larsson (1986) is used, a lower limit should be introduced for the shear strain at which the modulus reaches its maximum and then remains constant for even smaller strains. A comparison with FIG. 62 shows that this lower strain should be about $1.5 \cdot 10^{-5}$ in order to yield maximum moduli compatible with the corrected values for the initial shear modulus. This relation for the shear modulus then becomes

- $$G = \frac{72 \cdot \tau_{fu}}{I_P} \ln \frac{\tau_{fu}}{G \cdot \gamma} \qquad \gamma \geq 1.5 \cdot 10^{-5}$$

- $$G = G_0 = \frac{72 \cdot \tau_{fu}}{I_P} \ln \frac{\tau_{fu}}{G \cdot 1.5 \cdot 10^{-5}} \qquad \gamma \leq 1.5 \cdot 10^{-5}$$

4.3 EMPIRICAL CORRELATIONS

The ratios between the corrected initial shear modulus and the undrained shear strength for the clays have been plotted versus the inverses of the corresponding plasticity indices, liquid limits and natural water contents. The best relation, when only the results from the SGI and NGI sites were considered, was obtained for the plasticity index. According to this relation, the initial shear modulus can be expressed by

$$\boxed{G_0 = (208/I_P + 250) \cdot \tau_{fu}}$$

This relation is very similar to the relation previously suggested by Larsson (1986) on the basis of results from tests with large strains and empirical experience, provided that the limiting strain of $1{,}5 \cdot 10^{-5}$ is introduced. The two relations are shown together in FIG. 69.

Correlations for the initial shear modulus of

$$\boxed{G_0 \approx 504 \ \tau_{fu}/w_L}$$

and

$$G_0 \approx 541 \ \tau_{fu}/w_N$$

were also found, but with somewhat larger spread.

