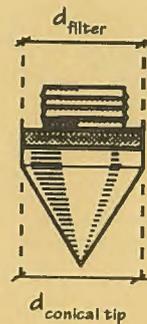
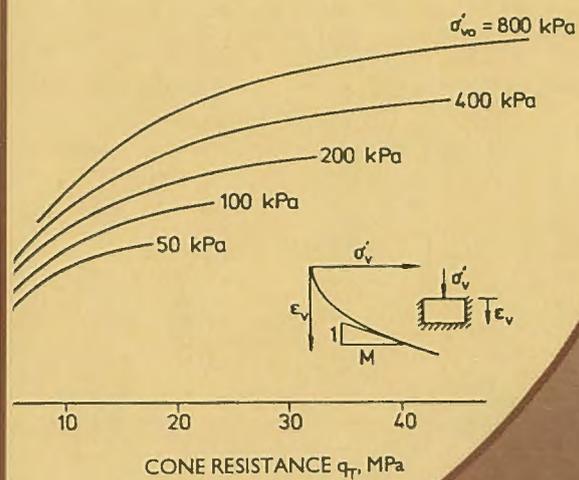
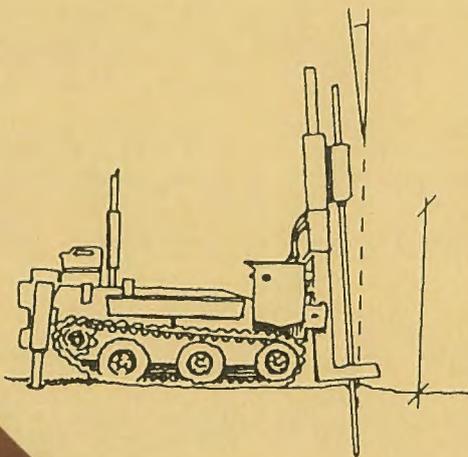


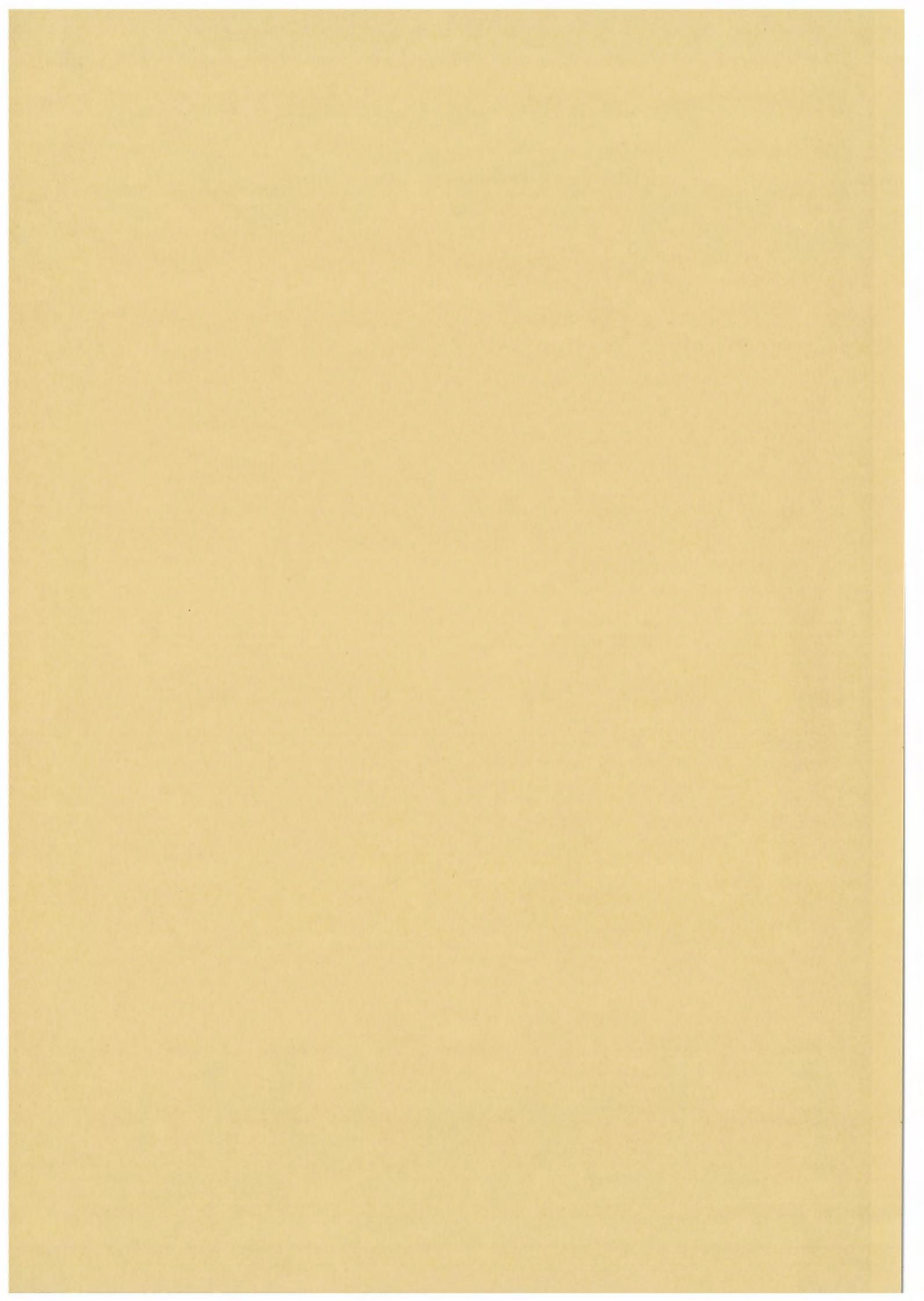
# The CPT test

equipment - testing - evaluation



$$d_{\text{filter}} = d_{\text{conical tip}} \begin{cases} +0.2 \\ -0.0 \end{cases} \text{ [mm]}$$







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

# Information 15 E

## **The CPT test**

equipment - testing - evaluation

An in situ method for determination of stratigraphy and properties  
in soil profiles

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# Preface

SGI Information No. 15E describes cone penetration testing with and without measurement of pore pressures, CPT tests. A description of the equipment and a guide for the performance of the test is given, together with recommendations for evaluation of the results. The latter are based on experience from Sweden and from abroad, and are adjusted to Swedish soil conditions and rules for classification. In various connections, this type of test has also been designated Dutch sounding test, combined cone penetration and pore pressure sounding test, CPTU test and piezocone test and in Sweden TrSP or TrS sounding. In this publication, the designation CPT test is used for tests both with and without pore pressure measurement in accordance with the standard produced by the Swedish Geotechnical Society.

Information No. 15E describes how CPT tests should be performed and evaluated according to the experience gained at SGI and what type of results and reliability can be expected.

The information is intended for engineers who perform or purchase CPT tests, or use the results from such tests.

The CPT test is included in a larger investigation concerning new in situ methods for determination of stratigraphy and properties in soil which has been carried out at SGI and was financed jointly by the Swedish Council for Building Research (Grant No. 880572-1), SGI and the Swedish Road Administration. Other publications related to this project are listed in the chapter "Literature".

Research concerning CPT tests at SGI has mainly been focused on special Swedish conditions with very soft fine-grained soils and has constituted a supplement to the extensive research which is carried out around the world. The parts of this publication which deal with standards for equipment, test performance and reporting of results have been co-ordinated with the standardisation work carried out in the Field Testing Committee of the Swedish Geotechnical Society. The illustrations have been supplied by the Section for Geotechnical Engineering at the Swedish National Road Administration.

This publication is written by Rolf Larsson, SGI. A large number of colleagues and research institutions have also made their knowledge and experience available.



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# CHAPTER 1.

## Introduction

In geotechnical engineering, there is a great demand for rational ways of continuously determining stratigraphy and assessing soil properties under natural conditions in situ. A large number of methods have been developed for this purpose in recent years. Following an extensive review of those methods which are mainly intended for soft and medium stiff soils with grain sizes up to the gravel fraction, the CPT test with measurement of pore pressure was selected as one of the methods considered to have good prerequisites for use in Swedish soils. Research and development of the methods has thereafter been carried out in Swedish conditions for a number of years.

The principal area for using the CPT test is, as mentioned above, determination of soil stratigraphy and a preliminary assessment of the geotechnical properties. The test is used in soft and medium stiff soils with grain sizes up to the gravel fraction, i.e. soils in which a probe can be pushed down without the use of blows or rotation. In these soils, the method is unsurpassed in terms of rational determination of stratigraphy. In pure sand, the test can be performed without measurement of pore pressure, except for tests at such water depths that the hydrostatic pressures are large enough to affect the test results. In all other types of soil, the CPT test is performed with simultaneous registration of the pore pressures generated during the penetration. The penetration comes to a stop because of gravel layers, stones, stiff and/or coarse-grained tills, very dense sand layers or bedrock.

Other types of in situ tests, as well as sampling and laboratory tests, may be better suited for a detailed determination of specific properties in different soils. Areas and levels where such supplementary investigations should be performed can be localised from the results of the CPT tests. A judgement of what method will be suitable for this purpose can also be made.

### FRICITION SOIL

For some types of soil, particularly sand, there is comprehensive empirical experience from CPT tests and the results are often used directly for estimation of bearing capacity and settlements in these soils. In this case, semi-empirical methods specially developed for this test method are normally used.

Also certain other geotechnical parameters can be estimated with relatively good accuracy. In sand, this is especially valid for the friction angle. When parallel results from dilatometer tests are available, whereby an estimation of the in situ horizontal stress can be made, the evaluation of the friction angle can be very good.

In some cases, different moduli and the relative density in sand can be estimated. However, the overconsolidation of the soil, the tendency to crushing of the material and the grain size distribution have large effects on the evaluation of these parameters and the latter factors cannot be estimated from the test results. The existing empirical relations are therefore only applicable to relatively even-graded, normally consolidated quartz sands.

### COHESIVE SOIL

In fine-grained soils, such as clay and gyttja, the undrained shear strength can be estimated and a general view of preconsolidation pressure and overconsolidation ratio obtained. The empirical relations for these properties are dependent on the consistency limits of the soil and a good estimation therefore requires supplementary soil sampling. For this purpose, disturbed samples are sufficient. An evaluation of properties in soft fine-grained soils also puts special demands on the test equipment and its handling.

## POSSIBILITIES WITH CPT TESTS

The advantages of electrical measurements at the tip of the probe are mainly increased accuracy, repeatability, the possibility of measuring several parameters and an almost continuous registration versus depth. In fine-grained soils, the addition of pore pressure measurement has led to:

- a possibility to correct measured values of tip resistance and sleeve friction for the unbalanced water pressures, which act on the end surfaces of the various parts of the probe and which in this type of soil may constitute large sources of error in the measurements. After such a correction, relevant values and compatible results between probes of different manufacture and design are obtained.
- a possibility to estimate the drainage properties in soil layers.
- a possibility for rational measurement of in situ pore pressures, mainly in more permeable seams and layers in the soil profiles.
- increased possibilities for evaluation of soil stratigraphy and classification of the soil layers.
- increased possibilities for estimation of properties in fine-grained soils.

## DEMANDS IN CPT TESTING

The CPT test with electrical measurement of cone resistance, sleeve friction and pore pressure at the tip of the probe is combined with high demands on accuracy, calibration and handling of the equipment. Strict adherence to a detailed standard for both equipment and the performance of the test is therefore essential.

CPT tests, especially with measurement of pore pressure, entail a somewhat higher initial cost than other test methods with the same purpose and also require skilled personnel with some knowledge and expertise in electronic measurement. The method also demands more in terms of preparations and checks after the test and also in terms of facilities for maintenance and calibration. On the other hand, it is the method that provides the most detailed information about stratigraphy. Recent research and development has also significantly increased the possibilities for evaluation of soil properties from the test results.

## CPT TESTING - DEVELOPMENT AND EMPIRICAL EXPERIENCE

Penetration testing in soft soils for determination of stratigraphy and depth to firm bottom has taken various forms over a long time; in Sweden since about 1915 when the Weight Sounding Test was introduced. In order to obtain a good resolution and measurements that are representative for the specific soil layers, the resistance to penetration should be measured at the tip so that it is independent of the rod friction. The first types of CPT tests with measurement of cone resistance were introduced around 1935 and the measuring principle was then mechanical. Since the 1950s, the measuring principle has increasingly changed to electrical measurements and further parameters have been incorporated. In addition to cone resistance, the friction against a sleeve located immediately above the conical tip became a commonly measured parameter. Other parameters which are sometimes measured are temperature and inclination of the probe.

Pore pressure sounding was introduced in 1975. The purpose of this test is mainly to obtain a picture of the soil profile where layers with different drainage properties can be better distinguished and where both relatively thin layers of sand and silt embedded in clay and thin clay layers in friction soil can be registered. This type of test was standardised in Sweden in 1984.

In Sweden, CPT tests and pore pressure soundings have often been performed as separate and sometimes parallel tests. In other countries, and nowadays also in Sweden, the two methods have been integrated into CPT tests where cone resistance, sleeve friction and pore pressure are measured simultaneously. CPT tests are extensively used in some countries as well as in offshore investigations.

Cone penetration tests have thus been used extensively for the last 40-50 years, mainly abroad but also in Sweden. The empirical experience from tests, mainly in normal sands with quartz and feldspar minerals, is comprehensive. Special calculation methods for bearing capacity and settlements in this type of soil have been produced and are commonly used, as are classification charts mainly for friction soils. Since the pore water pressure normally has no significant influence on the test results in friction soil, the gathered experience can be utilised independent of whether the pore pressure is measured or not. (In those cases where the water pressure has an influence on the results in this type of soil, which is mainly the case for tests at large water depths offshore, it constitutes a source of error

which can be eliminated by using equipment with pore pressure measurement and correction for the measured pore pressure.)

Research concerning CPT tests in sand has in recent years been concentrated to tests in large scale calibration chambers for further refinement of existing relations and, if possible, an extension of the number for evaluated properties. In this connection, also the possibility of improving the evaluation by using parallel dilatometer tests has been investigated.

The empirical data base from fine-grained soils is of a later date. For tests in this type of soil, it is only during recent years that geotechnical engineers have become fully aware of the importance of correction for pore pressures, temperature stability of the equipment, design tolerances and filter position. The entire data base from earlier CPT tests without pore pressure measurements, as well as large parts of the data base from CPT tests with pore pressure measurements, is therefore inapplicable (except for specific cones under specific conditions in specific regions). However, in recent years comprehensive investigations have been carried out in softer and fine-grained soils with all these factors being observed.

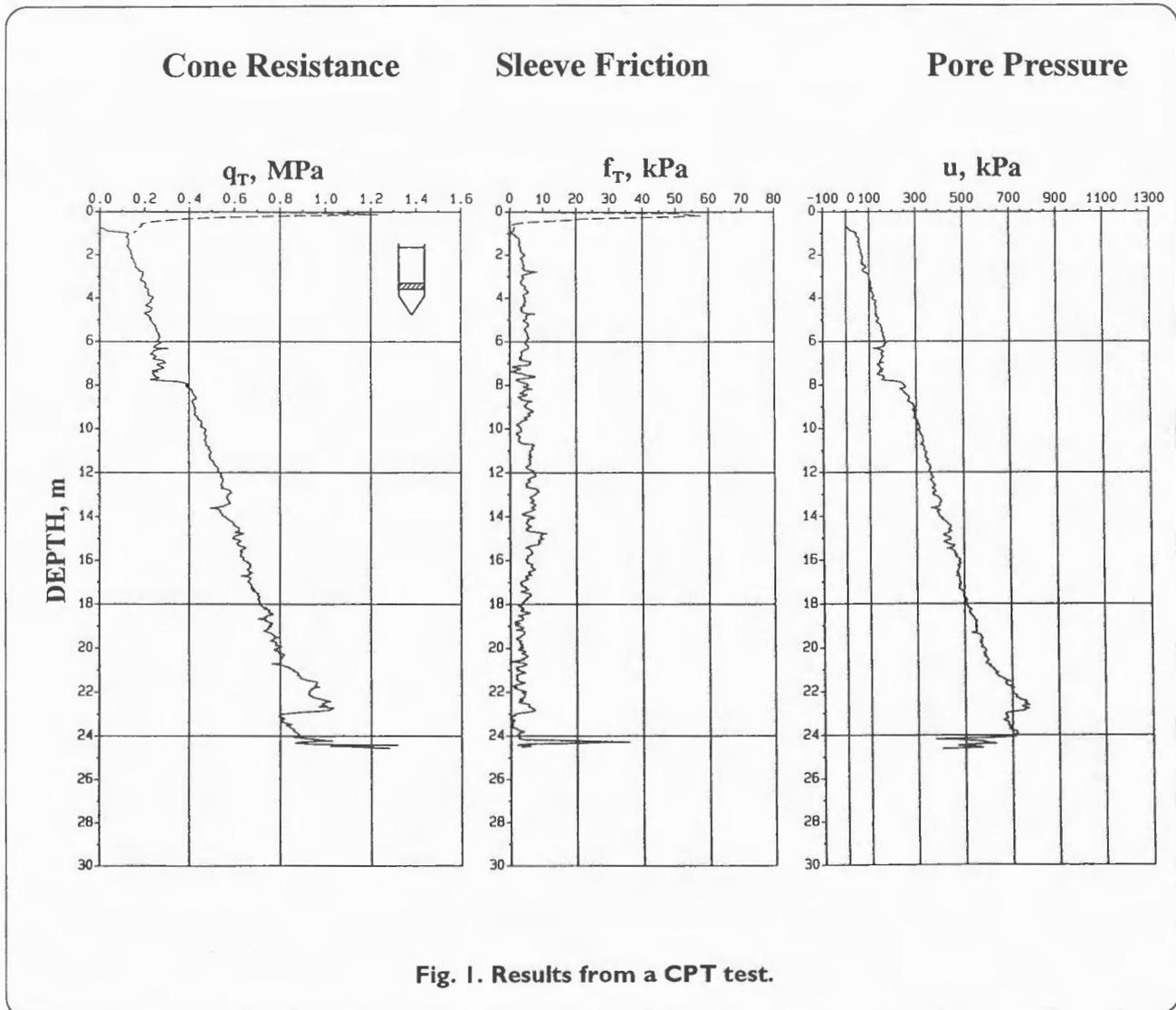
## CHAPTER 2.

# Principle of the CPT test

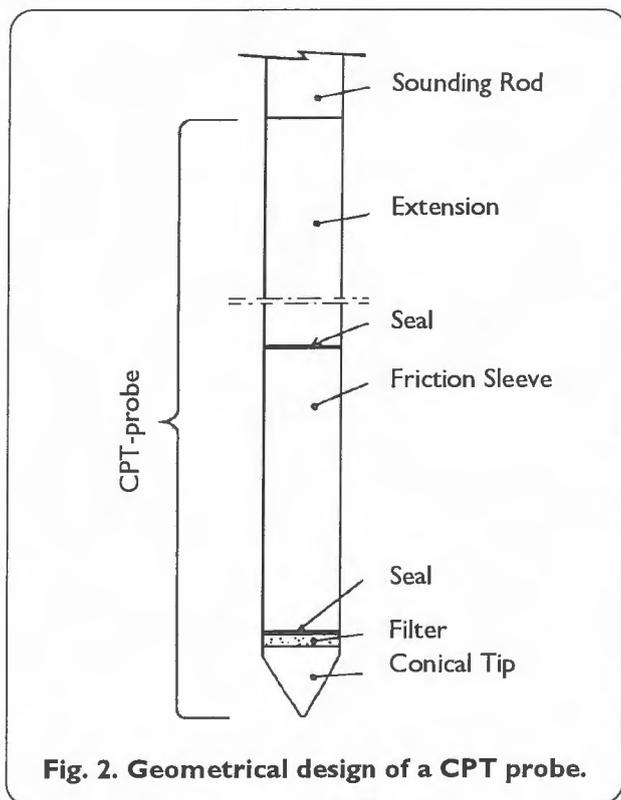
### 2.1 TEST METHOD

In the CPT test, a cylindrical probe with a cross sectional area of  $1000 \text{ mm}^2$  and a tip angle of  $60^\circ$  is pushed down into the soil at a constant rate of  $20 \text{ mm/s}$ . During this process, the following parameters are measured; the penetration resistance force against the cone tip, the friction force against the

cylindrical surface just above the tip and, in simultaneous pore pressure registration, the pore pressure generated at the tip during the penetration. The measurements are made by means of electronics and the frequency of the recordings must be high enough to provide almost continuous curves for the variation in the measured values with depth, *Figure 1*.



The geometrical design of the probe and some of the designations used are shown in *Figure 2*.



**Fig. 2. Geometrical design of a CPT probe.**

The main purpose of the test is usually to obtain a detailed picture of the stratigraphy and the variation in the soil properties with depth. Even though the measurements are taken almost continuously, the following aspects have to be considered:

- The measured values of the different parameters are not fully independent of each other because of the design of the cone. They also vary with the geometrical design of the cone and possible wear. This puts demands on standardisation and specified tolerances. In order to obtain relevant values for cone resistance and sleeve friction in soundings in profiles where high water pressures prevail or are created during the test, the measured values for both cone resistance and sleeve friction have to be corrected. This is especially valid for tests in fine-grained soils and for all tests at large water depths.
- The cone resistance is influenced by the volume of soil around the tip. The volume varies with the stiffness of the soil. The normal assumption is that for tests in layered soils, the thickness of a stiffer layer has to be 0.4-0.7 m if fully representative values of the cone resistance are to be obtained in the middle of the layer. For softer layers, the

corresponding thickness is 0.2-0.4 m. Thinner stiff and soft layers respectively are also registered in the continuous taking of measurements, but the values then have insufficient time to stabilise and become fully representative for the particular layer.

- The sleeve friction is measured as an average value over a length of 134 mm.
- The pore pressure is normally measured by using a 5 mm high filter and can be considered to be representative for the particular level. The possibility of registration of thin layers is thereby primarily dependent on the recording frequency and a sufficiently quick response by the pore pressure measuring system to pore pressure changes in the soil.
- The measurements of cone resistance, pore pressure and sleeve friction are taken at different positions along the probe. Values of the different parameters which are relevant for the same level will thereby be taken at different times<sup>\*)</sup>. The cone resistance must be corrected for the pore pressure acting when the reading of the cone resistance was taken and the sleeve friction must be corrected for the pore pressure acting when this later reading is taken. In densely layered soils, this puts special demands on the process and frequency of reading. The differences must also be taken into account in connection with temporary stops in the penetration process and related stress relaxation, for example when adding new sounding rods and reclutching after each full stroke of the drill rig.
- At temporary stops in the penetration process, also the dissipation processes for the generated excess pore pressures can be studied. This can be a considerable aid for soil classification and for estimation of the drainage properties. In more permeable soils and layers, where this dissipation is rapid, values of the prevailing in situ pore pressures can also be obtained.

<sup>\*)</sup> In continuous penetration at the standard rate using a probe with the filter placed in the normal position above the tip, this means that for a certain level a relevant value for the cone resistance is first registered, followed about 0.7 seconds later by the pore pressure and after another 3.4 seconds by the related sleeve friction. The normal filter position is defined as a 5 mm high filter placed with its lower end 5 mm above the shoulder of the conical tip.

## 2.2 STANDARDS AND RECOMMENDATIONS

In Sweden, there is a standard for CPT tests including pore pressure measurements recommended by the Swedish Geotechnical Society. A very similar standard has also been produced by the Norwegian Geotechnical Society. Beside these, there is the recommendation of the International Geotechnical Society, ISSMFE, for "Reference Test Procedures". The latter includes the CPT test among other tests.

In the recommendation by ISSMFE from 1989, the specifications for the CPT test were slightly altered in relation to earlier standards. The design of the probe was altered in such a way that a filter for pore pressure measurement could also be incorporated. In other respects, this recommendation is very vague with regard to pore pressure measurements and handling of "piezocone" data. It is primarily applicable for tests in coarse-grained soils and needs to be supplemented and clarified for tests in fine-grained soils. Such additions and clarifications have been incorporated in the Scandinavian recommendations, which are applicable to all types of soils.

The international recommendation is thus mainly applicable to the CPT test without pore pressure measurements, which in Sweden was previously called "tip resistance sounding, TrS". Other international designations for this type of test are Static Penetration Test, Quasi-Static Penetration Test and Dutch Sounding Test. The Scandinavian recommendations also include the combined cone penetration test-pore pressure sounding test and the test methods that have been designated TrSP sounding, CPTU test and Piezocone test. They are based on new and previous experience from CPT tests in sands and other coarse-grained soils and on recent experience from CPT tests with pore pressure measurements in fine-grained soils in Scandinavia and other parts of the world.

In the recommendations for the test equipment, it is primarily the probe, constituting the approximately 1.05 m long lower part where the measurements are taken, which has standardised dimensions and tolerances. For the rest of the equipment, the demands refer to the particular function.

The specifications in this information are in accordance with the Swedish recommendation.

## 2.3 DEFINITIONS

### Cone resistance $q_T$ (alt. $q_c$ )

The cone resistance is the force per unit area which is obtained by dividing the total axial force against the cone tip by the cross sectional area of the tip (1000 mm<sup>2</sup>).

The designation  $q_c$  is used for CPT tests without pore pressure measurement and  $q_T$  is used when the pore pressure at the base of the conical tip is measured.

Because of the design of the cone, the measured force on the tip is affected by unbalanced pore water pressures. These may constitute large sources of error in cases where the pore pressures are high. In these cases, the measuring element for the cone resistance does not register values that are relevant for the total cone resistance. In tests with measurement of the pore pressure at the base of the conical tip, the measured cone resistance can be corrected and the total cone resistance obtained as follows:

$$q_T = \frac{\text{Total axial force on the cone tip}}{\text{Cross sectional area}}$$

$$(q_c = \frac{\text{Uncorrected measured value of axial force on the cone tip}}{\text{Cross sectional area}})$$

(In the special case when the pore pressure  $u \approx 0$  or is insignificant, then  $q_c \approx q_T$ ).

The cone resistance is given in MPa or kPa.

### Sleeve friction $f_T$ (alt. $f_s$ )

The sleeve friction is obtained by dividing the total frictional force against the friction sleeve by the surface area of the sleeve (15,000 mm<sup>2</sup>). The measured values of the sleeve friction are affected by unbalanced water pressure on the end surfaces of the sleeve. The measured value must therefore be corrected for significant water pressures in order to obtain relevant values of the total sleeve friction.

$$f_T = \frac{\text{Total sleeve friction}}{\text{Sleeve surface area}}$$

$$(f_s = \frac{\text{Uncorrected measured value of sleeve friction}}{\text{Sleeve surface area}})$$

(The parameter  $f_s$  is only relevant in coarse-grained soils at low water pressures)

The sleeve friction is given in kPa or MPa.

### Friction ratio $R_f$

The friction ratio  $R_f$  is defined as the ratio between sleeve friction and cone resistance at the level under consideration

$$R_f = \frac{f_T}{q_T} \cdot 100, \quad \%$$

(Alternatively, the friction index  $I_f = q_T/f_T$  may be used).

### In situ pore pressure $u_0$ (kPa)

corresponds to the prevailing in situ pore pressure at the particular level in the ground, which is also reinstated after full pore pressure equalisation at stops in the penetration process.

### Registered pore pressure $u$ (kPa)

is the pore pressure measured during the penetration test ( $u = u_0 + \Delta u$ ). The designation  $u$  is only used for pore pressures measured at the normal filter position above the conical tip. For pore pressures measured at the alternative filter position halfway up on the conical part of the tip, the designation  $u_{\text{FACE}}$  is used \*).

### Generated pore pressure $\Delta u$ (kPa)

is the change in pore pressure ( $u - u_0$ ) which occurs because of the penetration process. The generated pore pressure can be positive or negative depending on the properties of the soil and is also strongly dependent on the position of the filter on the probe. (At the alternative filter position, the generated pore pressure becomes  $\Delta u_{\text{FACE}} = u_{\text{FACE}} - u_0$ ).

### Differential pore pressure ratio DPPR

The differential pore pressure ratio is defined as the ratio between generated pore pressure and cone resistance at the level under consideration

$$DPPR = \frac{\Delta u}{q_T}$$

## 2.4 TEST CLASSES

CPT tests are performed in all types of soil into which the probe can be pushed down, from dense coarse sand

with gravel sized particles to soft fine-grained soils. The probes used in Sweden often have a capacity of 5-tonnes axial force on the tip, partly because of the limitations of the pushing equipment. In other countries, probes with capacities of 10 and 20 tonnes are common. This makes tests in stiffer soils possible, but also entails limitations in measuring accuracy, which limits their usefulness in soft and fine-grained soils.

One wish is to have equipment which can be used in as wide a range of soil conditions as possible. Another is to have sufficient accuracy and resolution of the measurements that a detailed classification and evaluation of soil properties can be made also for soft and fine-grained soils.

In coarse-grained soils, considerable wear, mainly of the conical tip, occurs and fairly wide tolerances for the geometry of the tip must be accepted. In fine-grained soils, where also the pore pressures are used in the evaluation, the exact geometry is important and the discrepancies must be kept to a minimum.

These wishes and demands are contradictory and incompatible. Because of the wide range of the investigations and the fact that different demands are set depending on the soil type and use of the results, the tests are divided according to the Swedish recommendations into three classes with different demands on accuracy. These demands are lowest in class 1 and highest in class three.

The following factors are considered as a basis for the division into classes:

- The Swedish recommendation for a standard for the CPT test deals only with equipment where the measurements are taken by using electronics. They comprise cone resistance, sleeve friction and, in most test classes, also pore pressure.
- In coarse-grained soil, the only information normally obtained from the pore pressure measurements is that no excess pore pressures are generated. The exception is tests at large water depths, where measured values of cone resistance and sleeve friction need to be corrected for the water pressures. In fine-grained soils, the pore pressures should always be measured and cone resistance and sleeve friction corrected. In the case of insufficient adherence to the specified design dimensions, the measurement of generated pore pressure involves so much uncertainty that pore pressure measurement can only be used for correction of the other parameters and possibly for assessment of the in

\*) Only pore pressure measurements at the normal filter position are included in the recommendation.

situ pore pressure. For a more detailed classification of fine-grained soils and an estimation of their properties, fairly strict tolerances must therefore be applied.

- The measuring accuracy is limited by factors such as the resolution of the measuring system (the accuracy of volt and ampere meters and the number of digital units in transfer and storage of data). The resolution is limited by the maximum measuring range divided by the number of digital units. If, for example, the number of digital units for a five-tonne probe is 4000, then the resolution becomes  $(5000 \cdot 98,1)/(10 \cdot 4000) \approx 12$  kPa. A better measuring accuracy than  $\pm 1 - 2$  times the resolution cannot be achieved.
- The pore pressure measuring device must have a very small deformability of its own, as well as being insensitive to temperature changes and very accurate. This is also valid for the coarsest test class (see below), in which it must be possible to measure the pore pressure with an accuracy of  $\pm 10$  kPa with devices which often have measuring ranges up to 40 Bar  $\approx 4000$  kPa. This entails a measuring accuracy of about  $\pm 0.25$  %, all sources of error included. High class transducers of this type, which can be incorporated in the probes, are required and are available on the market.
- The probes can be made relatively insensitive to temperature changes for a limited extra cost. This should always be stipulated. Most tests are performed without control and registration of temperature and the only way to ensure that the demands on accuracy are fulfilled is to make the temperature stability so good that all possible influence of temperature changes is negligible.

#### Division into test classes:

##### *Class 1. Designation: CPT1*

- Range of application: All types of soil, but mainly coarse-grained soils. In fine-grained soils, only in cases where the stiffness or stratigraphy of the soil prevents the use of a higher test class.
- Measurement of cone resistance and sleeve friction, and possibly also pore pressure.
- Relatively wide tolerances for probe dimensions (see Chapter 3.2).
- Measuring accuracy according to recommended standard, (see Chapter 3.2), with generally accept-

ed inaccuracies of 100 kPa in cone resistance, 10 kPa in sleeve friction and 10 kPa in pore pressure.

- Possible interpretation: Stratigraphy and properties in coarse-grained soils and coarse silt. Stratigraphy and properties in stiff fine-grained soils can be determined, provided that the pore pressure is measured in the test. Evaluation of stratigraphy and properties in soft and medium stiff fine-grained soils should not be performed.
- Equipment: 5 - 20 tonne probes.
- Limitations: The ground water level has to be determined separately. At large water depths, the test should be performed with pore pressure measurement. If possible, a higher test class should be used in mixed and fine-grained soils.

##### *Class 2. Designation: CPT2*

- Range of application: All types of soil. In very dense sand and coarser soils, the possible penetration is limited. In soft and medium dense fine-grained soils, the tests should preferably be performed according to class CPT3 when the stratigraphy permits this.
- Measurement of cone resistance, sleeve friction and pore pressure.
- Narrow tolerances adapted to tests in fine-grained soils (see Chapter 3.2).
- Measuring accuracy according to recommended standard, (see Chapter 3.2), with generally accepted inaccuracies of 40 kPa in cone resistance, 4 kPa in sleeve friction and 5 kPa in pore pressure.
- Possible interpretation: Stratigraphy and properties in all types of soil. The accuracy is limited in soft and medium stiff fine-grained soils.
- Normal equipment: 5-tonne probes with pore pressure measurement.
- Limitations: Limited penetration in dense, coarse-grained soil. Insufficient accuracy for an adequate evaluation of properties in soft and medium dense fine-grained soil.

##### *Class 3. Designation: CPT3*

- Range of application: All types of soil. However, the penetration is limited mainly in sands and coarser soil, and also in other stiff soils. In those cases where load restrictions and accuracies for this test class cannot be fulfilled, the test class has to be lowered.
- Measurement of cone resistance, sleeve friction and

pore pressure.

- Narrow tolerances adapted for tests in fine-grained soils (see Chapter 3.2).
- Measuring accuracy according to recommended standard, (see Chapter 3.2), with generally accepted inaccuracies of 20 kPa in cone resistance, 2 kPa in sleeve friction and 1 kPa in pore pressure.
- Possible interpretation: Stratigraphy and properties in all types of soil, but mainly in soft and

medium stiff fine-grained soil.

- Normal equipment: Specially calibrated 5-tonne probes with pore pressure measurement or corresponding probes with lower measuring ranges.
- Limitations: Very limited penetration in stiff soils and when coarser particles are present in the soil.

The ranges for application of the various test classes are also shown in *Table 1*.

**Table 1. Range of application for different test classes.**

Class	Type of soil					
	Coarse-grained		Silt		Fine-grained	
	Stratigraphy	Properties	Stratigraphy	Properties	Stratigraphy	Properties
CPT1 <sup>A</sup>	● <sup>1</sup>	● <sup>1</sup>	○	○	-	-
CPT1 <sup>B</sup>	●	●	▲	▲	○ <sup>2</sup>	○ <sup>2</sup>
CPT2	(●) <sup>3</sup>	(●) <sup>3</sup>	●	●	▲	▲
CPT3	((●)) <sup>4</sup>	((●)) <sup>4</sup>	(●) <sup>5</sup>	(●) <sup>5</sup>	●	●

A) Without pore pressure measurement

B) With pore pressure measurement

Estimation:

- Good
- ▲ Relatively good
- Coarse
- Not possible

Remarks:

- 1) The ground water level has to be determined separately. At large water depths the tests should be performed with measurement of pore pressure.
- 2) Only in stiff fine-grained soil.
- 3) Somewhat limited application in coarse-grained soil.
- 4) Very limited application in coarse-grained soil
- 5) Limited application in stiff silt

## CHAPTER 3.

# CPT test equipment

### 3.1 PRINCIPLES OF MEASUREMENTS AND DETAILED CALIBRATION

#### 3.1.1 Measuring principle

CPT tests measure cone resistance, sleeve friction and, in most cases, also pore pressure. The axial force acting on the cone tip is transferred to the upper parts of the probe by a measuring body with applied strain gauges. The gauges must be applied in such a way that a possible eccentricity of the force does not affect the results. They must also be connected to compensating elements in such a way that temperature changes do not affect the results. The possibilities for this compensation are relatively good, but there are certain limitations. It is thus especially difficult to fully compensate for the effect of temperature gradients inside the probe due to changes in temperature.

In a corresponding way, the force from the friction sleeve is transferred by another measuring body with strain gauges. For manufacturing reasons, many cones have been designed with two almost identical measuring bodies, one on top of the other. The lower body measures the cone resistance and the other the sum of the cone resistance and sleeve friction. The measured value of the sleeve friction has then been obtained by the difference in signals from the two measuring bodies, so called "subtraction probes". This type of design entails a high risk of interference between the signals with related problems in calibration, interpretation and measuring accuracy. Newer designs are therefore usually of the "compression probe" type, in which the sleeve friction is measured separately by a measuring body placed in such a way that it is unaffected by the axial force from the cone tip.

The pore pressure is measured by pressure transducers. In order to obtain the required measuring accuracy, the transducers have to be of very high quality, with a very small deformability of their own in the measurements. Transducers of this type are on the

market and can be installed inside the probes. This requirement has often had to be specified separately when ordering the equipment, but it will hopefully become standard in all commercial equipment after introduction of the new recommended standards for CPT tests including pore pressure measurements.

The design of a CPT-probe is shown schematically in *Figure 3*.

#### 3.1.2 Correction for pore water pressures

As is also shown in the figure, the measurements of both cone resistance and sleeve friction are affected by unbalanced pore pressures at the base of the conical tip and at the end surfaces of the friction sleeve. To obtain the total cone resistance and the total sleeve friction respectively, the measured values  $F_c$  and  $F_s$  from the two measuring bodies have to be corrected for the water pressures acting on the end surfaces.

At the normal position of the filter, the pore pressure is measured in the slot between the cone tip and the friction sleeve, so that the unbalanced water pressure against the base of the cone tip and the lower end of the friction sleeve is then known. The pore water pressure at the upper end of the friction sleeve,  $u_U$ , is normally not measured but can be estimated with relatively good accuracy from

$$u_U \approx 0.7(u - u_0) + u_0 \approx 0.7\Delta u + u_0$$

Knowing the areas of the end surfaces affected by different water pressures, the measured values of axial force and friction sleeve can thus be corrected for the influence of the pore pressure. The size of the end areas is normally expressed by the area factors  $a$  and  $b$ .

**Area factor  $a$**  is used for correction of the measured value of cone resistance

$$a = A_N / A_T \approx (A_T - A_L) / A_T$$

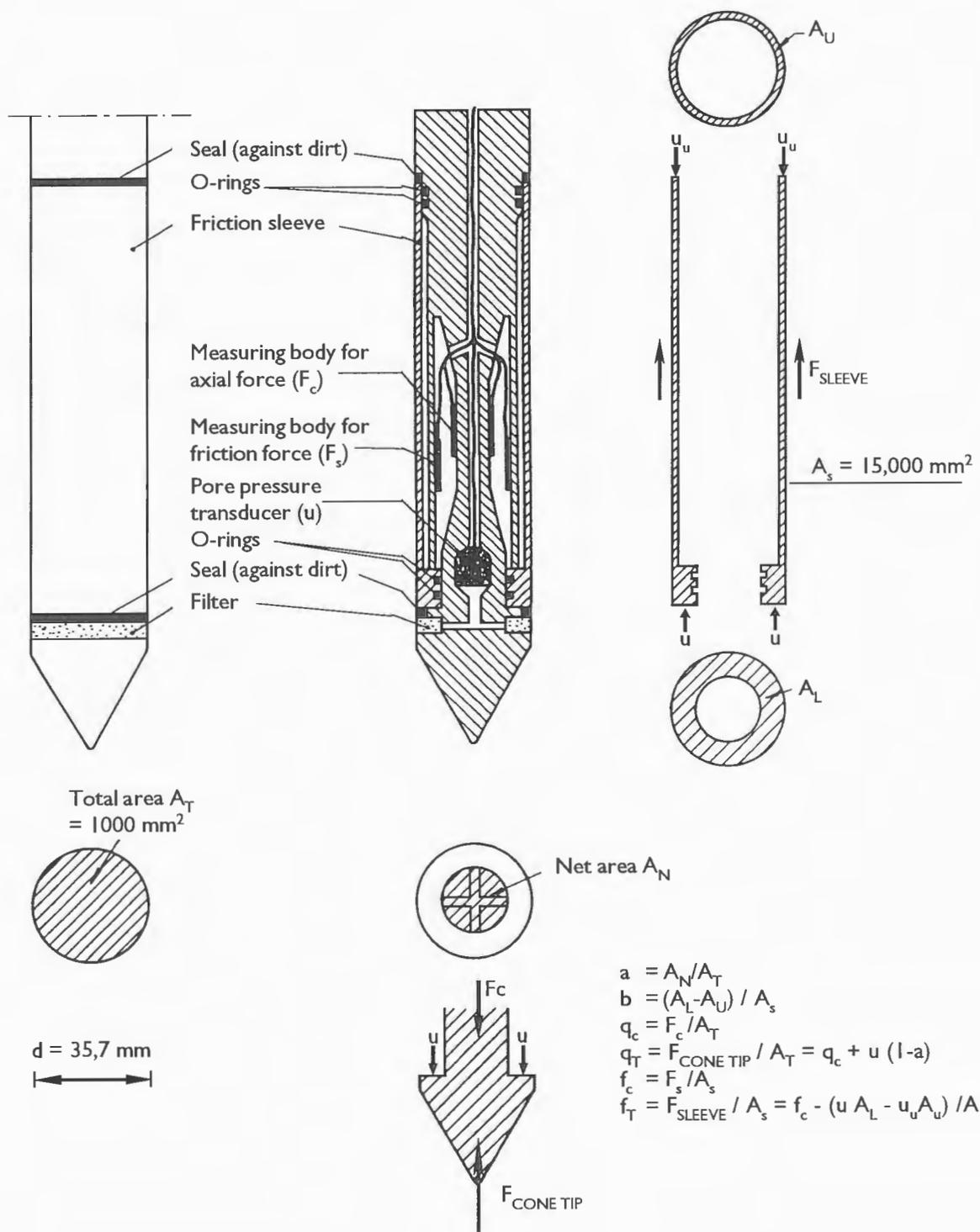


Fig. 3. Schematic design and measuring principle of a CPT-probe of "compression" type.

and

$$q_T \approx q_c + u(1-a)$$

Area factor **b** is used for correction of the measured value of sleeve friction

$$b = (A_L - A_U) / A_S$$

Using the empirical relation for the pore pressure at the upper end of the friction sleeve, the corrected total sleeve friction becomes

$$f_T \approx f_c - (ub + 0.3\Delta u(\frac{1-a}{15} - b))$$

### 3.13 Detailed calibration

There are a number of other sources of error in the measurements, such as internal friction in the O-ring seals. Also, the area factors cannot be estimated directly from geometry because of uncertainties about the way in which different areas are affected by the water pressures and how the forces are transferred. Even if it is intended that the measuring bodies for cone resistance and sleeve friction shall only be affected by the respective forces, this has to be checked. Each new probe therefore has to be subjected to a detailed calibration of area factors, influence of internal friction and possible "cross-talk" or interference effects. These values are specific for the particular probe, but they may change if tips with a different geometrical design are used. They must therefore be checked for each type of tip that is to be used and recalibrated following any change in the design of the cone.

Most probes are designed for use in soft to stiff soil, from organic soil and clay to coarse sand. The maximum force on the cone tip is normally 5 to 10 tonnes (50 to 100 kN). A high steel quality together with matching strain gauges and modern electronics as a rule gives very good properties in terms of stability, linearity and repeatability when the accuracy is expressed in terms of per cent of full scale. However, when these probes are used in soft clay, only about 1 % of the total capacity is used, which means that the very good accuracy at calibration for the full range becomes 100 times worse with respect to the measuring range that is utilised. Many probes still have a very good accuracy also in very low measuring ranges and 5-tonne cones may often be well suited for testing soft soils, provided they are calibrated for this lower range.

**Calibration of cone resistance and sleeve friction** is performed by stepwise axial loading of the cone tip and the friction sleeve respectively. When loading the friction sleeve, the cone tip is replaced by an adapter designed in such a way that the axial load is transferred to the lower end surface of the friction sleeve. The calibrations are made separately, but the non-loaded measuring body is read off simultaneously in order to check that it remains unaffected by loads in the active body\*). The calibration is performed for different measuring ranges and special attention is paid to ranges that are likely to be utilised. In calibration of a new probe, the measuring bodies are first subjected to 15 - 20 load cycles up to full range before the actual calibration is performed.