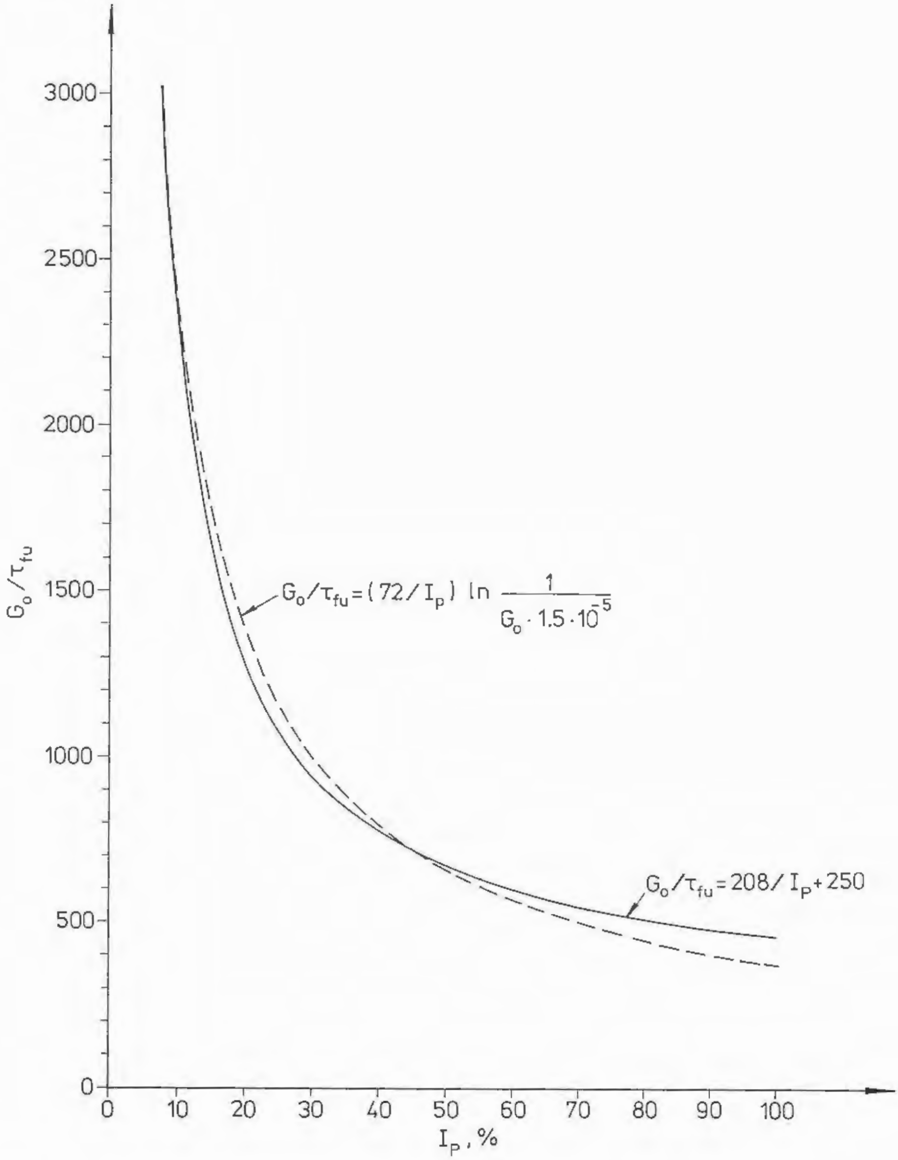


Fig. 69. Comparison between the best obtained relation for the initial shear modulus and the relation suggested by Larsson (1986) after introduction of a limiting shear strain.

The corrected measured moduli for the clays have then been plotted versus the corresponding values obtained by the various empirical relations. This has been done in order to illustrate the applicability of the relations to various soils and the accuracy that can be expected when using them. The empirical relations are

- $G_0 = 625 \cdot OCR^{k'} \cdot (\sigma' \cdot p_a)^{0.5} / (0.3 + 0.7\ e^2)$ (Hardin 1978)
- $G_0 = (72/I_P) \cdot \tau_{fu} \cdot \ln \dfrac{\tau_{fu}}{G_0 \cdot 1.5 \cdot 10^{-5}}$ (Modified Larsson 1986)
- $G_0 = (208/I_P + 250) \cdot \tau_{fu}$
- $G_0 = 504\ \tau_{fu}/w_L$
- $G_0 = 541\ \tau_{fu}/w_N$

As before, the undrained shear strengths are the strengths obtained in direct simple shear tests, corrected field vane or fall-cone tests and dilatometer tests. Most of the parameters required for Hardin's (1978) relation have been determined in the test fields and can also be estimated from the results of the dilatometer tests.

The various plots are shown in FIGS. 70-74.