**Calibration of area factors** must be performed in a special calibration chamber. This chamber is designed in such a way that the lower part of the probe can be inserted into the chamber and locked and sealed above the friction sleeve, *Figure 4*.

The probe can then be subjected to an all round pressure in the chamber. This is applied in steps and at the same time the measuring devices for cone resistance, sleeve friction and pore pressure are read off. **In this way, a calibration curve for the pore pressure transducer, the area factors a and b and measures of the internal friction inside the probe are obtained, *Figure 5*.** By using a special recording unit, also the response time of the pore pressure measuring system for rapid pressure changes can be checked.

The total correction for measured values of cone resistance and sleeve friction after calibration becomes

$$q_T = q_c - o_c - c_1 f_c + u(1-a)$$

and

$$f_T = f_c + o_f - c_2 q_c - (ub + 0.3\Delta u(\frac{1-a}{15} - b))$$

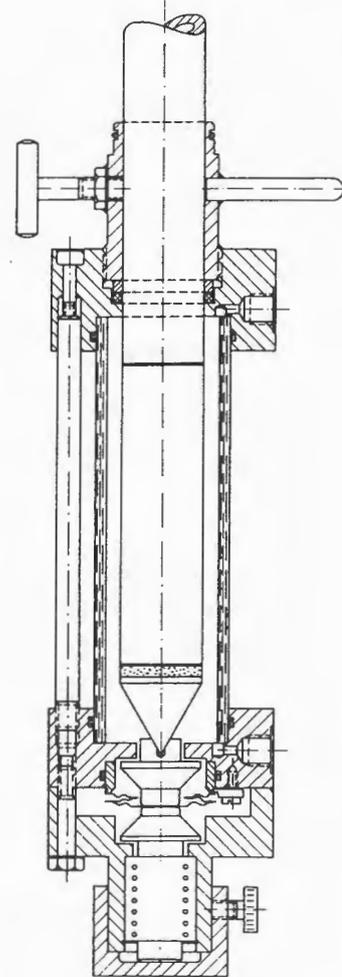
where  $o_c$  and  $o_f$  = internal friction (primarily in O-ring seals)

$c_1$  and  $c_2$  = constants for "cross-talk" between the measurements of cone resistance and sleeve friction

\*) Otherwise the constants for "cross-talk" between the measuring bodies must be determined.



Calibration chamber with probe mounted in the loading frame



Calibration chamber

Fig. 4. Calibration chamber type SGI for CPT probes.

$c_1$  and  $c_2$  shall preferably be very small and negligible.  $\sigma_o$  and  $\sigma_f$  are normally small, but may affect the results in primarily soft and sensitive clays where the cone resistance is very low and the sleeve friction is close to zero.

In correction for the internal friction, it is also necessary to check that the forces on the friction sleeve are so large that the full friction is mobilised. Otherwise, there is a risk of overcorrection, resulting in negative values for the sleeve friction.

Subsequent control calibrations of the pore pressure transducer can easily be performed by application of a pressure supply adapter, which is screwed into the thread for the cone tip and thus does not require a calibration chamber.

The calibrations are performed with the same electronic devices and registration equipment that will later be used in the field to check and calibrate the whole system and its error sources. High precision transducers which are inspected regularly are used as references.

The probes must also be calibrated for temperature effects. This is done by lowering the probes into water baths of different temperatures. The signals are then registered versus time until the readings have stabilised. The zero shifts per °C are evaluated from the results. In addition, a measure of the time taken for the immersed probe to stabilise at a new temperature is obtained. The latter information is valuable when it becomes necessary to stabilise the cone for the ground temperature in the field. According to experience, this

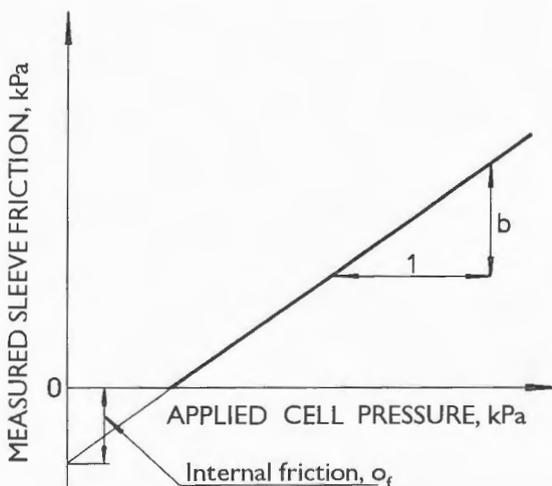
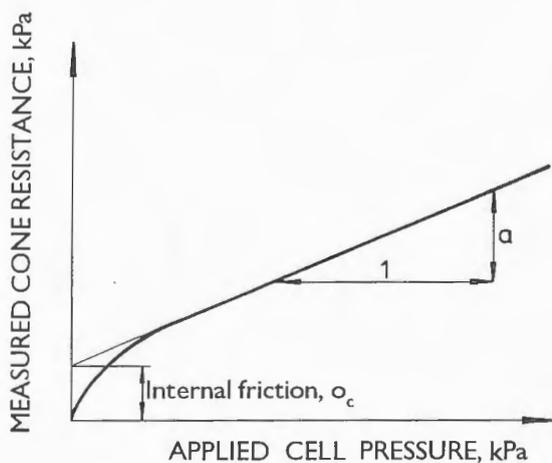
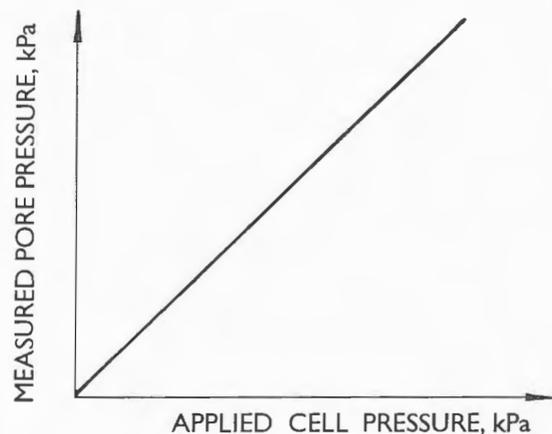


Fig. 5. Results from calibration of a probe in a calibration chamber

value is about 10 minutes, but it may vary for different cone designs.

If the probe is equipped with more measuring devices, such as temperature, inclination or seismic transducers, these must also be calibrated.

### 3.2 DESIGN SPECIFICATIONS AND DEMANDS ON THE EQUIPMENT

This section sets out the specifications for dimensions and demands on the equipment that according to recommendations by the Swedish Geotechnical Society should be applied to CPT testing equipment. Explanations, clarification and motives for these demands are also given, together with the consequences of deviations from the standard.

#### 3.2.1 Geometry of the probe

The outer parts of the probe consist of a conical tip, a filter, a friction sleeve and a cylindrical extension. The friction sleeve and the cylindrical extension must together have a length of at least 1000 mm, *Figure 6*. The diameter along this length must be constant (within given tolerances).

##### Conical tip

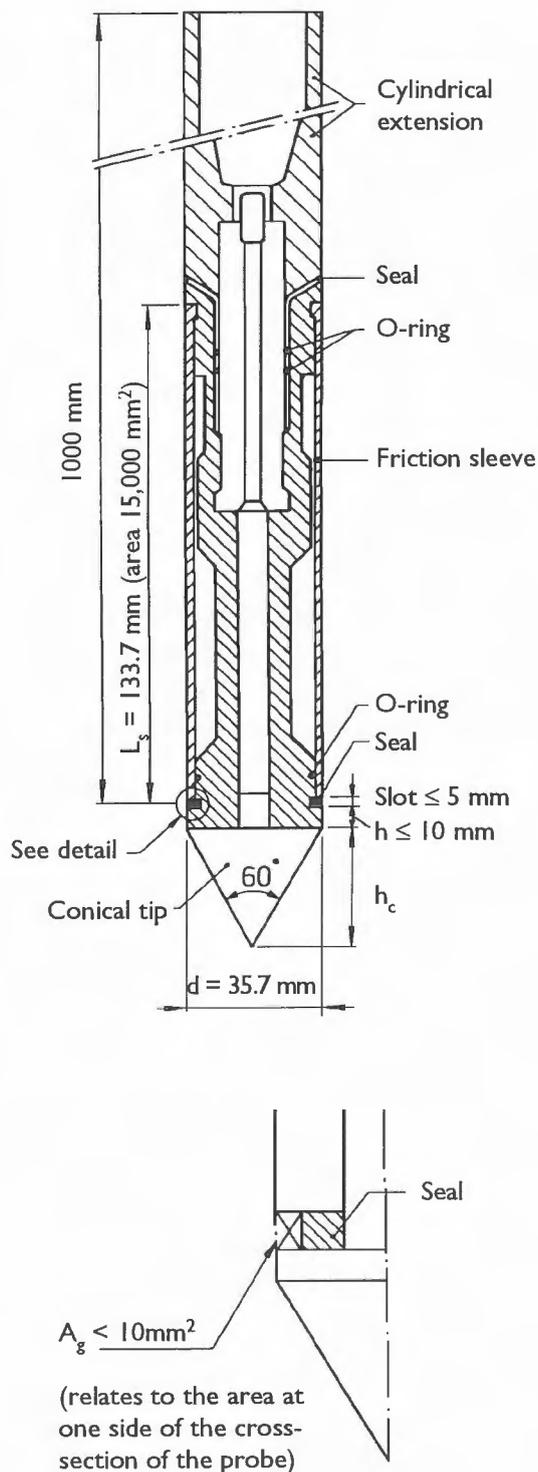
The tip of the probe has a conical part and a cylindrical extension. The tip angle shall be  $60^\circ$ . The cylindrical extension, including a possible filter, must be 10 mm (or shorter within the tolerances for wear). Design, dimensions and tolerances are shown in *Figure 7*.

CPT-probes with measurement of pore pressure must have tips with a cylindrical extension of 5 mm and a filter placed directly above this extension. It has been shown that the generated pore pressures are very sensitive to the location of the filter and therefore only tips and filters of this design shall be used.

CPT-probes with pore pressure measurements which have previously been used in Sweden normally have a cylindrical extension of 5 mm at the tip and filters of 5 mm height, and are thus within the recommendation. On the other hand, the extension is longer in the recommendation compared to earlier recommendations for "tip resistance sounding" without pore pressure measurement.

The cross-sectional area of the tip shall be  $1000 \text{ mm}^2$  at the cylindrical extension, which corresponds to a diameter of 35.7 mm.

The permissible surface roughness along the tip is maximum  $1 \mu\text{m}$ , which roughly corresponds to what normally results from the friction of the soil against



**Fig. 6. Specifications for dimensions of a CPT-probe according to recommendations by the Swedish Geotechnical Society (and ISSFME). The specified dimension of the seal gap  $A_g$  is valid only for class CPT1<sup>A</sup> without pore pressure measurement. For tests with pore pressure measurement the area  $A_g$  must not exceed  $0.5 \text{ mm}^2$ .**

the tip.

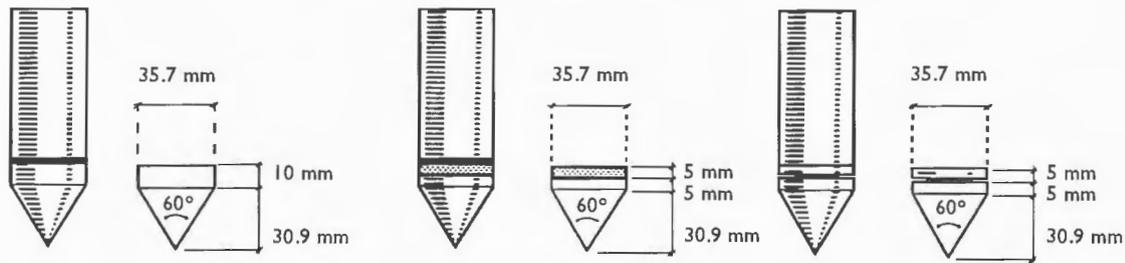
In CPT tests in sand, there is considerable wear of the tip and fairly large tolerances must be accepted. In test class CPT1, where the pore pressures are not measured or alternatively are used only for correction of cone resistance and sleeve friction, it is acceptable for the height of the conical part of the tip to be reduced by up to 7.2 mm and the cylindrical extension by 3 mm. The wear usually also results in a concave shape of the conical part, which is also accepted. However, the shape of the tip must always be symmetric and obviously distorted tips must be discarded.

In CPT tests in fine-grained soils, where the pore pressure is always to be measured and is used for classification of the soil and evaluation of its properties, these large tolerances cannot be applied. They are not compatible with other demands on the geometry of the probe and the generated pore pressures are sensitive to the dimensions of the cone tip. In CPT tests in fine-grained soils, only very moderate wear can be accepted. A certain roundness at the shoulder between the conical part and the cylindrical extension is unavoidable and a reasonable demand is that the height of the cylindrical part must not be 1 mm less than for a new tip. When the soundings are stopped against bedrock or a stone, there is always a certain deformation of the tip and a certain roundness of the extreme tip must be accepted unless a new tip is to be used for each test. A maximum wear-deformation of 4 mm is therefore accepted, provided that it is symmetric, but no further significant change in shape.

Corresponding demands are valid for tips with the alternative non-standardised filter location in the conical part of the tip. These tips should be designed with cylindrical extensions of 10 mm for new tips and the filters should be located half way up on the conical part.

Preferably, the probes should make it possible to measure simultaneously the pore pressures both at the conical part of the tip and above the cylindrical extension at the base of the tip. While waiting for such probes to be commonly available on the market, the standard tips should be interchangeable with tips with the filter located half way up on the conical part. If required, parallel or alternate tests can then be performed with the other filter location.

The diameter of the cylindrical part of the tip shall be within  $35.7 \text{ mm} + 0.3 \text{ mm} / -0.9 \text{ mm}$  in all CPT tests and within  $35.7 \pm 0.3 \text{ mm}$  in tests with pore pressure measurements in test classes CPT2 and CPT3.



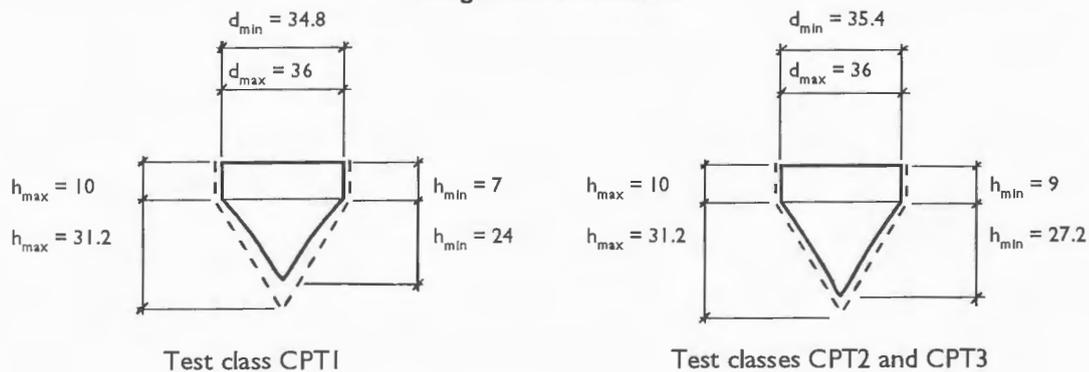
TIP FOR TESTS WITHOUT PORE  
PRESSURE MEASUREMENT

Porous filter

Slot filter

TIP FOR TESTS WITH PORE PRESSURE MEASUREMENT

### Design and dimensions



Test class CPT1

Test classes CPT2 and CPT3

TOLERANCES IN DIFFERENT TEST CLASSES

Fig.7. Design, dimensions and tolerances of the conical tip.

### Filter

The diameter of the filter shall be equal to the diameter of the tip. It may be slightly larger but absolutely not smaller. Permissible tolerances are shown in Figure 8.

The filter should fulfil these demands also after the test. It is normally used in one test only and is exchanged after each test.

The height of the filter is normally 5 mm. Filters with a lower height may be used, provided that an extra ring is used to fulfil the requirements on the dimensions of the tip.

The filters shall have fine pores, be incompressible and resistant against abrasion. Normally, filters made of sintered stainless steel or bronze with pore sizes of 2 - 20  $\mu\text{m}$  are recommended. Also ceramic filters may be used, but these are more brittle and the risk of damage is greater, especially when they are located on the conical part of the tip. Filters made of porous plastic are sometimes used. These filters can only be used at the standard location at the base of the tip. To fulfil the tolerances, they should be cast in a mould and not formed on a lathe.

There are now also probes on the market in which the filters have been replaced by a thin slot filled with a viscous fluid or a gel. Comparative tests have shown that this leads to certain limitations in measuring accuracy and that it may be difficult to fulfil the set demands. This mainly refers to tests in which the pore

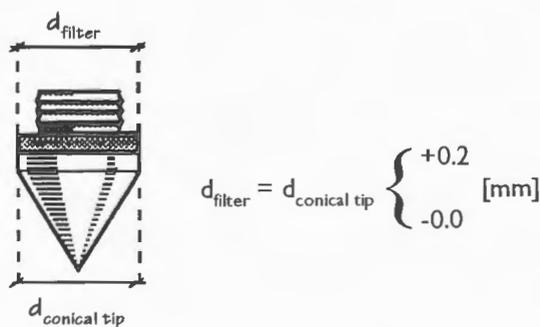
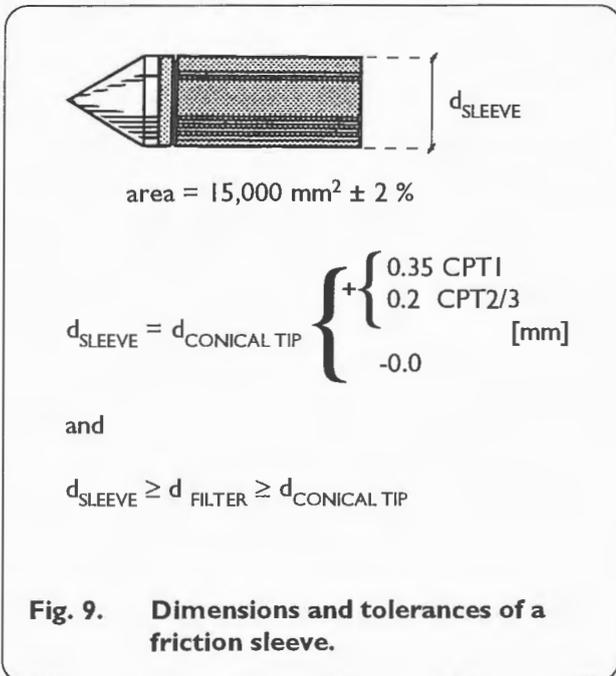


Fig. 8. Tolerances for the diameter of the filter.

pressure measurements are to be used for a detailed estimation of the soil properties and for measurement of pore pressure dissipation and equalized in situ pore pressures. For correction of cone resistance and sleeve friction, the method is normally quite satisfactory. When this method is used, the slot shall be located within the interval for the normal filter position, i.e. within 5 to 10 mm above the conical part of the tip.

**Friction sleeve**

The friction sleeve shall have a surface area of 15.000 mm<sup>2</sup> ± 2 %, which gives a length of about 133.7 mm. Dimensions and tolerances for the friction sleeve are shown in *Figure 9*. It shall be located directly above the tip and the filter. The maximum distance because of slots and seals is 5 mm.



The diameter of the friction sleeve shall be equal to or slightly larger than the diameter of the underlying parts. With a normal location of the filter between the tip and the friction sleeve, the generated pore pressure may be affected by the diameter of the friction sleeve and in this case the following criteria should be applied:

$$d_{\text{FRICTION SLEEVE}} \geq d_{\text{FILTER}} \geq d_{\text{CONICAL TIP}}$$

$$d_{\text{FRICTION SLEEVE}} = d_{\text{CONICAL TIP}} \begin{array}{l} +0.2 \text{ mm} \\ -0 \end{array}$$

If the pore pressures are measured at the conical face of the tip, they are not affected by the friction sleeve. The same criteria as for CPT tests without pore

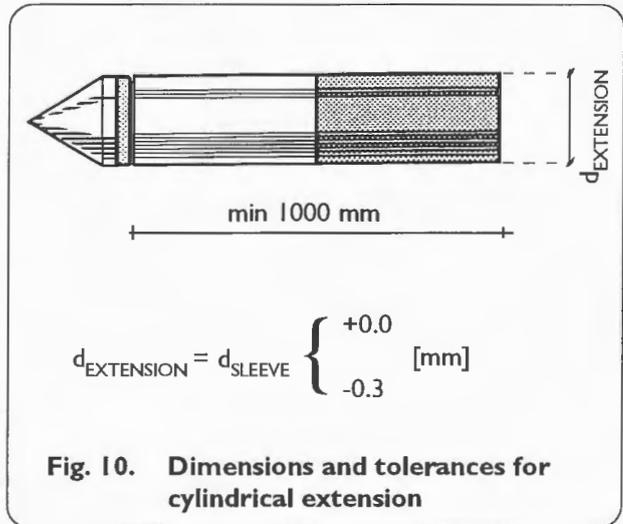
pressure measurement can then be applied

$$d_{\text{FRICTION SLEEVE}} = d_{\text{CONICAL TIP}} \begin{array}{l} +0.35 \text{ mm} \\ -0 \end{array}$$

The surface roughness *r* shall be within the limits 0.25µm < *r* < 0.75µm.

**Cylindrical extension**

The cylindrical extension shall have the same diameter as the friction sleeve with a tolerance of + 0 mm/-0.3 mm and together with the friction sleeve have a combined length of at least 1000 mm, *Figure 10*.