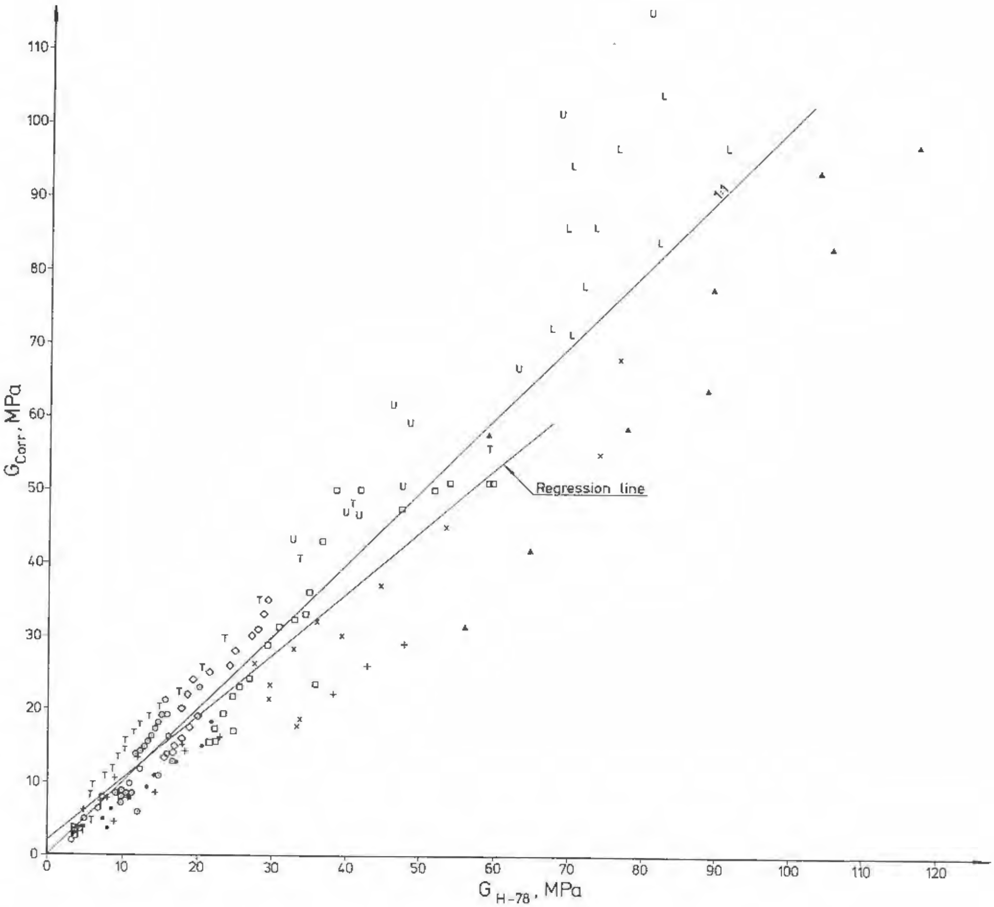


Fig. 70. Comparison between corrected initial shear moduli measured by seismic cone tests (and by cross-hole tests) and values obtained by the empirical relation suggested by Hardin (1978).

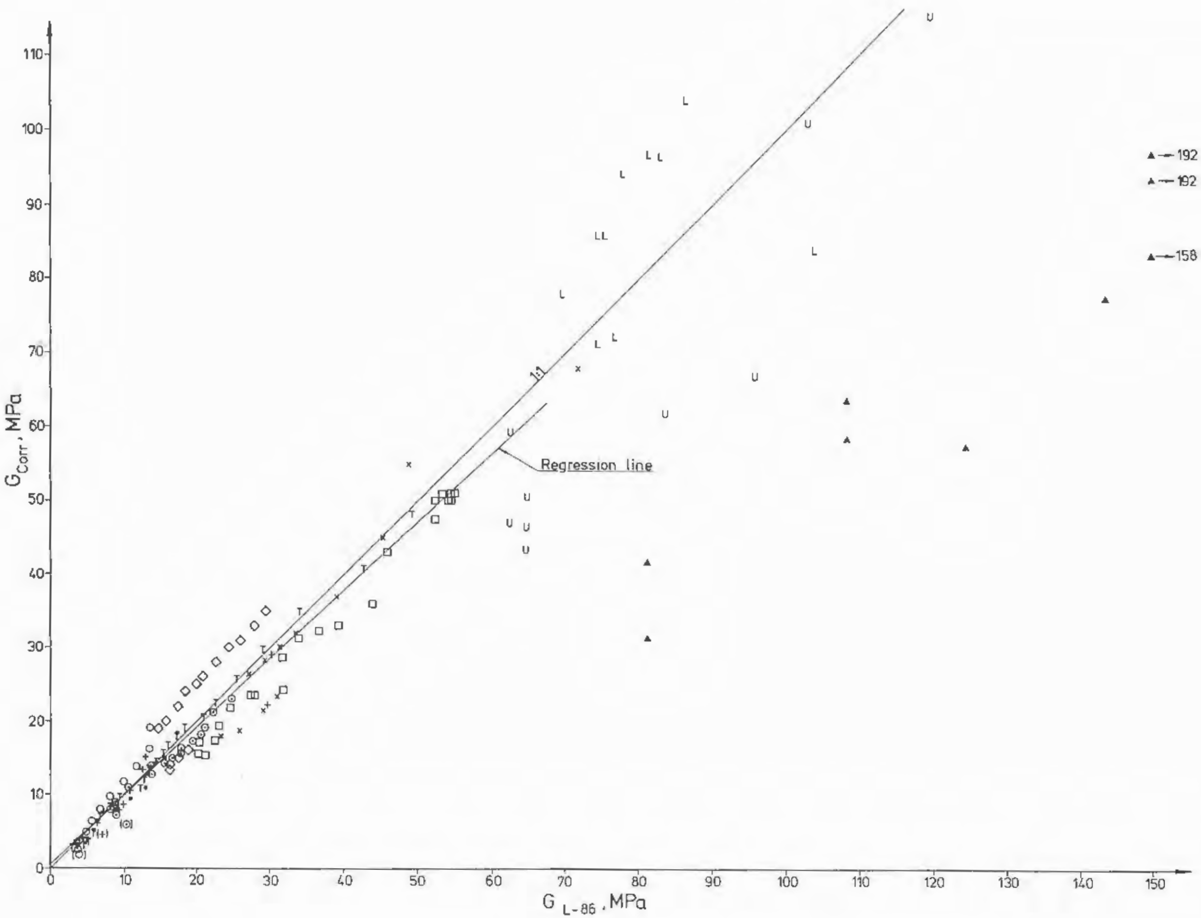


Fig. 71. Comparison between corrected initial shear moduli measured by seismic cone tests (and by cross-hole tests) and values obtained by the empirical relation suggested by Larsson (1986)(after introduction of a limiting shear strain).