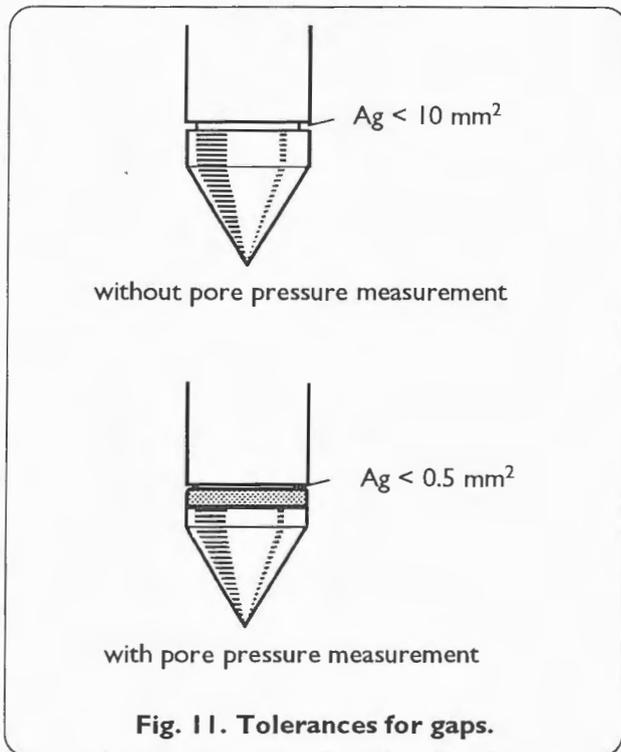


**Slots and seals**

Slots between the different parts of the probe must not be higher than 5 mm. Seals in these slots should be designed in such a way that they prevent soil particles from intruding. The seals must be so compressible in relation to the measuring element that no significant forces are transferred and create measuring errors. The part of the cross-sectional area of a gap which is not occupied by the seal must not exceed 10 mm<sup>2</sup> in tests without pore pressure measurement. In tests with pore pressure measurement and normal filter location, the corresponding area must not exceed 0.5 mm<sup>2</sup>, *Figures 6 and 11*. The latter demand is based on the fact that possible gaps in these slots create changes in the generated pore pressures which affect both the measured pore pressures and the corrections for pore pressure effects.

**Friction reducer**

It is common that a local enlargement of the sounding



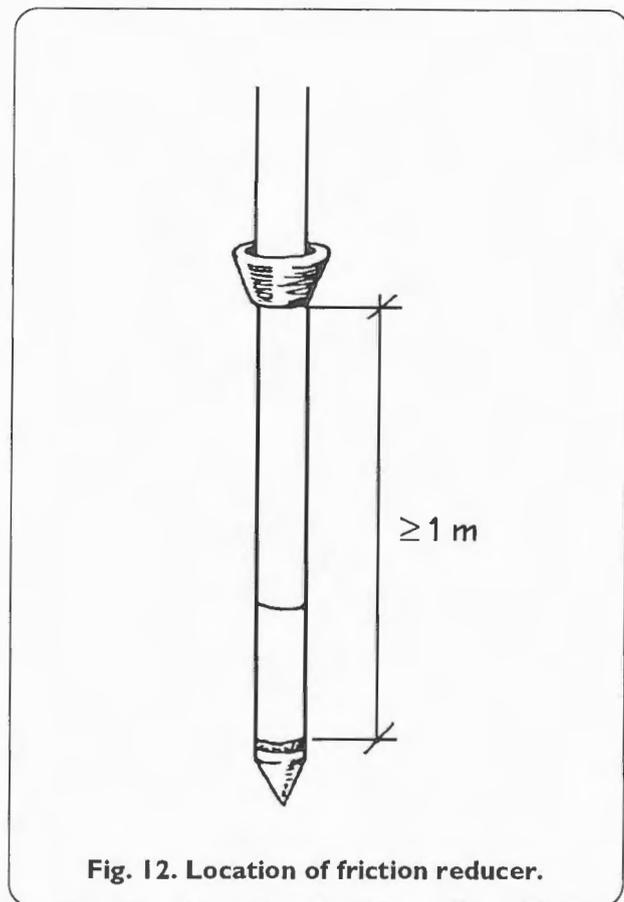
rod is applied at the lower part of the first sounding rod or at the joint between the CPT-probe and the sounding rods. The purpose of this enlargement is to reduce the friction against the following sounding rods and thereby the total penetration force. Such an enlargement is only allowed above the 1000 mm long part consisting of friction sleeve and cylindrical extension where the cross-section is standardised, *Figure 12*.

In order to further reduce the rod friction, bentonite slurry can be pumped down through the sounding rods and let out just above the friction reducer. This normally requires a combination of hollow sounding rods and cable-free signal transmission.

### 3.22 Pushing equipment

The pushing equipment shall have a stroke of at least 1 metre. It shall push down the probe and the rods at a constant specified rate. The equipment shall be ballasted or anchored in such a way that it does not move relative to the soil during the penetration and provides the required pushing force. Penetration by blows or rotation is not allowed.

The penetration shall proceed vertically and continuously and the measurements shall be taken during ongoing penetration at a constant rate. Stops in the penetration shall only be made for the addition of new sounding rods and re clutching of the pushing equipment. In connection with this, or at pre-selected levels,



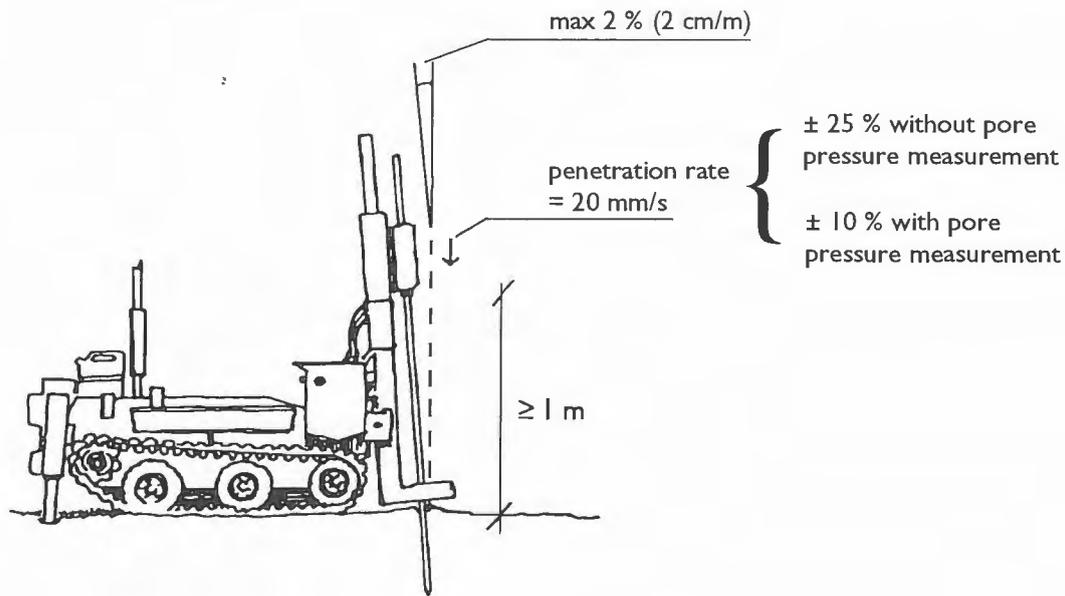
longer stops may be made for studies of the dissipation of the generated pore pressure with time. However, it is recommended that a continuous CPT test with as few and short stops as possible be carried out first and that possible studies of pore pressure dissipation with longer durations be made in subsequent supplementary tests. This does not apply to short stops in permeable layers where the pore pressure equalisation is rapid.

The pushing equipment shall be aligned as vertically as possible. The deviation from the vertical direction must not exceed 2 %. The axis of the sounding rods shall coincide with the direction of the applied pushing force.

The rate of penetration shall be 20 mm/s. The tolerances for tests without pore pressure measurements are 5 mm/s according to the recommendations by the Swedish Geotechnical Society and ISSFME. For tests with pore pressure measurements, the penetration rate shall be within  $20 \pm 2$  mm/s, *Figure 13*.

### Sounding rods

The sounding rods are selected with regard to the required pushing force and signal transmission system for measured data. Signal transmission by cable requires hollow rods (tubes) and joints. Further demands



**Fig. 13. Demands and tolerances for verticality and penetration rate.**

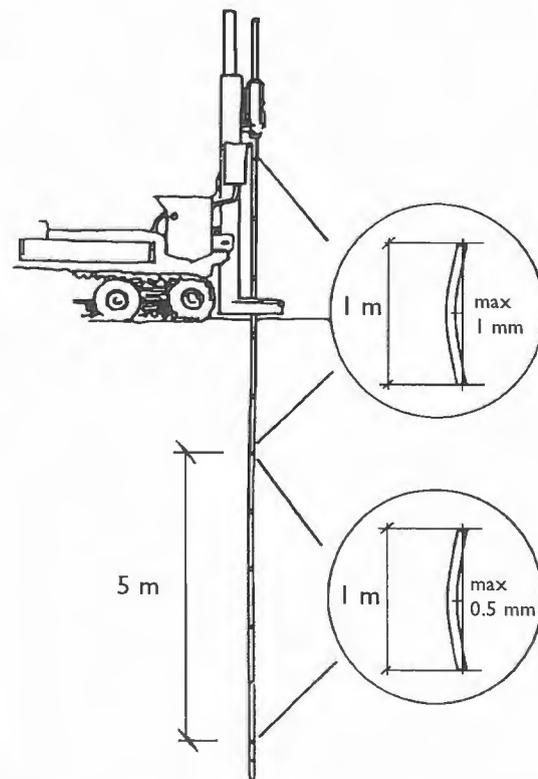
are that the joints shall be rigid and the rods shall be straight. For the lowest 5 metres, the maximum deflection at the middle of a 1 m long rod must not exceed 0.5 mm in relation to a straight line between the ends of the rod. The corresponding limit for rods higher up in the chain is 1 mm. The same demands on straightness also apply to the joints, *Figure 14*.

### 3.23 Measuring equipment

#### *Transducers, measuring instruments and data collection*

Forces and pressures shall be measured with suitable gauges or transducers and the signals shall be transferred to registering instruments and data collection units by a suitable method. There are three major alternatives on the market: (*Figure 15*)

- signal transmission by cable from the gauges/transducers to measuring instruments /data loggers on the ground surface
- acoustic (cable free) signal transmission through the sounding rods to measuring instruments /data loggers on the ground surface
- storage of the signals in an electronic memory



**Fig. 14. Demands on straightness of the sounding rods.**

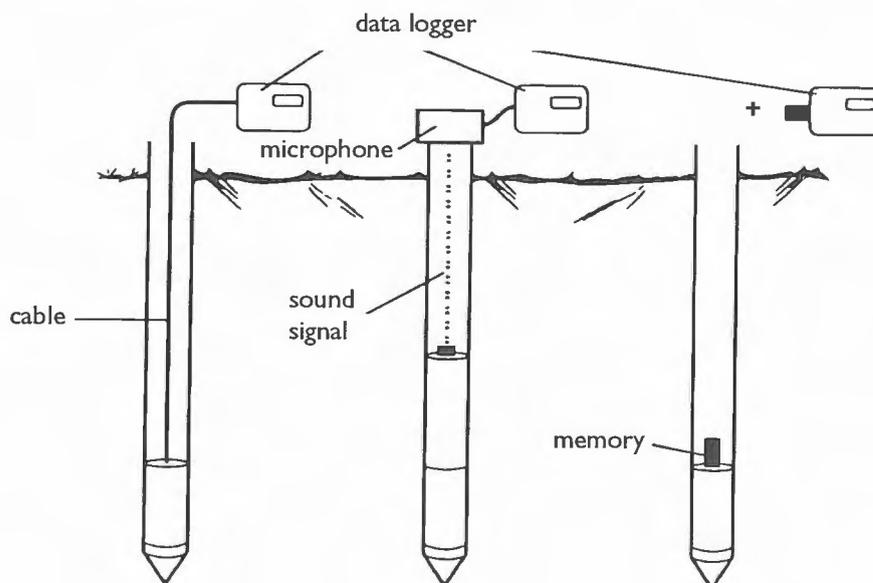


Fig. 15. Collection of CPT data in the field.

inside the probe, which is emptied into the data collection unit after the probe has been retracted to the ground surface

Methods of data registration which allow the values to be observed during the penetration process are recommended by the Swedish Geotechnical Society and ISSMFE, the main reason being a desire to check progress during the penetration operation. It is then possible to ascertain continuously that everything is working properly and to modify the method and the extent of the investigations on the basis of the obtained results.

The more checking and evaluation that is to be performed in the field, the more comprehensive will be the equipment and the demands on a working environment suited for electronic equipment. The possibilities also vary widely between different equipment. The extent of the required checking and evaluation of the test results in the field should be considered from case to case and the cost should be compared to possible extra costs for a later supplementary investigation. Direct evaluation in the field also offers a certain time saving. The development of equipment for data collection and data transmission is very fast and today it is possible for example to transfer the test data directly to the office during the penetration process via a telephone modem. It is therefore difficult to give specific

recommendations for the most suitable system for collection and registration of the test data. Examples of systems for collection and transfer of the test data are shown in *Figure 16*.

Pore pressure measurements shall be performed with high precision transducers having a very small volumetric deformation. The cavities in the tip should be designed in such a way that the total volume is as small as possible and that complete saturation at assembly is facilitated. Cavities, filter and channels shall be saturated with incompressible and deaerated fluids. Based on Swedish experience, vacuum treated glycerine or boiled water is recommended.

The purpose is accurate measurement of the rapidly varying water pressure in the surrounding soil with a minimum of required water transport in and out of the measuring system because of the pressure changes. These properties of the measuring system should also as far as possible be maintained in passing through layers where negative pore pressures and tendencies for sucking the fluid out of the measuring system are created.

The pore pressure measurements are normally made by transducers measuring absolute pressure, which entails that they may be affected by the barometric pressure. Normally, this is significant only in very time-consuming tests with long stops for measurement of pore pressure dissipation processes.

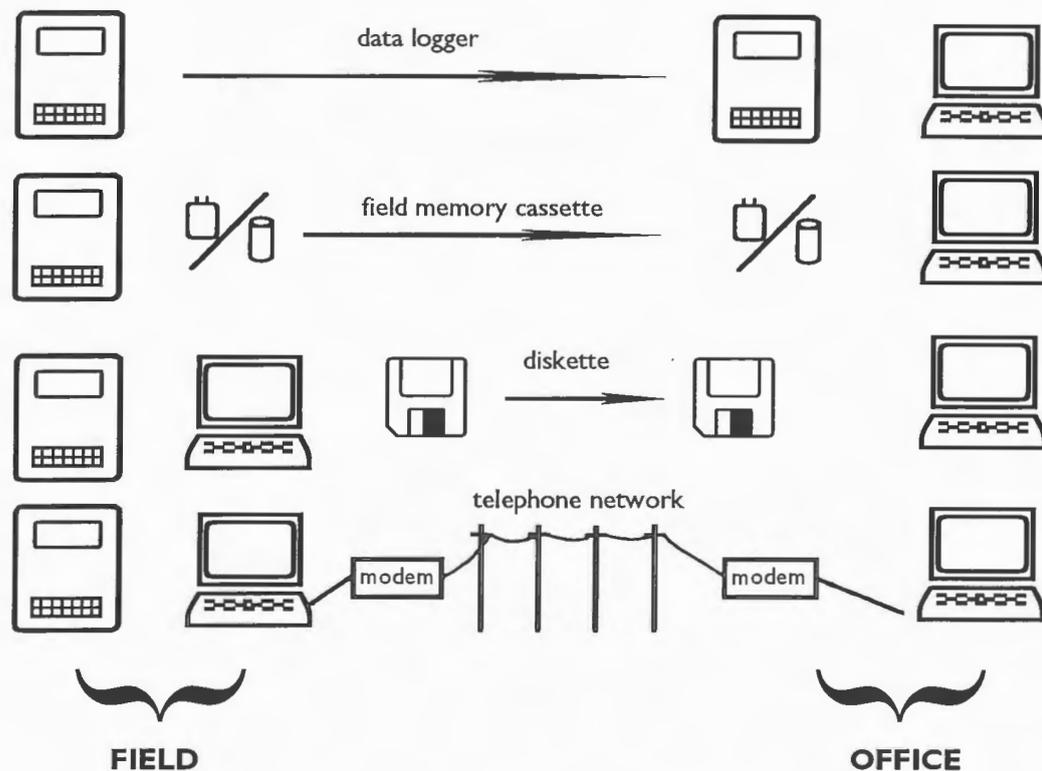


Fig. 16. Systems for collection and transfer of CPT test data.

### Reading intervals

The intervals between the readings of the different parameters may vary and are selected with regard to the soil volumes that affect the different parameters. Typical reading intervals for cone resistance and sleeve friction are 0.05 m. There is no obvious reason for a higher frequency since these values in principle represent averages for depth intervals with thicknesses between 0.13 and 0.7 m. For tests in class CPT1, sparser readings of these parameters can be accepted and are then made at least at every 0.2 m.

For pore pressure measurements, it is desirable to make readings at intervals corresponding to the height of the filter, i.e. about every 5 mm. This is often impracticable, but today the taking of readings at every 10 mm is possible with most common equipment. This frequency of readings should be aimed at, even if the standard today accepts readings at every 20 mm. If the same frequency of readings is used for cone resistance and sleeve friction, the latter readings may be averaged for depth intervals of 50 mm before they are stored.

Alternatively, the reading intervals can be varied in such a way that the latter parameters are only read at every 50 mm of penetration.

### Depth registration

According to the recommendations, the true level of the probe shall be known within  $\pm 0.1$  m. This demand may require a manual check to be made of the actual penetration depth and the recorded depth to be corrected accordingly. A manual check of the penetration depth shall always be made at the termination of the penetration. The resolution of the electronic depth registration should be at least 0.01 m.

### Temperature compensation

All measuring elements and electronics should be stable at temperature changes. When handling the equipment, precautions shall be taken in order to minimise the temperature variations, but certain changes cannot be avoided. Practical experience has shown that the stability against zero shifts that can be achieved in

ordinary 5-tonne probes is

- 2.0 kPa/°C for cone resistance
- 0.1 kPa/°C for sleeve friction
- 0.05-0.1 kPa/°C for pore pressure (depending on the measuring range for the transducer which is normally 1 - 2 MPa)

This stability is required and should be checked. For probes with higher measuring ranges, a proportional increase in the temperature sensitivity is accepted.

All errors due to temperature effects, together with other sources of error, shall be within the demands on total accuracy specified below. Since the temperature in most cases is not measured, the most rational way of ensuring that the demands for measuring accuracy are fulfilled is to have such good stability for temperature changes that all possible temperature effects can be accommodated within the demands.

#### *Measuring accuracy*

When all possible sources of error (internal friction, errors in the measuring equipment, eccentric loading of tip and friction sleeve, temperature effects etc.) are accounted for, the total error in the measured values must, according to the Swedish recommendations, not exceed

- 2 % of the typical value (the average value) in any of the soil layers which are to be classified and whose properties are to be evaluated \*)
- 1 % of the measured value of static pore pressure

However, these demands cannot be fulfilled at very small forces or pressures. The equipment and test class to be used are selected with regard to the type of soil to be investigated; the coarser and stiffer the soil, the more robust the probe and the lower the test class. This entails that there are certain practical minima for the pressures and forces that can be measured in the various test classes. For the different test classes, there are therefore lower limits representing inaccuracies which can be accepted generally:

\*) The term "soil layer" in this context refers to separate soil layers or, in the case of thick homogeneous layers, depth intervals of one metre.

Test class	Cone resistance	Sleeve friction	Pore pressure
CPT1	100 kPa	10 kPa	10 kPa
CPT2	40 kPa	4 kPa	5 kPa
CPT3	20 kPa	2 kPa	1 kPa

This accuracy must be verified by regular calibration of the particular equipment.

(The standard recommended by ISSMFE states that the accuracy shall not be less than 5 % of the measured value or 1 % of the maximum measured value in the particular soil layer. This is difficult to interpret and requires clarification).

In all test classes, probes of high quality are required. In test class CPT1, 5 - 20 tonne probes are used. In test class CPT2, 5-tonne probes can be used and in test class CPT3 either probes with lower measuring ranges or specially calibrated 5-tonne probes are used.

### **3.24 Special arrangements**

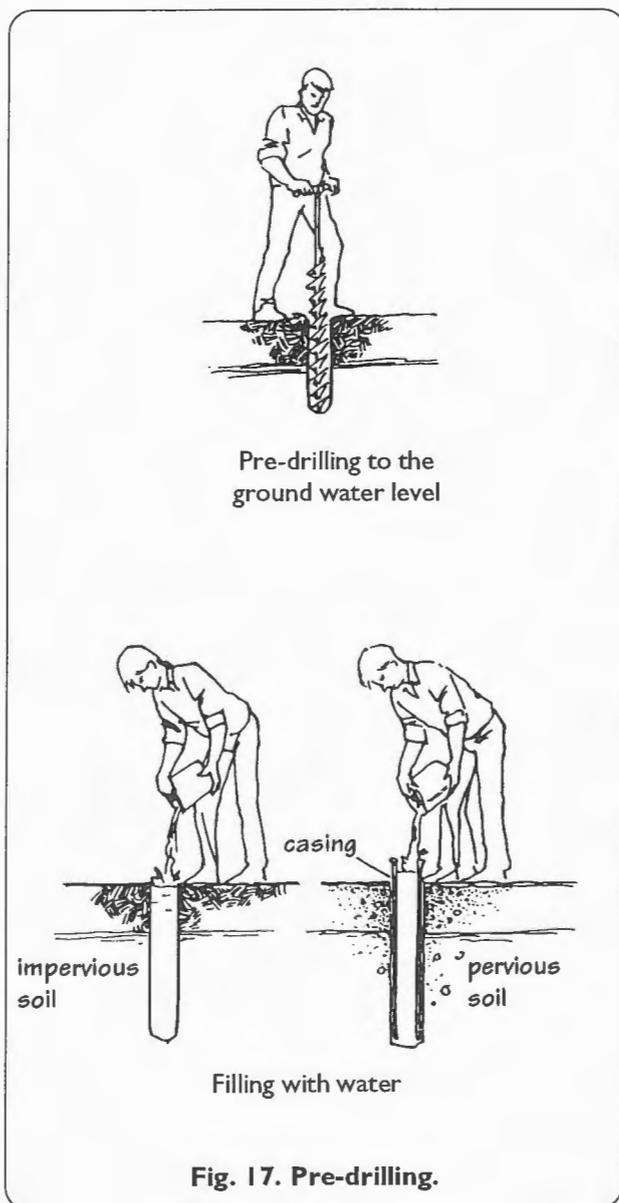
#### *Pre-drilling*

In tests with pore pressure measurement in soft soils, passage through the dry crust brings special problems. The dry crust is normally not saturated and often large negative pore pressure changes occur. Both these facts entail a risk of loss of saturation in the probe and the measured pore pressures in lower layers becoming non-relevant. In addition, large cone resistances and sleeve frictions occur in the stiff dry crust. This leads to a risk of hysteresis effects and zero shifts, which in turn lead to lower measuring accuracy for cone resistance and sleeve friction in the underlying softer layers. Furthermore, there is an increased risk of stiffer particles being pressed into the slots between the different parts of the probe and thus affecting the results of the measurements.