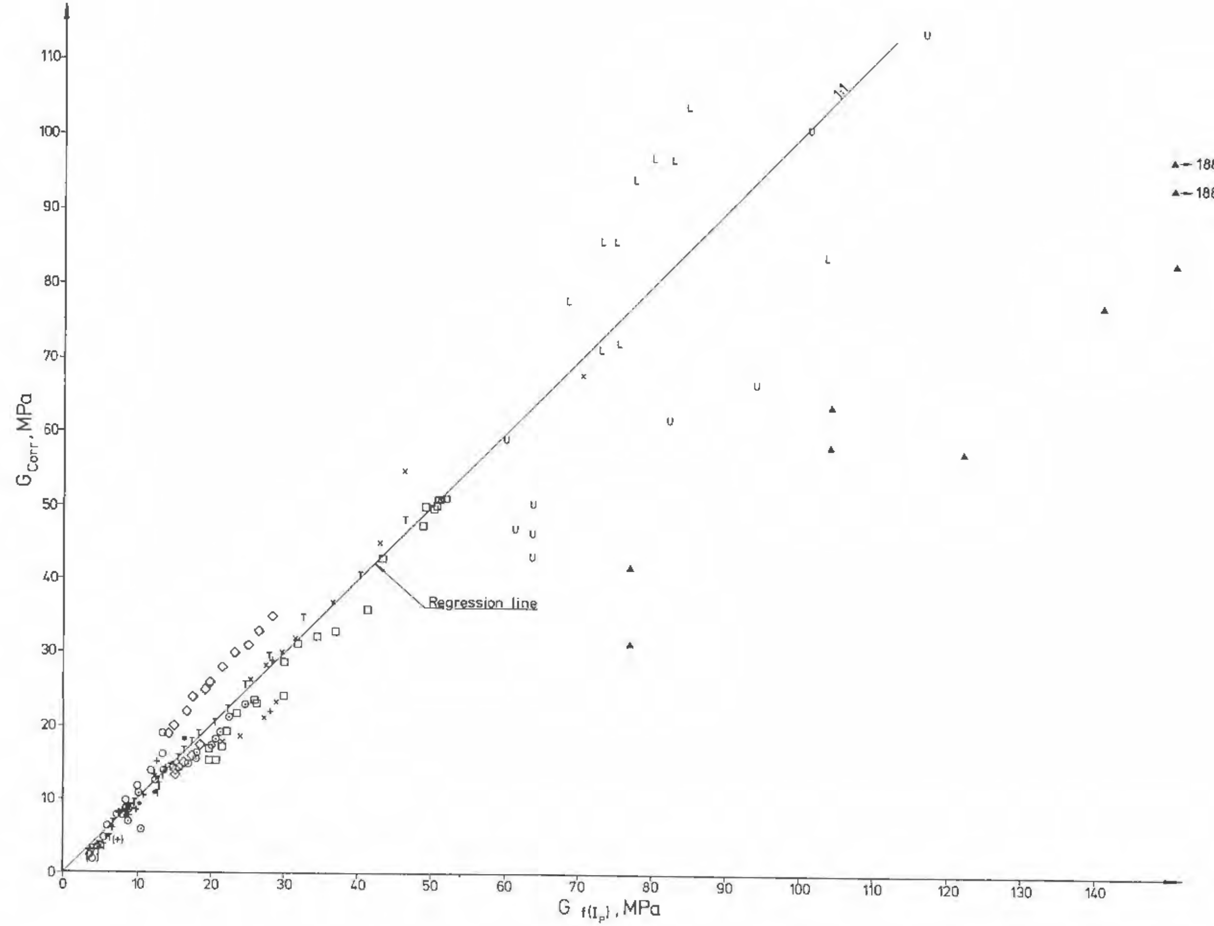


Fig. 72. Comparison between corrected initial shear moduli measured by seismic cone tests (and by cross-hole tests) and values obtained by the empirical relation between G_0, τ_{fu} and I_P.

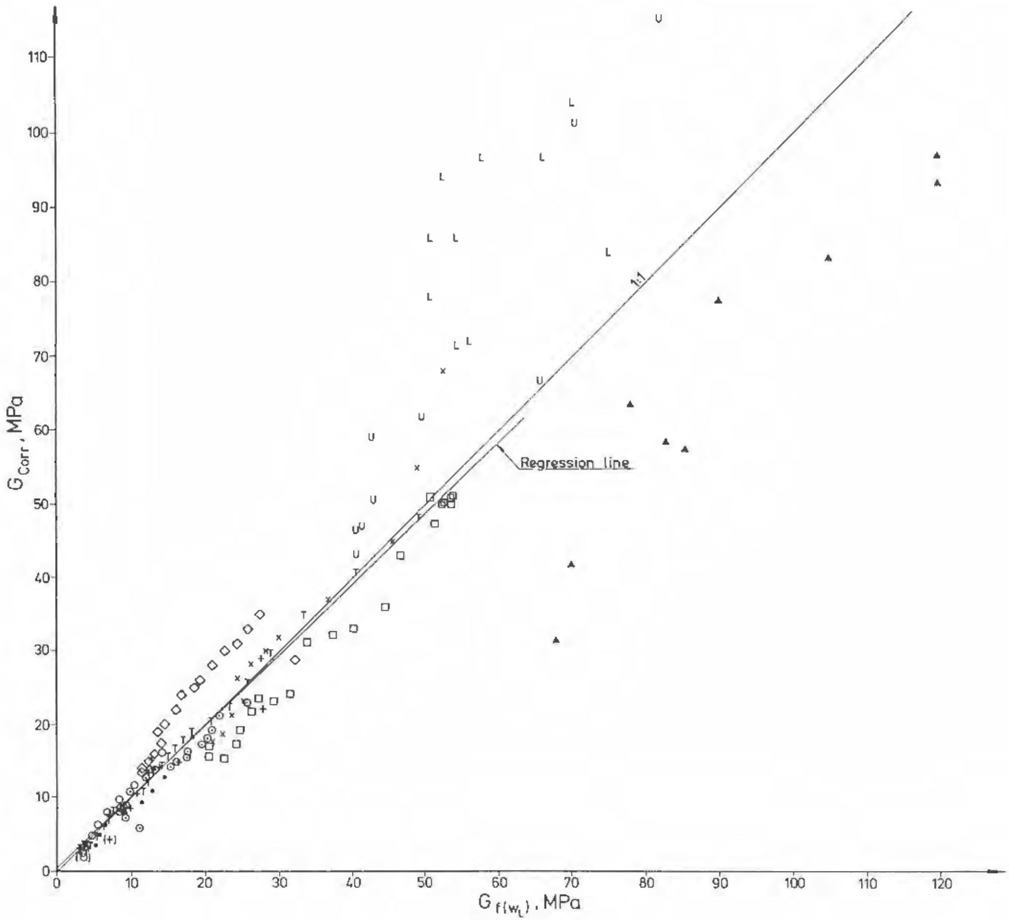


Fig. 73. Comparison between corrected initial shear moduli measured by seismic cone tests (and by cross-hole tests) and values obtained by the empirical relation between G_0, τ_{fu} and w_L.