In CPT testing in soft soils, pre-drilling through the dry crust should therefore be performed after an initial test has provided information on its thickness and stiffness. The pre-drilled hole is normally filled with water and on most occasions a casing is required, partly to keep the hole open and partly to retain the water level, *Figure 17*.

Fills with infusions of coarser material are always pre-drilled.

In some cases, the pre-drilling may be replaced by punching a non-instrumented dummy probe with a diameter of 45 - 50 mm through the stiffer layers.



**Fig. 17. Pre-drilling.**

### *Guidance of rods*

When required, the sounding rods above ground level and in water shall be provided with guide rollers, casings or other arrangements to prevent them from buckling out.

### *Inclinometer*

In order to provide information on the verticality of the probe and possible declination from the vertical direction, the probe can be fitted with an inclinometer. The need for this information will depend on the soil conditions and the type of investigation, and increases with penetration depth. Tests in homogeneous soils can normally be performed to depths of 15 - 20 m without any problems associated with deviation. In inhomogeneous soils and tests adjacent to excavations or struc-

tures, the corresponding depth is less than 10 m. The inclinometer can also be connected to a stop function for the pushing equipment. The risk of damage to the probe because of bending is normally greater than the risk due to vertical overloading.

### *Overload protection*

The measuring elements in the probe are designed for maximum measuring ranges, which should not be exceeded. The sounding rods and their joints have a corresponding maximum load capacity. When using normal CPT-probes with at least 5-tonnes axial load capacity and lightweight pushing equipment, this is normally not a problem because the pushing capacity is seldom larger. The measuring range of the friction sleeve is normally adapted in such a way that there is no great risk of it being overloaded before the cone tip.

Pore pressure transducers are often selected in such a way that a large part of the measuring range is utilised. A simple check that the measuring range is sufficient is that it shall be larger than the in situ pore pressure at maximum test depth + 20 times the maximum expected undrained shear strength. This rule accommodates all possible filter locations. With the normal filter location, it is sufficient if the measuring range is larger than the prevailing in situ pore pressure + 15 times the expected undrained shear strength. In overconsolidated soils and especially with a normal filter location, the penetration pore pressures will be considerably less than these values.

The sounding rods are normally sounding rods with diameters of 32 or 36 mm, or alternatively hollow 36 mm rods or rods for standard piston sampling. Also for these rods, the load capacity is normally no problem when using lightweight pushing equipment.

When using special probes with lower load capacities or when seeking to ensure that a certain calibrated loading range is not exceeded, load restrictions should be applied to the pushing equipment. This is also the case when heavier pushing equipment with higher load capacity than the probes or rods is used.

The pushing equipment is normally hydraulically operated and the simplest form of load restriction is an adjustable safety valve for the oil pressure. The regulation of the valve is calibrated in such a way that a scale for the maximum pushing force is obtained. However, the total pushing force is the sum of the cone resistance and the friction along the probe and the rods, and the friction can often constitute the major part of the load. The regulation may therefore have to be

supplied with an extra scale showing how much the load may be increased depending on the cone resistance that is read off when the set maximum total load is exceeded.

The greatest risk of damage to the probes occurs at penetration in soft soils and stop against firm bottom, especially against inclined bedrock and in soil containing stones or boulders. There is then a pronounced risk of bending and buckling of the probe or the rods. This risk can be minimised by the introduction of overload protection of the type outlined above.

A more highly developed system for overload protection can be introduced by using electronics. In this case, limits can be set for cone resistance, sleeve friction and pore pressure and preferably also for inclination. If any of these limits is exceeded, an electrically operated valve on the pushing equipment can be activated and the penetration is stopped instantly. The operator can then analyse the situation and decide whether the test should be continued or terminated.

#### *Temperature sensor*

The probe shall be handled in such a way that in all stages of preparation and performance of the test it is kept as close to the ground temperature as possible (in Sweden normally 7 - 8 °C). It is not possible to fulfil this demand completely, but when taking zero readings and during all the subsequent phases of the test, an effort should be made to keep the probe at a temperature within  $\pm 5$  °C of the desired level.

## CHAPTER 4.

# Performance of CPT test

### 4.1 PREPARATIONS

#### 4.1.1 Checks

Before starting a CPT test project, the probe shall be calibrated and tips and friction sleeves shall be inspected. This also applies to tips and sleeves brought along as spares and possibly exchange tips with the filter located at the conical face. O-ring seals and dirt seals in the slots are inspected. All parts that are not up to the requirements are exchanged. All sounding rods and adapters that are to be brought to the field are also inspected.

If the signals are to be transmitted by cable, this is pre-threaded through the rods.

Operation of the electronics is checked in connection with calibration. Possible safety functions in the pushing equipment are checked as well as any extra equipment such as generators.

An estimation is made of the number of tests to be performed with pore pressure measurement and a corresponding number of filters are selected. These are checked individually by being assembled together with the tip to be used in the same test. The filters shall be easy to mount, have the right diameter and after the tip has been screwed on the filters shall be easy to rotate with the fingertips without being too loose. (The demand for easy rotation does not apply to filters located on the conical face of the tip.) Possible sharp edges from manufacture can be removed on a lathe or by file and rubbing with emery. This first check of the filters is made with dry filters and those which do not fulfil the requirements are repaired or rejected. After saturation, it is too late to do anything about them.

A checklist to be used before and during a CPT test project is shown in *Table 2*.

This checklist should be used together with the manufacturer's instructions for the specific equipment in order to obtain good quality assurance.

The checks listed under "Start of project" are

normally made in the workshop, store and/or laboratory before the equipment is taken out to the field.

A more detailed clarification of the various points in the checklist is given below:

#### *Straightness of sounding rods*

Before the test, the straightness of primarily the lower five rods is checked. The check is made before each test and special attention is paid after tests in soft soils with stop in penetration against bedrock or other firm bottom, especially when the latter is inclined, and in soils containing stones or other large objects.

#### *Wear*

The wear and surface roughness of the tip and the friction sleeve are checked before each test and a check is also made of the relations between the diameters of the tip, the filter and the friction sleeve. The much stricter demands on tests with pore pressure measurements in soft, fine-grained soils than in tests without pore pressure measurement in sand must be observed.

#### *Seals*

The condition of the seals between the different parts of the probe shall be checked before each test. Possible soil particles which have intruded into slots and seals are removed. No significant wear of the seals can be accepted in the higher test classes.

#### *Calibration check and function check*

A calibration check of the measuring system shall be made before each CPT test project and regularly during test programmes of long duration; at least every three months. Simpler function checks can be performed at the site.

**Calibration check** refers to a check of the calibration of cone resistance, sleeve friction and pore pressure against force transducers and pressure transducers respectively, which shall have such a precision that

**Table 2. Checklist for a CPT test project.**

Check	Time				
	Start of project	During project	Start of test	After test	At least every 3 months
Sounding rods	●		●		
Seals	●		●		
Filter fit	●		●		
Wear and tolerances	●		●	●	
Calibration check	●				●
Safety functions	●				●
Function check		●			
Zero shift		●	●	●	
Verticality of pushing equipment			●		
Rate of penetration			●		

it can be verified that the probe fulfils the requirements in the various test classes. In this check, adapters are used to ensure that the different parameters can be measured in a controlled way and without causing damage to the equipment. The check can be performed at any place where sufficiently accurate reference equipment is available. In the calibration check, the related electronics and data logging systems to be used in the field shall be used in order to check the whole system and its sources of errors. During application of force on the tip and friction sleeve respectively, also the other two parameters are read off in order to check that there is no interference between the measured parameters. The date and the values measured at calibration are reported in a ledger which is to be kept together with the CPT test equipment.

**Function check** refers to a simple check of that the

electronics and measuring elements are functioning and yield signals that are of the right level. However, this simple check is not intended to be used for changes of any calibration factors. The function check can be performed by checking the tip and friction forces against a pressure cell or by application of a known load. Also in this case, adapters are required to prevent damage to the equipment. Sensitive pore pressure transducers can be controlled by lowering the probe into a water filled hole, for example when pre-drilling through the dry crust. During the whole testing programme, the zero readings of the measuring elements are checked before and after each test and they shall remain stable.

In the event of major damage to the probe, a new detailed calibration of the cone shall be made after repair and possible exchange of measuring elements.

### *Distance to adjacent tests and boreholes*

The distance to adjacent test points should be at least 2 metres. In cases where the tests are supplemented by taking disturbed samples, the CPT test is performed first and the sampling depths are selected with guidance from the test results.

### *Pushing equipment*

The verticality of the pushing equipment is checked at each set-up at a new test point. Possible safety arrangements for maximum load shall be calibrated. The rate of penetration is checked before each test.

### *Filter height*

After saturation and assembly of the probe, the fit of the filter is checked once again. Its height shall be such that there is no play and at the same time the filter shall be easy to rotate with the fingertips. This check ensures that there are no unnecessary slots and that there are no forces built in at assembly which can affect the measurements, *Figure 18*.

#### **4.12 Saturation of filter and treatment of fluids**

After the first check, the filters are saturated. In cases where there is a risk of encountering soil layers in which negative pore pressures develop, for example dense sand and silt or overconsolidated clay, non-saturated soil or dry crust without pre-drilling, glycerine is chosen. In other tests, water may be used as an alternative.

When using glycerine, the dry filters are lowered into the fluid and treated by highvacuum for a couple of hours. *Figure 19*. A larger quantity of glycerine is treated in the same way. Filters and fluids are then kept in airtight containers.

When using water, the filters are boiled for at least 15 minutes. Filters and boiled water are then allowed to cool under airtight lids and are then kept in well filled airtight containers. In addition, a larger quantity of water is deaerated, for instance by using a water jet. Also this water is kept in an airtight container.

Saturation of filters and fluids is normally performed in a store room or laboratory.

#### **4.13 Further details**

The CPT equipment includes mounting equipment for the filters, pre-drilling equipment and casings, possible cables, generator and fuel and/or batteries for power supply, registration papers, data cassettes or disks and

test records, and also cleaning equipment and vaseline for the probes. In areas where artesian water pressures may occur, equipment for sealing the test holes should also be taken to the site.

For further details and checks reference is made to the manufacturer's manuals for the particular equipment used.

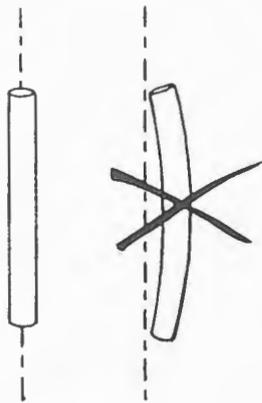
## **4.2 TEST PERFORMANCE**

- **The pushing equipment** is set up at the test point and aligned vertically. If required, it is anchored or ballasted. Pre-drilling is performed if necessary. Normally, the pre-drilled hole is filled with water and when required a casing is used to retain the water and keep the hole open. Pre-drilled and water-filled holes can be required for adaptation of the probe to ground temperature also in cases when the tests are to be performed from the ground surface. These holes are then placed a few metres away from the test point.
- **The penetration rate** is set and checked together with possible load restrictions.
- **The measuring system** is started and allowed to warm up during assembly of the probe. Instructions for handling the measuring systems are specific for each type of system, and more detailed guidelines must therefore be sought in the manuals from the respective manufacturer.

The probe shall at all times be kept and assembled in such a way that extreme temperatures are avoided. It shall thus be protected against direct sunlight, heat and cold. The same applies to filters and fluids.

- In the most common method of **mounting filters with glycerine**, the probe is turned upside-down and the tip is unscrewed. An average size plastic funnel is threaded on the probe with its spout downwards. The spout shall be cut off in such a way that it fits the diameter of the probe. A piece of rubber tube is pre-threaded on the spout. When the funnel is in place, the rubber tube is rolled down to seal against the probe (the friction sleeve) whereby the funnel becomes fixed. Special care must then be taken because the friction sleeve is not locked in place until assembly is complete, *Figure 20*.

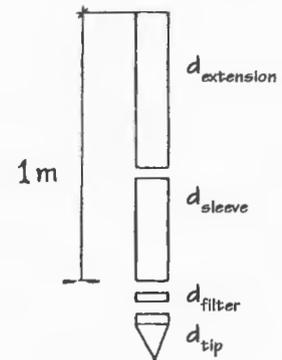
Deaerated glycerine is carefully poured into the funnel. A syringe is used to remove all the air bubbles in the cavities in the tip and the probe, in channels and threads and on seals and other loose parts. The filter is carefully transferred from its container to the funnel and all parts are assembled below the surface of the fluid. (The containers for filter and glycerine are sealed



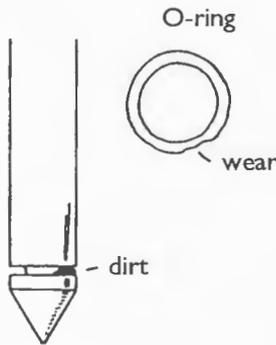
Straightness of sounding rods



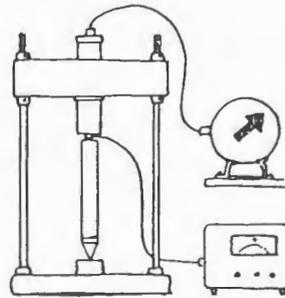
Wear of the conical tip



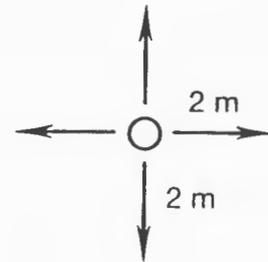
Relation between different parts



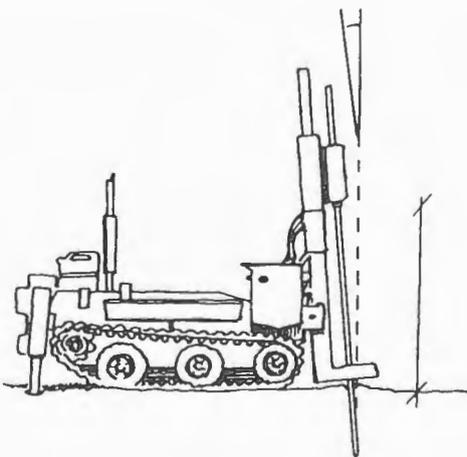
O-rings and dirt seals



Calibration



Distance to adjacent test points/boreholes



Check of pushing equipment



Filter fit

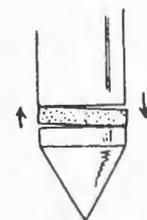


Fig. 18. General checks before a CPT test.

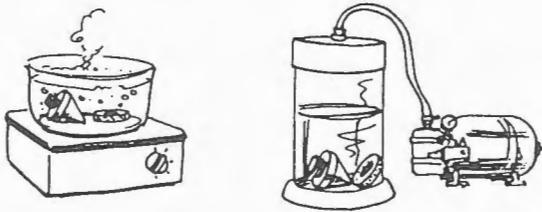
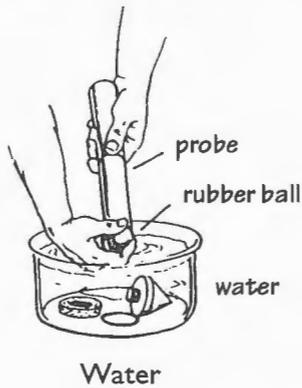
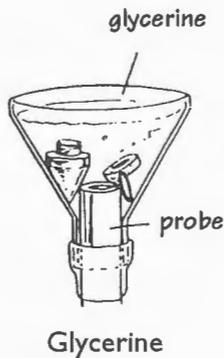


Fig. 19. Saturation of filters.



Water



Glycerine

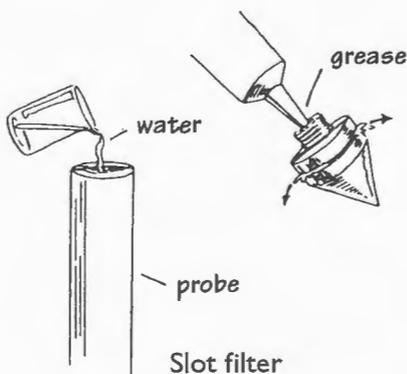


Fig. 20. Mounting a filter.

directly after filling with glycerine and transfer of the filter respectively.) After the tip has been screwed on, the fit of the filter is checked once again so that there is no play and that the filter can be easily rotated with the fingertips. The glycerine in the funnel is then poured off, the rubber tube is rolled up on the spout and the funnel is removed. The probe is then immediately transferred to the test point.

- Tests from the ground surface or in pre-drilled but not water-filled holes are then started without unnecessary delay. In water-filled holes, the probe can be allowed to **adjust to ground temperature** for a certain period of time before the test is started. When the probe is fitted with a temperature sensor, this is recorded and a check is made that the temperature has stabilised and is within acceptable limits. Otherwise, the time for stabilisation is selected with guidance from the time it took for the particular probe to stabilise in the temperature calibration and by reading the various measuring elements and checking that the readings are stable. If the test is to start from the ground surface but the probe needs to be stabilised for ground temperature in a pre-drilled hole, the lower part of the probe should be enclosed in a plastic bag filled with glycerine during the stabilisation, *Figure 21*.

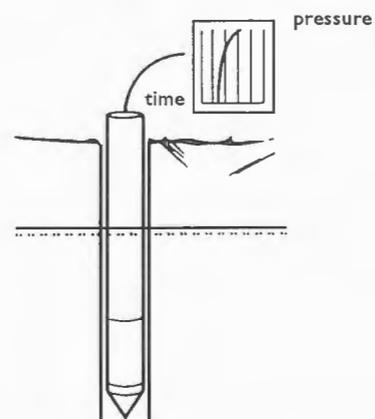


Fig. 21. Adaptation to ground temperature.

- When using **water** in the probe and filter, different methods can be used at assembly. The method used at SGI is to unscrew the tip and put it in a bowl with deaerated water together with seals and other loose parts. These are carefully saturated and possible air bubbles are removed by using a syringe. The filter is then transferred to the bowl. The containers for water and filters are sealed without unnecessary delay. The probe is turned upside-down and its cavities and

channels are saturated and then sealed by a rubber ball being pressed against the lower end of the probe. In this condition, the probe is turned and its lower part is lowered below the water surface in the bowl. The rubber ball is taken away and the cone is assembled with its lower part under water, *Figure 20*.

The lower part of the probe is kept under water. First, the fit of the filter is checked and then a plastic bag is filled with water and threaded onto the lower part of the cone. The probe is transferred to the pre-drilled and water-filled hole with its lower end inside the water-filled bag. When the tip is well below the water surface in the hole, the bag is pulled upwards and torn apart. The entire assembly and transfer operation is thus performed under water and there is no possibility of air entering the system.

This method can also be used for glycerine but then there is no need for a plastic bag after assembly.

Saturation and assembly are often the critical phase which decides whether the result of the test is satisfactory. Very great care must therefore be taken in every step; deaeration of filters and fluids, transport and storage, assembly in the field and use only of deaerated fluids kept in airtight containers which are carefully sealed directly after use.

Assembly in the field is often difficult and demanding, especially in bad weather and lighting conditions. Special aids in the form of mounting cylinders, which can be threaded onto the probe and in which a vacuum can be applied to the filter, fluid and the lower part of the probe at various stages of assembly are being tried out but are not yet in general use.

- Another method, which is intended to make the procedure easier, is to replace the porous filter by a thin slot. Here too, a careful saturation is required. This is achieved in the following way. The conical tip is unscrewed and the spout of a grease tube is inserted in the hole at the back of the tip. Grease is pressed in until it fills all the cavities in the tip. The spout is retracted while grease is still being pressed out of the tube so that the tip is completely filled. The probe is turned upside-down and the cavity at the pore pressure transducer is filled with deaerated water, whereupon the tip is screwed on. The surplus grease, which is pressed out when the tip is screwed on, is wiped off and the probe is ready for use, *Figure 20*.

As previously mentioned, this method leads to extra uncertainty in the measurements. On the other hand, assembly and handling are facilitated and the possibility of maintaining saturation in stiff and non-saturated

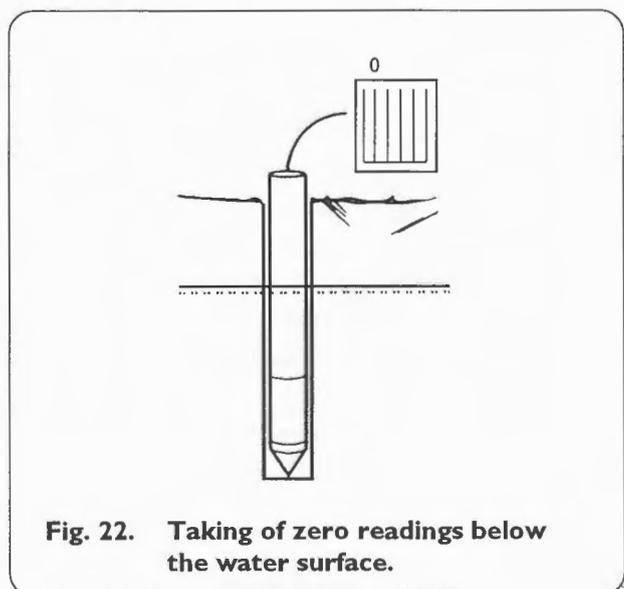
soil is relatively good. The usefulness of the method must therefore be judged from case to case with regard to the requirements on accuracy and the particular soil profile.

When planning a CPT testing project, it must be remembered that the preparations before the test, especially when pore pressures are to be measured, normally take longer than the test itself. The exceptions are very deep tests and tests with interruptions for various special tests.

- The temperature of the probe when **taking zero readings** should preferably be equal to the ground temperature, which in Sweden averages 7-8 °C; a couple of degrees higher in the southern part and 3-4 °C in the northern parts of the country. For practical reasons, the temperature of the probe must be allowed to deviate about  $\pm 5$  °C from the guiding values for probes with good temperature stability.

After stabilisation of the probe temperature (and allowing time for warming up the electronics in the probe) the zero readings for the measuring elements before the test are taken. The probe should then be freely suspended. In case the zero readings are taken when the probe is submerged in a water-filled hole, a note has to be made of how far below the surface the tip is located at the zero reading. In a corresponding way, the level of the tip below the ground surface when taking the zero reading of the depth recorder is noted, *Figure 22*.

- The **penetration** is performed at a constant rate of 20 mm/s. Stops for adding of new sounding rods and re-clutching the pushing equipment are performed without unnecessary delay.



**Fig. 22.** Taking of zero readings below the water surface.

When a stop in penetration occurs because of a pre-set restriction in pushing force, the cone resistance at the stop is read off (if possible). If it then appears that the load on the tip is far from a critical value and that the stop occurred because of excessive rod friction, the pushing force can be increased.

During penetration, the curves for cone resistance, sleeve friction and pore pressure versus depth are studied in order to ensure that everything is working properly.\*) A preliminary estimation of soil types can also be made, together with a judgement of possible problems and measures, to improve the quality of the results, (for example increased pre-drilling, alternative filter location or change of probe).

- Special tests with **interruptions for excess pore pressure dissipation** in different layers or seismic tests are normally carried out in separate CPT tests. For some equipment, there are reading routines where recording of the pore pressure dissipation starts automatically each time the penetration is interrupted. In these cases, shorter stops of 5 - 10 minutes may be included in the ordinary tests.

- In the subsequent evaluation of the test results, it is necessary to know the **in situ pore pressure**. It is therefore of great value if the equalised pore pressures can be read off in more permeable layers during the CPT test. Otherwise, supplementary pore pressure observations are required. All tests which are stopped at firm bottom or in coarse soil should therefore be concluded with a reading of the pore pressure versus time for a few minutes to ascertain whether the soil is so permeable that the pore pressure equalisation is rapid and a stable, equalised measure of the in situ pore pressure can be obtained. This can be done relatively easily even if such a reading routine is not provided for the particular equipment. Corresponding measurements of in situ pore pressures can also be made in other permeable layers.

- The **depth at stop of penetration** is also recorded manually, knowing the length of the probe, the length of the added sounding rods and the part of these remaining above the ground.

- The probe is then retracted. Directly after it has been taken up and before there is a change in the

temperature of the probe, the zero values after the test are read off. It should then be observed that large negative pore pressures are often created at retraction of the probe, and these remain in the pore pressure measuring system afterwards. **The zero readings after the test** should therefore be taken both directly after the probe has been retracted and the tip unscrewed so that possible negative pressures have been equalised.

Possible exceptional wear is checked and noted.

Before each new test, the filter is replaced with a new one and the probe saturated and assembled as described earlier. Slots and seals are inspected and cleaned.

- After the test, the **water level in the hole** is studied. When artesian water flows out of the hole, this must be sealed. This can be done by pushing a long casing into the hole and subsequently filling the hole with swelling bentonite pellets during retraction of the casing. When the flow of water has been stopped, the hole normally closes itself with time at greater depths. However, it is very essential to stop the water flow very quickly since the damage can otherwise be considerable.

In more normal ground water conditions, the free water level which develops in the holes after a time is studied and noted.

- Directly after the test, all the necessary information and observations made are noted in a record, *Figure 23*, and the test point is levelled and its coordinates measured. A summary of the different phases during a CPT test and after its completion is given in *Figure 24*.

- **Supplementary pore pressure measurements** by installation of filter tips should be started as soon as possible, preferably so early that they are stabilised within the time for the other tests in the investigation. They should be placed in such a way that they are unaffected by the other tests.

- Possible **sampling** in connection with the CPT tests is performed after the penetration tests.

- The probe shall be disassembled and cleaned **after each CPT testing project** and at the end of each working day. O-rings and seals are inspected and smeared with Vaseline and any moisture is dried out before the probe is reassembled. Drying may only take place at room temperature, not by heating.

The probe is subjected to a calibration check after each project. When the time interval between the projects is short, this calibration check can be included in the preparations prior to the next project.

\* Alternatively, this is performed directly after each test.

## RECORD OF CPT TEST

Test site \_\_\_\_\_ Date \_\_\_\_\_

Project No. \_\_\_\_\_ Test point No. \_\_\_\_\_

Operator \_\_\_\_\_

-----  
 Pre-drilling \_\_\_\_\_ m Free ground water level \_\_\_\_\_ m

-----  
 Probe type \_\_\_\_\_ Probe No. \_\_\_\_\_

Calibration date \_\_\_\_\_

Filter type \_\_\_\_\_ Fluid used \_\_\_\_\_

-----  
 Selected measuring range    cone resistance \_\_\_\_\_     kPa     MPa  
    pore pressure \_\_\_\_\_     kPa     MPa  
    sleeve friction \_\_\_\_\_     kPa     MPa

Scales                            cone resistance \_\_\_\_\_     kPa/cm     MPa/cm  
    pore pressure \_\_\_\_\_     kPa/cm     MPa/cm  
    sleeve friction \_\_\_\_\_     kPa/cm     MPa/cm

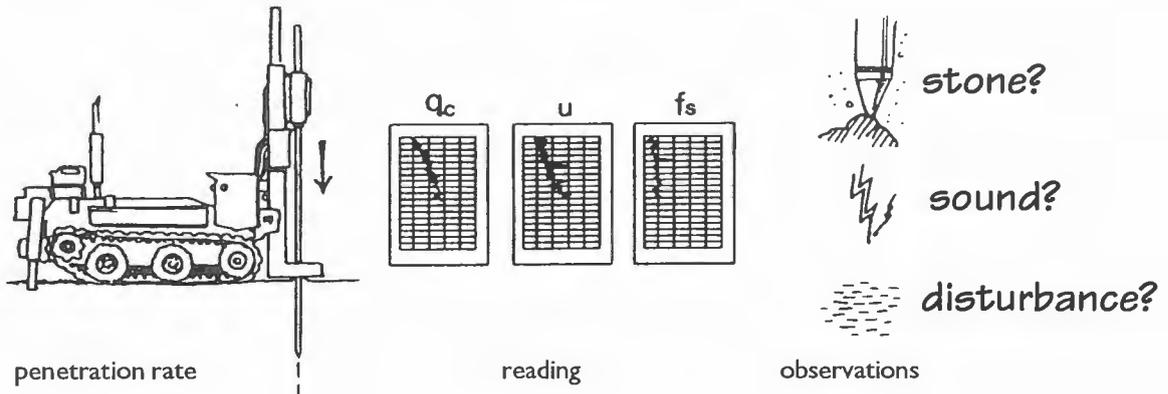
Zero reading                    before                                    after

   cone resistance \_\_\_\_\_    \_\_\_\_\_  
    pore pressure \_\_\_\_\_    \_\_\_\_\_  
    sleeve friction \_\_\_\_\_    \_\_\_\_\_

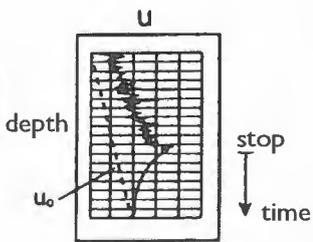
-----  
 Measured depth at stop of penetration \_\_\_\_\_ m

-----  
 Observations and remarks \_\_\_\_\_  
 \_\_\_\_\_  
 \_\_\_\_\_  
 \_\_\_\_\_

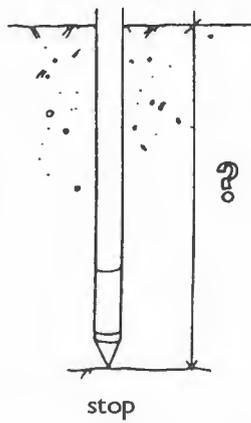
**Fig. 23. Example of a record of CPT tests.**



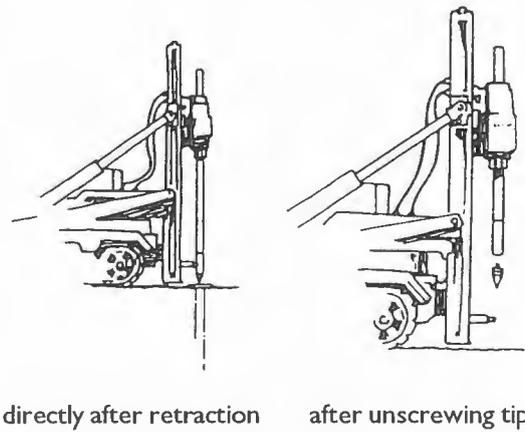
a) Penetration



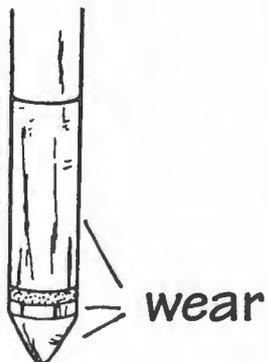
b) Equalisation of pore pressure at stop



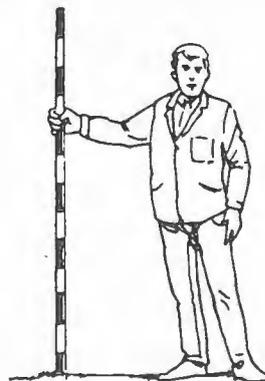
c) Manual check of penetration depth at stop



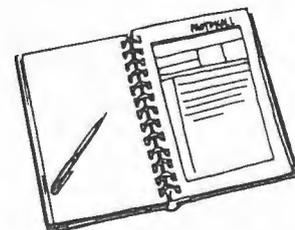
d) Zero reading (check after test)



e) Check of exceptional wear



f) Levelling/measurement of co-ordinates



g) Record

Fig. 24. Performance of a CPT test.

### 4.3 SPECIAL PROBLEMS

Problems in CPT tests on land are most often related to pore pressure measurements in stiff, non-saturated or overconsolidated soil. The difficulty here is to maintain the saturation in the pore pressure measuring system and to obtain relevant measurements of the pore pressure.

In cases where the problems are limited to the dry crust, they can be avoided by pre-drilling through the crust. It is then often advisable to start the test programme with a test from the ground surface using a probe with high load capacity. The results yield information on the stratigraphy, the penetration resistance and the thickness of the dry crust. The required depth of pre-drilling can then be determined on the basis of this information. If several probes with different measuring ranges are available, the most suitable probe can also be selected. When testing through the dry crust, it should be observed that in those cases where the saturation has been lost, the measured pore pressure in the underlying soil layers is not relevant. Measured values of about the right size are then often obtained at about 5 m below the free ground water level, but the response and the accuracy of the measurements remain in question.

In tests in very soft soils, pre-drilling often has to be performed in order to avoid the large forces that occur in the dry crust, which otherwise will create zero shifts and hysteresis effects, reducing the accuracy of the measurements in the underlying soft layers. When using very sensitive special probes and/or applying load restrictions for specially calibrated measuring ranges, the maximum permitted load is also often exceeded in the dry crust.

Problems with negative pore pressures at larger depths can be dealt with by a more elaborate saturation technique or possibly by using a slot filter with grease or alternatively by using tips with the filter placed on the conical face of the tip. With this filter location, the generated pore pressures are almost always positive and it is also often possible to obtain a more detailed picture of the stratification. On the other hand, the measured values of cone resistance and sleeve friction cannot then be corrected to fully relevant values. Tests with the filter placed on the conical face of the tip are therefore performed as supplements to tests with the normal configuration of the probe. Supplementary tests with the filter located on the conical face of the tip are of great value for evaluation of the soil properties also in fully saturated soils without problems with the

pore pressure measurements.

Tests in gyttja and organic clays can involve special problems. This type of soil often contains both shells and gases and has shown to be especially prone to clogging filters. In these soils, it is recommended that glycerine be used for saturation of filters and pore pressure measuring systems.

# Data processing and reporting of test results

## 5.1 PARAMETERS FOR EVALUATION

The basic parameters obtained from a CPT test are:

- Total cone resistance,  $q_T$
- Total sleeve friction,  $f_T$
- Total pore pressure,  $u$

In order to obtain the basic parameters  $q_T$  and  $f_T$ , the pore pressure has to be measured at the base of the conical tip between the tip and the friction sleeve and the measured values of cone resistance and sleeve friction corrected for this. In the literature, a number of empirical correlations have been suggested to enable the use also of pore pressures measured on the conical face of the tip for correction of cone resistance and sleeve friction. However, these can be grossly misleading and should not be used.

Since the pore pressure varies strongly depending on where along the probe it is measured, the designations  $u$  (and  $\Delta u$ ) should only be used for pore pressures measured at the normal filter position. The pore pressure measured half-way up on the conical face of the tip constitutes a fourth basic parameter which can be obtained from supplementary CPT tests and should be defined as

- Total pore pressure at the conical face of the tip,  $u_{FACE}$

For interpretation of the results, the following basic parameters are also required:

- Initial in situ pore pressure,  $u_0$
- Initial vertical stress in situ,  $\sigma_{v0}$  (calculated from the density of the soil)

The initial pore pressure is estimated from observations of the free ground water level and equalised pore pressures measured in more permeable layers at temporary stops in the penetration test. If the latter values are missing, supplementary pore pressure measurements must be made at a number of levels.

The initial vertical stress in situ is estimated by using the density of the soil. This estimation can often be made using the classification of soil type and stiffness obtained from the test results to estimate an approximate density. In clay and organic soil, an accurate evaluation of the test results requires samples to be taken for determination of the liquid limit,  $w_L$ . A more accurate determination of the density can also be made from these samples. In the basic parameters required for interpretation of CPT tests in fine-grained soil, the following can therefore indirectly be included

- Liquid limit,  $w_L$

In more coarse-grained soils, the interpretation of the CPT test can be improved by parallel dilatometer tests. The latter tests provide a parallel soil classification, a good estimation of the density and an estimation of the coefficient of earth pressure  $K_0$  as supplements to the basic CPT-parameters. The coefficient of earth pressure can also to a certain extent be considered as a basic parameter for mainly coarse soils.

### Secondary parameters for interpretation

Different relations between the basic parameters are used for interpretation of the test results. For a preliminary interpretation the following parameters are often used

- $\Delta u = u - u_0$   
and  $\Delta u_{FACE} = u_{FACE} - u_0$  respectively
- Friction ratio  $R_f = (f_T/q_T) \cdot 100, \%$
- Differential pore pressure ratio  $DPPR = \Delta u/q_T$

## 5.2 DATA PROCESSING

The read off values of the parameters are presented on a plotter in the field for checking and preliminary evaluation. At the same time, they are stored in a memory for further processing, presentation and interpretation in the office. In order to reduce the required storage space, a certain selection and reduction can be made in the field. In test classes CPT2 and CPT3, the pore pressure should be recorded as frequently as possible, preferably at every 5 - 10 mm of penetration and all measured values should be stored without being processed in any way. For cone resistance and sleeve friction, it is sufficient if values are stored at about every 50 mm of depth. Preferably, averages of the measured values over the depth interval should be stored.

The recorded depth of penetration shall be controlled against the manual measurement at termination of the penetration. In the event of significant differences, the operator's notes and the field plot of the data are studied in order to find out if the error has occurred at a certain level or gradually, where this is possible. In the first case, the recorded depths below the error level are adjusted by a constant value and in the other case, all the recorded depth values are multiplied by a correction factor to make the recorded depth match the manually measured depth. Smaller errors, in the order of  $\pm 1\%$ , can for example be created because of temperature effects in some depth recording units.

The zero values read off before the start of the test shall be corrected for possible water pressure acting on the probe when the zero reading was taken. If the probe is submerged in water, for example in a pre-drilled water-filled hole, it is affected by a certain water pressure at the zero reading which also affects the zero values read off. This applies foremost to the pore pressure but also to the cone resistance and the sleeve friction in various degrees, depending on the design of the probe. These corrected zero values shall be compared to the zero readings read off after retraction of the probe after the test.

If significant differences have occurred, these shall be analysed. The most common cause of zero shifts is large loads at the termination of the test when no load restriction has been applied. Termination of the test because of deflection and large inclination is also often a cause for zero shifts. If the zero shift can be traced to the end of the test, the measured values are used without correction. If the zero shift can be traced to a certain level where very high loads occurred, for

example in a very stiff dry crust or when passing a very stiff layer or a stone, the first zero values are used down to this level, below which the latter zero values are used.

If no logical explanation can be found, an analysis should be made of possible temperature effects and with guidance from this the averages of the zero values before and after the test or alternatively gradual transitions between these are used, provided that the zero shifts are moderate.

In the event of large zero shifts which cannot be deduced with certainty, the test shall be repeated. Also in other cases where the corrections have been of such a size that they have significantly affected the interpretation, the results should be treated with caution and the tests should be supplemented.

In the following presentation, all measured values of pore pressure are plotted versus depth in order to obtain as high a resolution as possible. The measured values of cone resistance and sleeve friction are corrected according to calibration data and the pore pressures that were read off simultaneously with these data respectively. If the values of cone resistance and sleeve friction have been averaged for certain depth intervals, a corresponding averaging has to be made for the pore pressures measured simultaneously in the particular depth intervals before the correction is made.

At insertion of pore pressure, sleeve friction and cone resistance in the formulas for calculation of the parameters  $R_c$  and DPPR, averages are also used for the corresponding depth intervals. However, in this averaging and also in correlation of the different parameters to penetration depth, it has to be observed that the parameters are measured at different locations along the probe. In relation to the extreme apex of a new tip on the probe, the cone resistance is measured about 21 mm higher, the pore pressure is measured about 38 mm higher and the sleeve friction about 110 mm higher (and the pore pressure at mid-height of the conical face about 16 mm higher). The penetration depths which are recorded simultaneously with the readings of the respective parameters therefore have to be recalculated with regard to the location where the parameter was measured. This is done after the correction of cone resistance and sleeve friction for related pore pressures has been performed.

The results can then be plotted and presented, and a further interpretation made. The latter can be done manually or with the aid of interpretation programmes which are normally PC-based.

### 5.3 GRAPHICAL PRESENTATION

The test results are presented as curves for the basic parameters  $q_T$ ,  $f_T$  and  $u$  versus depth.

Uncorrected values of  $q_c$  and  $f_c$  must not be presented without clear information that they represent uncorrected values which cannot be used for interpretation, except for those special cases in which the influence of the pore pressures is not significant, (i.e. mainly in sands).

As support for a preliminary manual assessment and interpretation, curves for  $u_0$ ,  $\Delta u$ ,  $R_f$  and DPPR (and possibly measurements of  $\Delta u_{FACE}$ ) versus depth are presented.

The Swedish recommended standard for CPT tests (and also the international recommendation) specifies that the presentation shall be made with the following relations between the scales:

Depth -  $q_T$                     1 m - 2 MPa

Depth -  $f_T$                     1 m - 50 kPa

Depth -  $u$                       1 m - 20 kPa

This is adjusted to results in coarser soils and is not very useful in fine-grained soils. However, it is a good rule always to make an initial presentation of the results in these scales in order to obtain a unified picture of the relative stiffness of the soil and then make an additional more detailed presentation of all parameters in scales which are selected with consideration to the measured values in the particular test, *Figure 25*. In this latter presentation, the parameters  $u_0$ ,  $\Delta u$ ,  $R_f$  and DPPR (and possibly measurements of  $\Delta u_{FACE}$ ) are also plotted versus depth. They are plotted with the following relations between the scales:

Depth -  $R_f$                     1 m - 2 %

Depth - DPPR                  1 m - 0.5

If inclinometers and/or temperature sensors have been recorded, the readings from these shall also be presented versus depth.

In the report, information shall also be given on:

- Test area
- Operator
- Date

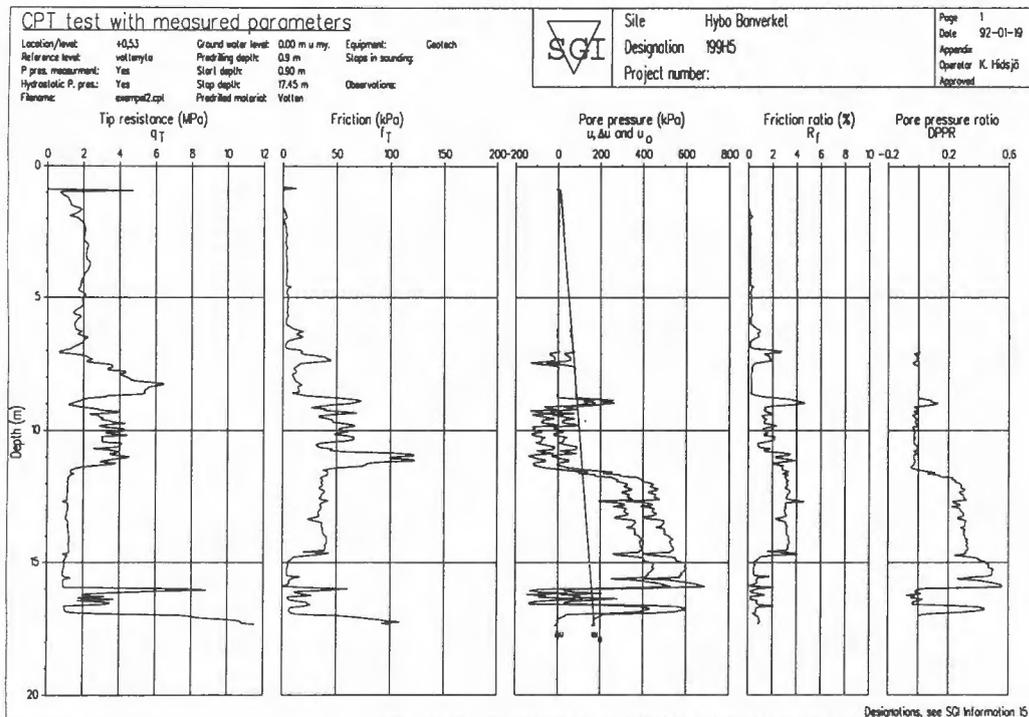
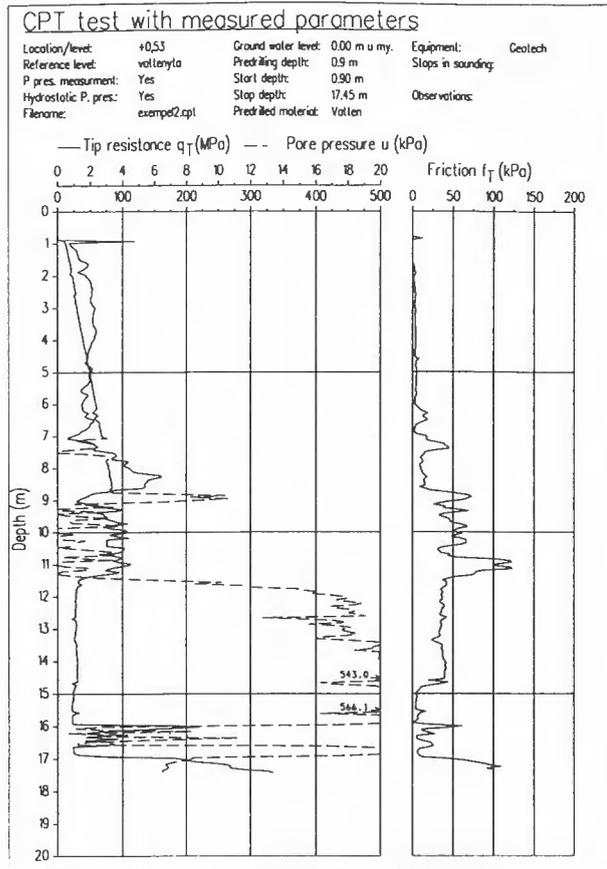
- Designation of the test point
- Location and level of the test point
- Reference level
- Free ground water level
- Pore pressure observations
- Depth of pre-drilling
- Information on the pre-drilled material
- In tests from the bottom of excavations, the depth of the excavation and the nature of the excavated material should also be specified
- Type and make of the equipment
- Probe number, location of filter element and measuring ranges
- Date of last calibration
- Area factors
- Type of fluid in the pore pressure measuring system
- Possible longer interruptions in penetration for performance of special tests (type seismic tests or pore pressure dissipation tests)
- All observations made by the operator during and after the test regarding indications of presence of stones, sounds from the rods, disturbances which may affect the results, bent rods or joints, excessive wear of the probe or significant zero shifts.
- Differences in zero values before and after the tests are given in kPa. Significant differences in the calibrations before and after the time for the test shall also be stated, as well as errors in the depth registration.
- If significant zero shifts have occurred, information shall be given on how these have been handled in processing and presentation of the data.