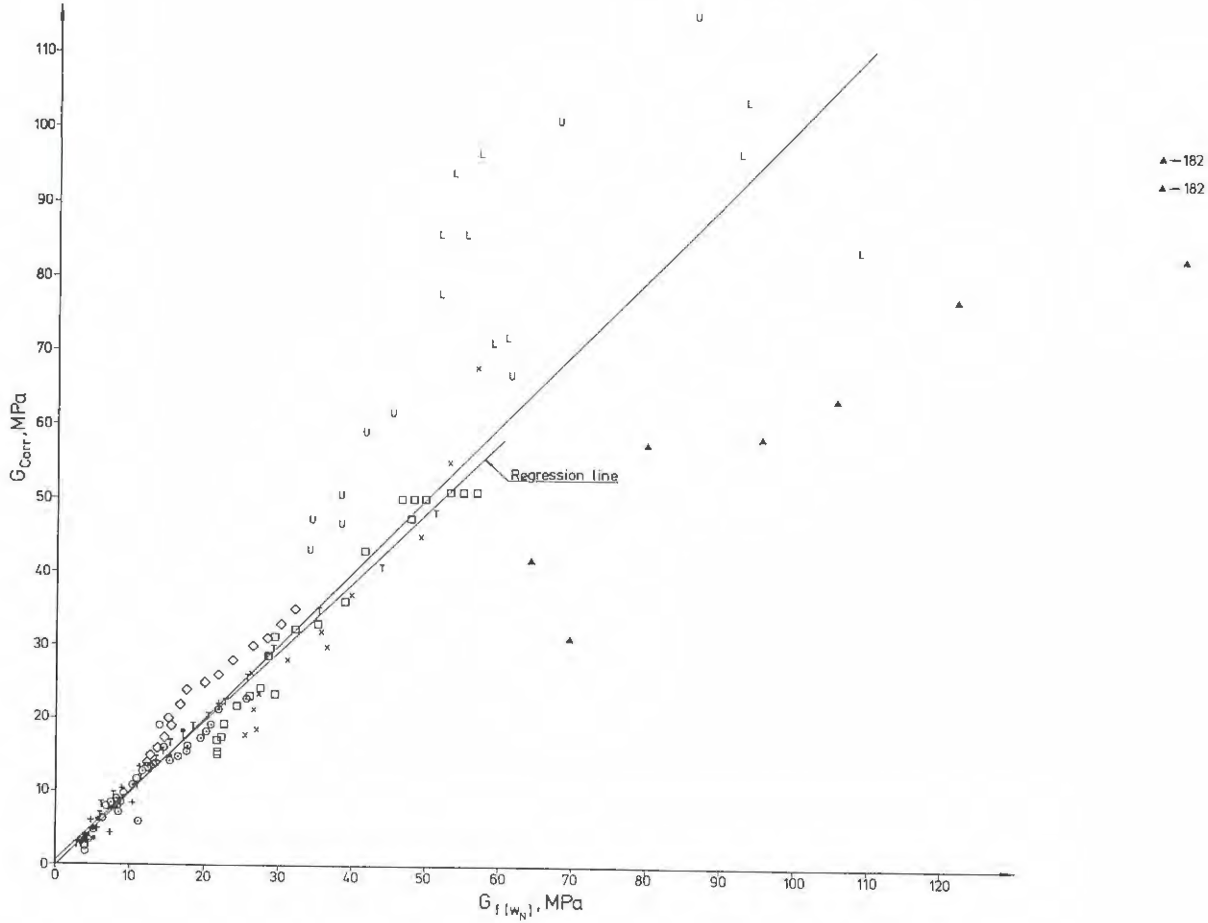


Fig. 74. Comparison between corrected initial shear moduli measured by seismic cone tests (and by cross-hole tests) and values obtained by the empirical relation between G_0, τ_{fu} and w_N.

Linear regression analyses for the data from the SGI and NGI sites give the following correlations

Relation (Units in kPa)	Coefficient of correlation
$G_{corr} = 0.8442\ G_{(H-78)} + 2203$	0.9263
$G_{corr} = 0.9258\ G_{(L-86)} + 732$	0.9766
$G_{corr} = G_{(I_P)} + 86$	0.9776
$G_{corr} = G_{(w_L)} + 779$	0.9655
$G_{corr} = G_{(w_N)} + 758$	0.9730

In these mainly high and medium-plastic clays, the relations based on the undrained shear strengths thus gave superior correlations and especially those also related to the plasticity index.