The Field Committee of the Swedish Geotechnical Society recommends that completed CPT tests be represented by the following symbol on the plans for the field investigations:



### 5.4 FURTHER PROCESSING OF DATA

In the further processing of the data, these normally have to be filtered and averaged. The filtering is performed in such a way that measured values which are not relevant are excluded. Examples of such data



**Fig.25. Presentation of results from a CPT test**  
 a) presentation with scales according to recommended standard  
 b) detailed presentation with selected scales

CPT-test

File name : EXEMPEL2.CPT

Test area : Hybo Banverket  
 Operator : K. Hidsjö  
 Date : 92-01-19  
 Designation : 199H5  
 Location/level: +0,53  
 Ref. level : vattenyta

Ground water level : 0.00 m. b. vattenyta  
 Predrilling depth : 0.9 m. b. vattenyta  
 Start depth : 0.90 m. b. vattenyta  
 Stop depth : 17.45 m. b. vattenyta  
 Predrilled material : Vatten  
 Equipment : Geotech  
 Stops in sounding :

Observations :

Geometry : Normal filter position  
 P Press. measurement : Yes  
 P Press. at zero reading : 0.00 kPa  
 Fluid in P Press. system : Glycerin

Pore pressure observations : Hydrostatic from G.W.L

Calibration data cone 3087 Calibrated 89-10-02

Area factor a: 0.580 b: 0.014  
 Internal friction Oc: 8.000 Of: 1.000  
 Cross talk c1: 0.001 c2: 0.001

Scale factors and measuring ranges [MPa]

Pore pressure		Friction		Tip resistance	
Range	Factor	Range	Factor	Range	Factor
1.00	2756	0.10	1148	20.00	3150

Zeor reading	Before	After	Diff	Correction
Pore pressure	112.50	112.50	0.00 [kPa]	No
Friction	27.01	27.43	0.43 [kPa]	Average before/after
Tip resistance	1968.63	1984.13	15.50 [kPa]	Average before/after

Depth correction : No

Sampling : 2 levels

From	To	Density	L limit	Classification
0.00	0.90	1.00	0.00	Exc
0.90	1.10	2.00	0.00	Sa Med

Boundaries between layers : 11 levels

6.05	6.70	7.00	7.45	8.80	9.10	11.40	14.80
16.00	16.50	17.00					

Irrelevant measurements : not given

Fig. 25 (continued)  
 c) example of a report

are relatively low values of cone resistance and pore pressure and high values of sleeve friction respectively, which are measured directly at restart of the penetration after an interruption for adding of new sounding rods and re-clutching the pushing equipment. Other examples of irrelevant peaks are values when passing shells and other coarse particles in clay profiles. The averaging is performed in order to make the information manageable and relevant.

Before this is done, the presented curves for  $q_T$ ,  $f_T$ ,  $u$ ,  $\Delta u$ ,  $R_f$  and DPPR shall be studied. Depths of borders between significant seams and layers are identified and marked. Possible depth intervals in which the measurements are considered as unreliable or insignificant are also identified and marked (for example depth intervals where the pore pressure measurements are erroneous because of loss of saturation).

For very thin layers, a classification and interpretation of the soil and its properties has to be performed manually. In order to obtain an interpretation by using a computer programme, relevant measured values of cone resistance, pore pressure and sleeve friction are required. However, in order to obtain relevant measured values of cone resistance and sleeve friction, the layer has to have a certain minimum thickness (0.2 - 0.7 m). Much thinner seams and layers can be detected and classified by studies of the generated pore pressures in the profile and by relating possible variations to tendencies for changes in cone resistance and sleeve friction at the corresponding levels and by also relating them to results from samplings and corresponding generated pore pressures at adjacent test points. This type of interpretation can only be made manually, preferably before any computer aided interpretation.

In filtering and averaging, the soil profile is divided into depth intervals of 0.2 m as a suggestion. However, these intervals are not allowed to pass a pre-marked border between layers, but in this case the pre-marked level should constitute the lower limit of this interval and the upper limit of the next. For each interval, the corrected parameters within its limits are collected. If a border of the interval consists of a pre-marked border between layers, the values on this border are excluded. Otherwise, the values on the borders of the intervals are included in the data for both the overlying and the underlying intervals.

First, the averages of the parameters within the intervals are calculated and then the standard deviations. All values that differ more from the averages than the standard deviations are filtered out and then

new averages are calculated.

These averages are then used for classification of the soil within the respective intervals and for interpretation of its various properties. This process should also specify the intervals in which possible parameters have been included in the evaluation, which have been considered as unreliable.

# Evaluation of stratigraphy and classification of soil

## STRATIGRAPHY

**Cone resistance** is a measure of the stiffness and variation of the soil. The regularity of the curve for cone resistance versus depth is also a measure of the grain size of the soil. The resolution is relatively good for fine-grained soils up to the gravel fraction, where the curves become so irregular that they are difficult to evaluate. In very stratified soils, the possibility of interpreting the stiffness in separate layers is limited. The cone resistance is affected by the properties in both overlying and underlying soils to a distance equal to 5 - 20 diameters of the probe. A stiff layer embedded in soft soil should be 0.4 - 0.7 m thick if its properties are to be registered to the full extent. Soft layers in stiffer soil should be 0.2 - 0.4 m thick in order to yield a registration of a relevant cone resistance.

The **sleeve friction** is, among other things, a measure of the horizontal pressure that develops during the penetration and is thus affected by the type of soil and its overconsolidation ratio. The transitions between different types of soil are primarily signified by changes in the relation between the sleeve friction and the cone resistance, which is normally expressed by the friction ratio  $R_f = f_T/q_T$ .

The registered **pore pressures** are representative for the soil at the level of measurement and very thin layers can thereby often be registered. The relation between the excess pore pressure and the cone resistance,  $DPPR = \Delta u/q_T$ , is an indication of what type of soil is penetrated and in clay, the generated pore pressure is also a measure of the undrained shear strength. The interpretation is complicated by the fact that also other factors and primarily the overconsolidation ratio also affect the pore pressure. It is also dependent on the location of the filter on the probe. The limitation in its resolution mainly depends on the frequency of reading and whether the pore pressure measuring system can be kept fully saturated.

Problems in the interpretation can occur in non-

saturated soil, in multi-graded soil and in overconsolidated soil, as well as in the occurrence of stones or shells, for example, in clay profiles. The interpretation and evaluation in fine-grained soils are facilitated by parallel tests with the filter located on the conical face of the tip. Shorter interruptions in the penetration for studies of the pore pressure dissipation on levels which are difficult to interpret can also be helpful, *Figure 26*.

In the assessment of thin layers of coarser soil in clay, which in some aspects can be very important, also the results of tests in adjacent test points should be studied in order to verify the existence and extent of the layer. Also thick layers can prove to be locally embedded lenses or to be of highly varying thickness.

## SOIL CLASSIFICATION

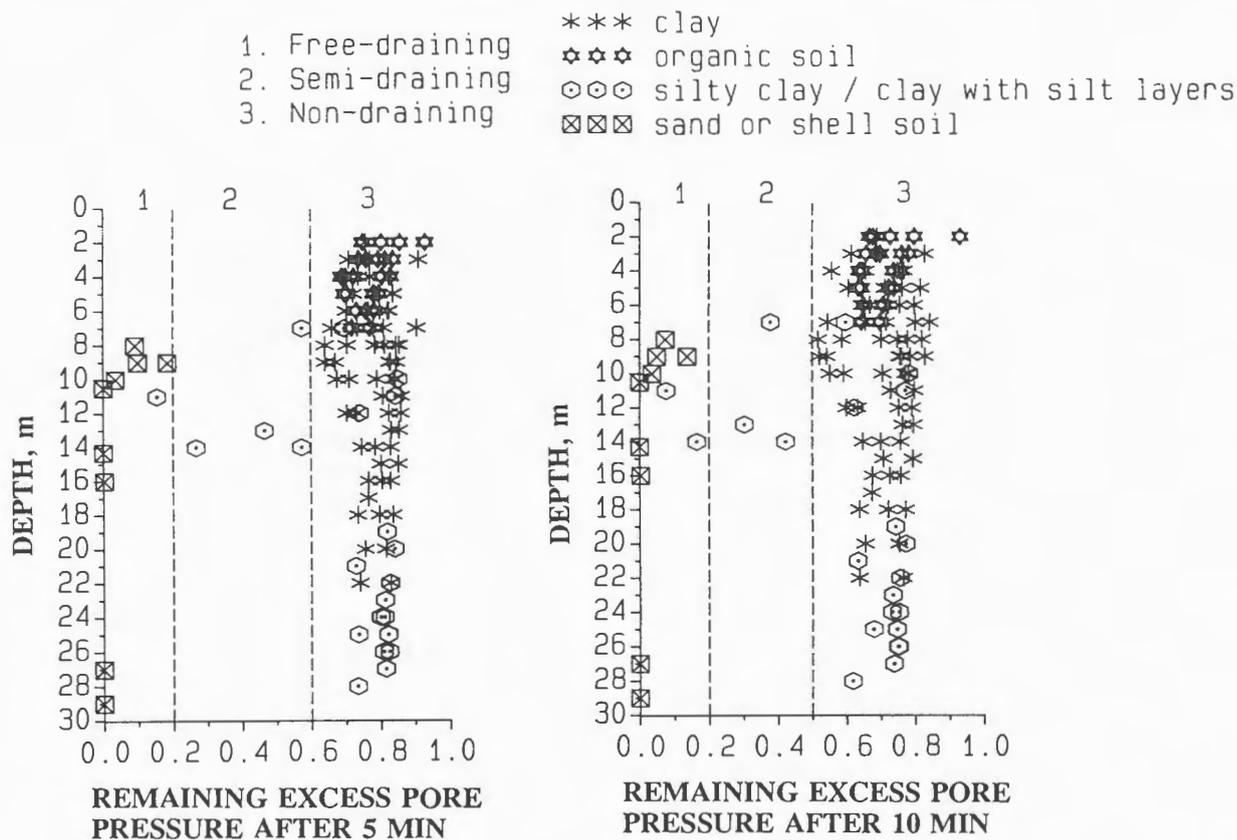
In the tests, cone resistance, sleeve friction and pore pressure are normally measured. Since a large number of factors affect the results, no unambiguous soil classification can be made on the basis of these parameters alone; instead, supplementary data are required. These mainly consist of a measurement of the natural pore pressure profile in situ, sampling, further measured parameters or parallel tests with other methods. The properties of the soil affect the results of the CPT-tests in the following ways, among others:

### Cone resistance

- The coarser soil, the higher the cone resistance
- The higher the in situ horizontal stress, the higher the cone resistance
- The higher the degree of compaction or the denser the material respectively, the higher the cone resistance
- The higher the overconsolidation ratio, the higher the cone resistance

### Sleeve friction

- The coarser the soil, the lower the friction ratio



**Fig. 26. Classification of the drainage properties in soil with the aid of shorter pore pressure dissipation tests. (Remaining excess pore pressures are expressed in relation to the maximum excess pore pressure when stopping of the penetration. From SGI Report No. 42 (Larsson and Mulabdic 1991).**

- The denser the friction soil, the lower friction ratio
- The higher the sensitivity, the lower the friction ratio
- The higher the overconsolidation ratio in cohesive soil, the higher the friction ratio

#### Pore pressure

- The more fine-grained the soil, the higher the pore pressure
- The higher the undrained shear strength, the higher the pore pressure
- The denser the friction soil, the lower the pore pressure (applies mainly to the normal filter location)
- The higher the overconsolidation ratio, the lower the pore pressure (applies mainly to the normal filter location)
- The higher the sensitivity, the higher the pore pressure

- The coarser the material, the lower the pore pressure ratio
- The higher the overconsolidation ratio, the greater the difference between pore pressures measured on and above the conical face of the tip respectively
- The more fine-grained the soil, the longer the time required for excess pore pressure dissipation

As is apparent from the statements above, an unambiguous classification cannot be made from these often ambiguous relations, but the parameters are nevertheless often used for a **preliminary classification** of the soil. In addition to the measured parameters in the test, the foremost supplement required is then a measurement of the initial pore pressure profile. Normally, use is made of one (or several) of the classification charts, which have been produced empirically on the basis of, for example, the way in which the relation between cone resistance and friction ratio normally

varies with the type of soil. The evaluation of soil type made in this way can then be checked by comparison with a corresponding evaluation using as a basis the normal variation between pore pressure ratio and cone resistance with soil type. Often, especially in even-grained soils and pure clays, the classifications are in agreement, but in other cases conflicting classifications may be obtained. The soil classification must then be made more elaborate and, if possible, more bases for forming an estimate have to be utilised. A preliminary classification can be made as follows:

All known values of the density of the different soil layers are used to calculate the variation of the total overburden pressure with depth. The effective vertical stress is then calculated using the measured in situ pore pressures. If values of the density from samples are missing, values from parallel dilatometer tests, for example, can be used. On levels or in entire profiles where determinations of the density are missing, an empirical estimation of the density has to be made in parallel with the classification of the soil.

In the uppermost layers of dry crust or fill, an estimated density is applied. The total and effective vertical stresses are then calculated in the underlying depth intervals, which are normally given thicknesses of 0.2 m. The density in the particular interval is given as the determined density or, when this is missing, as the density in the adjacent overlying soil.

The parameters  $(q_T - \sigma_{v0})/\sigma'_{v0}$  and  $f_T/(q_T - \sigma_{v0})$  are calculated for the interval and the principal character of the soil is estimated by using the diagram in *Figure 27*.

In depth intervals where a previous better classification is missing, the **division into sand, silt and more fine-grained soil** is thus mainly made on the basis of the relation between the cone resistance and the normal in situ stress condition and the developed sleeve friction in relation to this cone resistance. In soft clay, the measured sleeve friction is very small and relatively unreliable, but in overconsolidated clay, where the cone resistance may be of the same size as for soft coarser soil, the measured values normally become larger and more reliable. Possible uncertainties in the measurements of sleeve friction usually have a relatively small influence on this division. The main exceptions are highly sensitive clays and/or silty clays. In these soils, the sleeve friction may be very low at the same time as the measured stiffness in relation to the overburden pressure places the soil in the region for silt in the diagram in *Figure 27*. However, these soils often

developed very high pore pressures in the tests and a check of whether the factor  $B_q [= \Delta u / (q_T - \sigma_{v0})]$  is higher or lower than 0.6 can be used to judge whether the soil should be classified as silt or clay.

In cases where the soil in the interval is classified as sand or silt, its stiffness is also classified and, if stiffness values are missing, its density is calculated by using the same diagram. The classification of stiffness in coarser soil was previously normally made only on the basis of the cone resistance  $q_T$  or alternatively the net cone resistance  $(q_T - \sigma_{v0})$ . This has been shown to result in a gradual change in the classification with depth as the vertical stress and, to an even higher degree, the cone resistance increases. The classification according to the normalised net cone resistance  $(q_T - \sigma_{v0})/\sigma'_{v0}$  is more correct from this point of view, but the empirical experience is somewhat more limited. The limits for the various sub-designations with respect mainly to the denseness of sand and silt are therefore preliminary. However, the borderline between silt and "clay and organic soil" can be considered to be well established.

The normalisation through division by the effective vertical stress can introduce a source of error in superficial layers where the vertical stresses are low. A check must therefore be made so that the net cone resistance is  $\geq 1.5$  MPa if the soil is to be classified as a sand and  $\geq 500$  kPa to be classified as silt. A check must also be made so that the net cone resistance is greater than 20 MPa if a sand is to be designated as very dense, greater than 10 MPa to be designated as dense, greater than 5 MPa to be designated as medium dense and greater than 2.5 MPa to be designated as loose. The corresponding limits for silt are 10, 5, 2.5 and 1.0 MPa.

The limits between the different groups of soil are furthermore not distinct and apply in principle only to even-grained soils. For example, soils whose parameters plot near the border between sand and silt can thus consist of silt, sandy silt, silty sand or sand. Correspondingly, materials whose parameters fall within the silt region but are close to the border with cohesive soils can often be clayey. More multi-graded materials such as clayey sand or fine-grained till will probably fall within the regions for silt or clay depending on their clay content. The fact that the certainty and resolution in the soil classification based on CPT tests is limited has led to a preference to assign this classification to "type of soil behaviour from a geotechnical point of view" rather than to classification with respect to grain

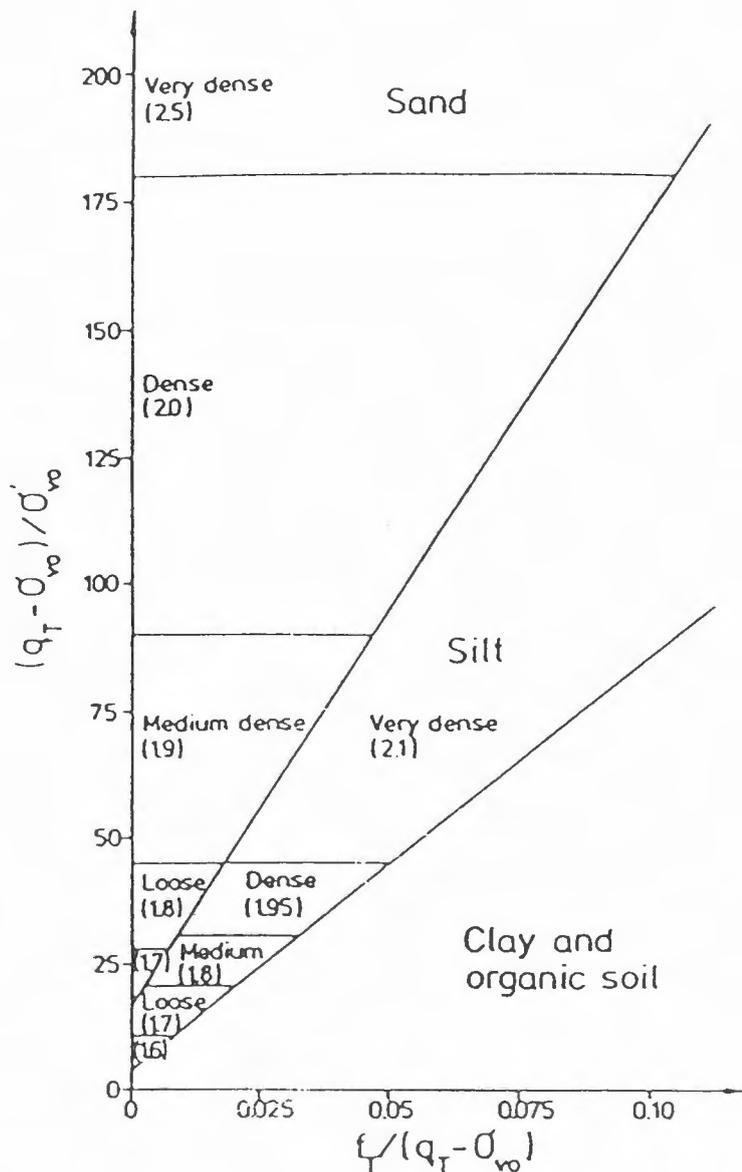


Fig. 27. Diagram for evaluation of silt and sand and separation of cohesive soil. Numbers in brackets represent estimated water saturated density of the soil in  $t/m^3$ .

size distribution. Because of the complex influence of different factors on the measured parameters, this simple division based only on cone resistance and sleeve friction should not be made without a parallel check of the generated pore pressures. Otherwise, there is a considerable risk of, for example, a layer where relatively high pore pressures are developed being classified as pure sand only because the soil is also stiff and the friction ratio is low.

In those cases where the soil has been classified as “clay and organic soil” in the first diagram or after further checks on this have been specified at an earlier stage, the classification process proceeds with the

special classification charts for this type of soil. These charts are based on the parameters net cone resistance  $(q_T - \sigma_{v0})$  and  $B_q = \Delta u / (q_T - \sigma_{v0})$ . Parameter  $B_q$  is calculated first.