When the two NTH sites at Utler and Lade with low-plastic clays are included in the regression analyses, the correlations change to

Relation (Units in kPa)	Coefficient of correlation
$G_{corr} = 1.1020\ G_{(H-78)} - 1945$	0.9500
$G_{corr} = 0.8441\ G_{(L-86)} + 2031$	0.9605
$G_{corr} = 0.8858\ G_{(I_P)} + 1839$	0.9728
$G_{corr} = 1.2436\ G_{(w_L)} - 2980$	0.9663
$G_{corr} = 1.1568\ G_{(w_N)} - 1303$	0.9674

The inclusion of low-plastic clays with somewhat uncertain values of undrained shear strengths thus brought a general deterioration of the correlations for the relations based on undrained shear strengths and consistency limits, whereas the Hardin relation, which is insensitive to both parameters, actually became a somewhat improved correlation.

When the third NTH site at Barnehagen, which in relation to the reported plasticity indices yielded the most diverging relations between G_0 and τ_{fu}, is included, the correlations become

Relation (Units in kPa)	Coefficient of correlation
$G_{corr} = 0.9727\ G_{(H-78)} + 520$	0.9388
$G_{corr} = 0.6340\ G_{(L-86)} + 6837$	0.9127
$G_{corr} = 0.6570\ G_{(I_P)} + 6925$	0.9187
$G_{corr} = 1.0057\ G_{(w_L)} + 1433$	0.9309
$G_{corr} = 0.7641\ G_{(w_N)} + 6867$	0.8867

The results clearly show that the relations based on the plasticity index become too sensitive to be used with confidence in low-plastic clays. If a relation between G and τ_{fu} is to be used at all in this type of soil, it is better to use the relation based on the liquid limit, which is much less sensitive to possible errors in the consistency limits.

The Hardin relation appears to give the best correlation for low-plastic clays, even if a more elaborate determination of the undrained shear strength might have improved the correlation for the other relations.

The relatively good correlation obtained for the relation based on undrained shear strength and natural water content in high and medium-plastic clays is believed to be a result of the fact that, for most of the clays included in the investigation, the natural water content is roughly equal to the liquid limit. No such correlation is believed to exist from a fundamental point of view and the last regression analyses also support this belief.

The clayey organic soils (clayey gyttjas) have not been included in the basis for the correlations as these have been intended for clays. The results from the clayey gyttjas and also from the highly organic clay at Välen, however, show that when the organic content exceeds 6 %, and the soil according to Swedish classification is an organic soil, it is better to use the empirical relation based on undrained shear strength and liquid limit or the Hardin relation.

4.4 Recommendations

If empirical relations are to be used to estimate the initial shear modulus in clays, it is advisable to use

- $$G_0 \approx (208/I_P + 250) \cdot \tau_{fu}$$

(or alternatively $G_0 = (72/I_P) \cdot \tau_{fu} \cdot \ln \dfrac{\tau_{fu}}{G_0 \cdot 1.5 \cdot 10^{-5}}$)

for <u>high and medium-plastic clays</u> and

- $$G_0 \approx 625 \cdot OCR^{k'} \cdot (\sigma' p_a)^{0.5} / (0.3 + 0.7\, e^2)$$

(or alternatively $G_0 \approx 504\, \tau_{fu}/w_L$)

for <u>low-plastic clays and for clayey gyttjas</u>.

It should be observed that the relations above should only be used together with undrained shear strengths determined in direct simple shear tests, corrected field vane or fall-cone tests, dilatometer tests evaluated according to Larsson (1989) or some other type of test that gives directly compatible results.

5. DISCUSSION

Sources of errors in the tests and in the correlations

The measured shear moduli at shallow depths often appear to be low in relation to the other soil properties. There are several possible reasons for this.

This is the part of the soil profile where there may be some influence of the slightly non-vertical travel path of the shear wave. When the distance between the beam and the sounding point is kept small, the possible error is also small. Another possible error associated with the test procedure is that there may be excessive disturbance in the weak soil layers directly below the crust. That there should be any errors specifically related to the seismic cone method is, however, contradicted by comparisons with cross-hole tests which yield compatible results also at these levels.

The low values of modulus may also be connected with special properties of the soil itself. In this part of the profile, there are often root threads and various weathering effects because of the proximity to the ground surface. Microfissures and other inhomogeneity may also occur . All these factors may result in lower shear moduli.

The soil in this part of the profile is also overconsolidated to some degree because of ageing and fluctuating ground water levels. This affects the overconsolidation ratio and the undrained shear strength, but it is not quite certain whether it also affects the shear moduli in exactly the same way as if these properties had been created only by vertical loading and time effects.

The most likely explanations for the low measured shear moduli at shallow depths appear to be related to the special properties of the soil close to the ground surface and the measured shear moduli may therefore be assumed to correspond to the actual soil properties. This, on the other hand, entails that the empirical relations for the shear modulus are less accurate for this particular part of the soil profile, where they generally yield somewhat overpredicted moduli.

It has been shown that the results from seismic cone tests in soft clays are sensitive to the strain level in the tests. A tentative correction has been suggested, but this is uncertain and should only be used in those cases where the strain level for practical reasons cannot be kept under $1 \cdot 10^{-6}$. This can often be achieved by restricting the hammer blows in the upper part of the profiles and gradually increasing the impact energy with depth. In very soft clays, this procedure may not keep the strains below the critical limit although it helps to reduce the required correction.

There is a significant scatter in all the empirical relations and this has to be expected. In the seismic test itself, it is not the shear modulus but the travel time of the shear wave for a certain distance that is measured. The shear modulus is then calculated from the estimated velocity, $G = \rho \cdot V_s^2$. This means that all errors in depth recordings and time measurements are incorporated and magnified and also minor errors in the bulk density are incorporated. A certain spread therefore always exists in the "measured" shear moduli.

The empirical relations cannot be more accurate than the parameters on which they are based. The Hardin (1978) relation is based on not less than five separate soil parameters, OCR, I_p, σ'_v, K_0 and e. This would restrict its practical usefulness but for the fact that the key parameter is the void ratio and the other parameters have much less influence on the results. A considerable scatter, however, must be expected.

The other relations are based on only two parameters; the undrained shear strength and the plasticity index or alternatively the liquid limit. The undrained shear strength, however, is not a unique concept. Different values are obtained depending on what type of test is applied. In this report, the shear strength tests obtained in direct simple shear tests, corrected field vane and fall-cone tests and dilatometer tests are used. These are also the shear strengths that should be used if the presented empirical relations are to be applied.

The direct simple shear test normally gives results that are directly compatible to the corrected field vane test (Larsson et al 1984). The fall-cone test is calibrated against the field vane test (Hansbo 1957), and the evaluation of the dilatometer test is calibrated against the three other types of test (Larsson 1989). These are also the tests normally used in Sweden.

The results from shear strength tests normally show scatter and sometimes the different methods yield different results. An estimation of a representative shear strength normally has to be made. This is based on the empirical reliability of the various methods and their inherent problems and shortcomings in various types of soils and in special soil conditions (Larsson et al 1984).

Similarly, neither the plasticity index nor the liquid limit are quite unique parameters. At present, there are two different methods in use for determining the liquid limit; the Casagrande percussion method and the fall-cone method. In general, the two methods give compatible results but differences can be observed for very high-plastic and very low-plastic soils. The results are somewhat sensitive to the way in which the specimens are treated before the tests and especially the percussion test is sensitive to the equipment and the operator's skill.

Also the determination of the plastic limit is somewhat sensitive to details in the test procedure and to the operator. A detailed report on the determination of consistency limits and possible errors in the tests has been given by Karlsson (1974 and 1981).

The possible errors in the determination of both liquid limit and plastic limit are incorporated in the evaluated plasticity index. The accuracy of the determination is therefore not as good as would be desired and in low-plastic clays the relative errors may become very large.

Empirical relations that are strongly dependent on the plasticity index can therefore be used with confidence only in high and medium-plastic clays. In low-plastic clays, relations based on the liquid limit may be used as the relative error for this parameter is limited, even if it may be significant.

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