In the diagram in *Figure 28*, the soil is divided into five main groups:

- Heavily overconsolidated clay
- Overconsolidated or very silty clay
- Normally consolidated clay or slightly overconsolidated silty clay
- Normally consolidated silty clay and/or highly sensitive clay
- Normally consolidated gyttja or organic clay

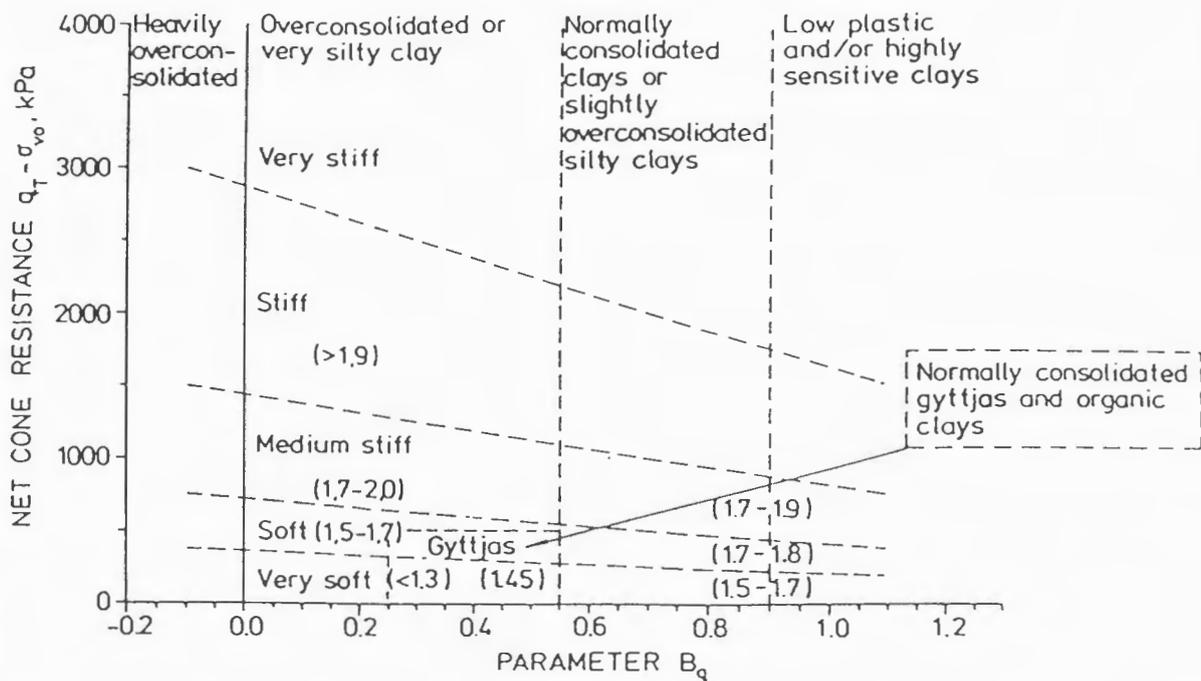


Fig. 28. Classification chart for clay and organic soil.

The designation normally consolidated refers to overconsolidation ratio OCR 1 - 1.5, overconsolidated to OCR 1.5 - 10 and heavily overconsolidated refers to OCR > 10 in accordance with the Swedish classification system.

The soil in the groups is also summarily classified with respect to the undrained shear strength as very soft, soft, medium stiff, stiff or very stiff. This division is preliminary because the relation between the net cone resistance and the shear strength depends among other things on the consistency limits of the soil. After a more careful evaluation of the undrained shear strength, the sub-designations with respect to the undrained shear strength should be adjusted according to:

Undrained Shear strength kPa	Designation
< 12,5	Very soft
12,5 - 25	Soft
25 - 50	Medium stiff
50 - 100	Stiff
> 100	Very stiff

In this classification, it is not possible to distinguish normally consolidated **gyttja and organic soil** from overconsolidated soft clay and these fall into the same group. Overconsolidated gyttja will mainly be classified as clay.

In cases where density values are missing in the interval, a relevant chart for estimation of density is used, *Figure 29*. The borders between different density groups mainly coincide with the borders in the preceding chart, but certain modifications have been made based on the empirical data base.

After completion of the process of classification and possible estimation of density in the first interval, the process is repeated in the next interval with insertion of known or estimated density in the now overlying interval. In extreme cases, the effective stresses in superficial layers calculated with assumed and evaluated density in the interval respectively can differ so much that the evaluation is affected. This should be checked and if this is the case, the process must be repeated with insertion of the new estimated density for the particular interval.

The main purpose of the CPT test is not to classify the soil but to clarify the stratification and the bound-

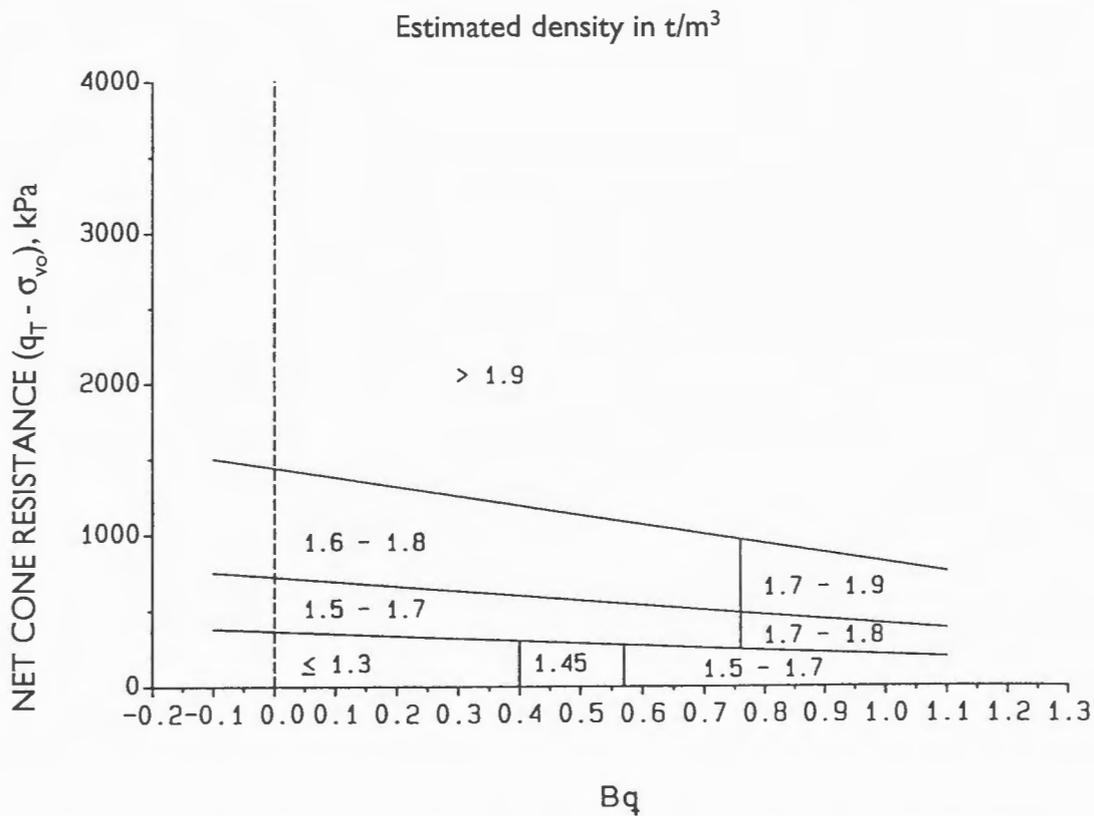


Fig. 29. Chart for estimation of density in cohesive soil on the basis of results from CPT tests.

aries between different layers and to give an overview of the properties of the soil. Presentation and evaluation are normally aided by a computer in order to rationalise the comprehensive work of calculation and plotting. A preliminary classification is then often performed on the computer for those depth intervals in which the type of soil has not been entered manually as an input. Because of the difficulties and ambiguities in the interpretation process described above, this preliminary classification must be checked manually and judged by using all the available penetration test data, parallel field tests, existing soil samples and further information on the soil conditions. The classification should most appropriately be performed as an interactive process.

Also after a manual check and assessment, the classification is not unambiguous but an indication of the type of soil for which the results of the test are typical. The classification may be more reliable if results from parallel tests with alternative filter location or excess pore pressure dissipation tests are available. The CPT test ought always to be supple-

mented by at least disturbed sampling for a correct classification of the soil and determination of, for example, liquid limit for a more careful evaluation of the properties in clay and gyttja. The preliminary classification and interpretation is then a great aid in determination of the required sampling and can later be modified with regard to the results from this.

# Evaluation of undrained shear strength

The undrained shear strength in fine-grained soils is mainly evaluated from the net cone resistance ( $q_T - \sigma_{v0}$ ).

The relation between the undrained shear strength and the net cone resistance is sensitive to the liquid limit of the soil,  $w_L$

$$\tau_{fu} = \frac{q_T - \sigma_{v0}}{13.4 + 6.65w_L}$$

The undrained shear strength evaluated in this way directly corresponds to the undrained shear strength obtained from corrected field vane and fall cone tests, direct simple shear tests and dilatometer tests, *Figure 30*.

In the test, only a limited volume of soil is involved and the undrained shear strength may be over estimated in fissured and non-homogeneous soils. The evaluated shear strength is therefore only applicable to homogeneous soil. **In very fissured and non-homogeneous soil only about half of this shear strength can be counted upon.**

If values of the liquid limit of the soil are missing, a rough estimate of the undrained shear strength can be made according to

$$\tau_{fu} = \frac{q_T - \sigma_{v0}}{16.3} \quad \text{for clay}$$

$$\tau_{fu} = \frac{q_T - \sigma_{v0}}{24} \quad \text{for gyttja}$$

As an alternative to the net cone resistance, the generated pore pressure can be used for evaluation of the corresponding undrained shear strength. The main reason for using the excess pore pressure for this evaluation is that the resolution and accuracy are often better for this parameter in soft soils. It is then prima-

rily the excess pore pressure at the conical face of the tip,  $\Delta u_{FACE}$ , which can be used. **For normally consolidated and slightly overconsolidated soil**, the shear strength can be evaluated from

$$\tau_{fu} = \frac{\Delta u_{FACE}}{16}$$

This relation is slightly sensitive to the plasticity and/or the sensitivity of the soil and can be elaborated to

$$\tau_{fu} = \Delta u_{FACE} / (17.23 - 1.65w_L)$$

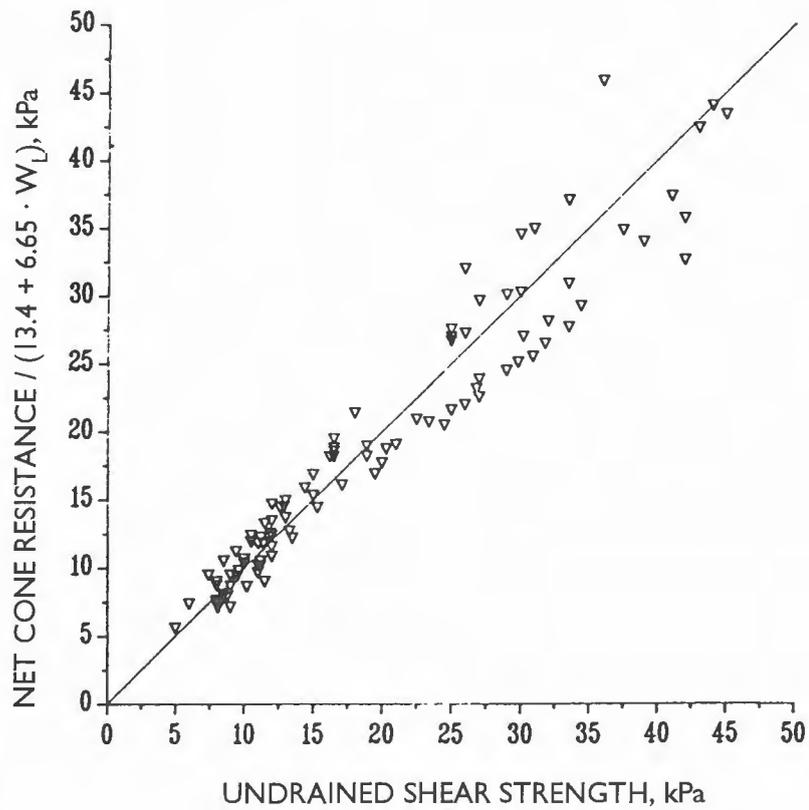
or alternatively

$$\tau_{fu} = \Delta u_{FACE} / (13 + \ln St)$$

The relation is also sensitive to the overconsolidation ratio and the homogeneity of the soil and should only be applied to normally consolidated and slightly overconsolidated soil.

A similar relation can be found for the generated pore pressure at the normal filter position and the undrained shear strength for almost normally consolidated soil, [ $\tau_{fu} = \Delta u / (14.1 - 2.8 \cdot w_L)$ ]. However, this relation is so sensitive to the overconsolidation of the soil and to some extent also to the exact geometry of the probe that it **should normally not be applied.**

The accuracy of the evaluation of the undrained shear strength mainly depends on the accuracy achieved in the test (the test class in which the test has been performed). When very accurate estimates are required, the results from the CPT tests should be calibrated locally against field vane tests and preferably also direct simple shear tests.



**Fig. 30.** Comparison between undrained shear strength evaluated from net cone resistance in CPT tests in Swedish very soft to medium stiff clays and reference data from direct simple shear tests and corrected field vane tests.

# Evaluation of preconsolidation pressure

A preliminary estimation of the preconsolidation pressure in **cohesive soils** can be made from the net cone resistance, *Figure 31*. The relation is sensitive to both the plasticity and the overconsolidation ratio of the soil.

$$\sigma'_c \approx \left( \frac{q_T - \sigma_{v0}}{1.21 + 4.4w_L} \right) / (1.07 - 0.54 \log OCR)$$

The equation is solved by iteration with insertion of the estimated in situ effective vertical stress  $\sigma'_{v0}$  and  $OCR = \sigma'_c / \sigma'_{v0}$ .

In tests where the pore pressure has been measured at the conical face of the tip, the preconsolidation pressure in **normally consolidated** and **slightly overconsolidated** soils can be estimated from

$$\sigma'_c \approx \Delta u_{FACE} / a$$

where

$$a = 2.05 + 2.62 \cdot w_L \leq 4.7$$

When values of the liquid limit of the soil are missing, a very rough preliminary estimate of the preconsolidation pressure might be made from

$$\sigma'_c \approx \Delta u_{FACE} / 3.5 \quad \text{for clay}$$

and

$$\sigma'_c \approx \Delta u_{FACE} / 4.7 \quad \text{for gyttja}$$

However, the latter relations are so sensitive to the plasticity of the soil that they should normally not be applied.

In soils with an overconsolidation ratio higher than 2, this has to be accounted for according to

$$\sigma'_c \approx \frac{\Delta u_{FACE}}{1.4 \cdot \alpha \cdot OCR^{-0.8}}$$

Also this equation is solved by iteration and insertion of the in situ effective vertical stress.

A corresponding relation for the excess pore pressure at the normal filter location can be written

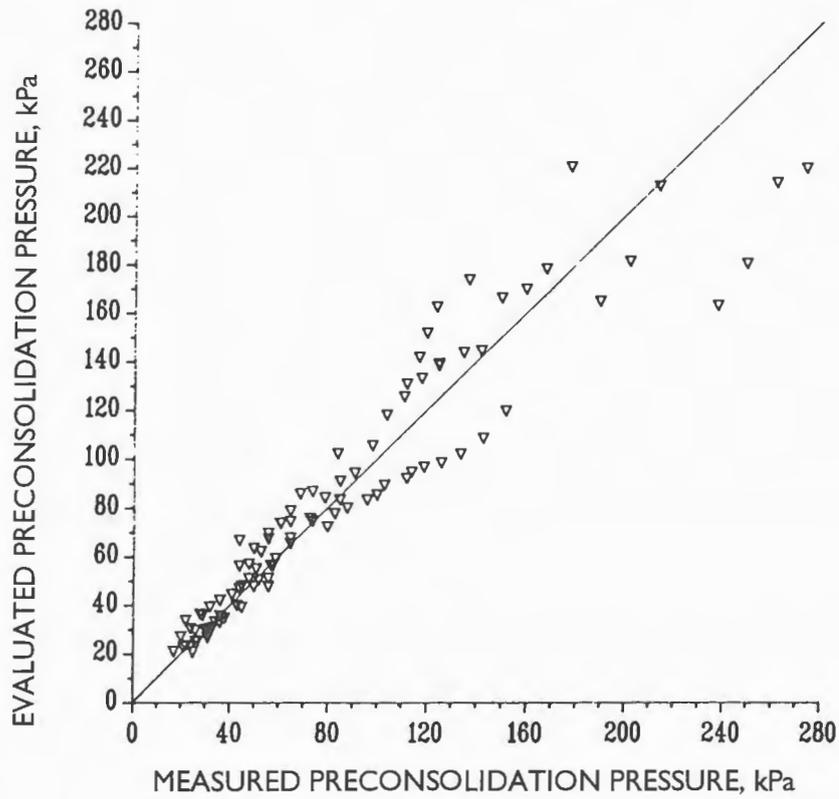
$$\sigma'_c \approx \Delta u / [b(1.10 - 0.96 \log OCR)] \quad \text{for } OCR \leq 14$$

where

$$b = 2.0 + 1.16w_L \leq 3.16$$

However, this relation is **considerably more unreliable**, mainly because of the sensitivity to the overconsolidation ratio and to the exact geometry of the probe.

Preconsolidation pressures evaluated from CPT tests can only be used as **supplements to oedometer tests** and never as replacements for these. Also with this limitation the tests have to be performed with very high accuracy if an evaluation of the preconsolidation pressure is to be worthwhile.



**Fig. 31.** Comparison between preconsolidation pressures evaluated from net cone resistance in CPT tests and reference values from oedometer tests.

## Evaluation of overconsolidation ratio

The overconsolidation ratio is the quotient between the preconsolidation pressure and the in situ effective vertical stress,  $OCR = \sigma'_c / \sigma'_{v0}$ . This parameter is thus obtained indirectly at the evaluation of the preconsolidation pressure according to the preceding chapter. There are also a number of empirical relations between overconsolidation ratio, in situ effective vertical stress and measured parameters in CPT tests in **cohesive soil**. These can be used as supplements to the indirect estimation of the overconsolidation ratio.

Depending on which data are available and to some extent its value, the overconsolidation ratio can be estimated from the following relations

- $\Delta u_{FACE} / \sigma'_{v0}$
- $(\Delta u_{FACE} - \Delta u) / \sigma'_{v0}$
- $(q_T - \sigma_{v0}) / \sigma'_{v0}$
- $(q_T - u) / \sigma'_{v0}$
- $f_T / \sigma'_{v0}$

**All these relations** are more or less dependent on the liquid limit of the soil. They **involve a considerable spread and the estimation of the overconsolidation ratio is relatively approximate**. As many of the relations as possible should be used and the results should be gathered and judged with consideration to type of soil and other factors.

For clay and organic clay with overconsolidation ratios up to 3, a relatively good relation has been found for the **pore pressure at the alternative location**

$$\log OCR \approx 0.24 \frac{\Delta u_{FACE} / \sigma'_{v0}}{2.8 + 2.65w_L} - 0.14$$

The usefulness of this relation in very silty clay is uncertain.

When **pore pressures from both normal and alternative filter location** are available, the relation

$$\log OCR \approx 0.15 \frac{\Delta u_{FACE} - \Delta u}{\sigma'_{v0} (0.31 + 1.27w_L)} - 0.05$$

can be used for overconsolidation ratios up to 15. Since the sources of error increase, the uncertainty also increases, especially in normally consolidated and only slightly overconsolidated soil.

For the **net cone resistance**, the relation

$$\log OCR \approx 0.32 \frac{q_T - \sigma_{v0}}{\sigma'_{v0} (1.13 + 5.76w_L)} - 0.22$$

can be used. The relation is very sensitive to the liquid limit of the soil, but it can be used for all overconsolidation ratios and for all types of cohesive soil. In highly fissured clay, the overconsolidation ratio may be underestimated.

Another way of using results from tests with **normal filter location** is to apply the relation

$$\log OCR \approx 0.167 \frac{q_T - u}{\sigma'_{v0} (5.0w_L - 0.6)} - 0.05$$

This relation has the same advantages and limitations as the preceding relation. However, the spread is larger for normally consolidated and slightly overconsolidated soil and in silty clay.

Also the measured **sleeve friction** can be used as an indication of the overconsolidation ratio in cohesive soil. The measuring accuracy is limited, especially in soft soils, but this parameter is independent of the other and thus offers an additional separate indication of the size of the overconsolidation. The overconsolidation ratio can then be roughly estimated from

$$\log OCR \approx 0.13 \frac{f_T}{c \cdot \sigma'_{v0}} - 0.015$$

where

$$c = 0.233w_L - 0.047 \leq 0.28$$

The overconsolidation ratio in **friction soil** cannot be estimated from the results of CPT tests.

## Evaluation of relative density

The density in **friction soil** can be estimated from results of CPT tests and is then normally expressed in terms of relative density  $I_D$

$$I_D = \frac{e_{\max} - e}{e_{\max} - e_{\min}}$$

The estimation is made from the relation between the cone resistance and the effective overburden pressure. This relation depends, apart from the density, also on the internal friction in the soil, the grain size distribution and the compressibility of the soil and also on its overconsolidation and related horizontal stress. The estimation of the relative density that can be made therefore **only applies to normally consolidated and relatively even-graded sand with a mineral composition of mainly a mixture of quartz and feldspar**. For this type of sand, the relative density can be estimated from

$$I_D \approx -131 + 66 \cdot \log\left(\frac{q_T}{\sigma'_{v0}{}^{0.5}}\right), \quad \%$$

where  $q_T$  and  $\sigma'_{v0}$  are expressed in kPa (Lancelotta 1983).

The spread is normally within about  $\pm 10\%$  for normally consolidated sand and is mainly affected by the compressibility of the soil, *Figure 32*.

In overconsolidated sand the relative density is overestimated, especially in **loose sand**, *Figure 33*.

For soil coarser than sand, the evaluated relative density should be reduced by 10 - 15 % according to Lunne and Christoffersen (1983).

The relations between cone resistance, effective overburden pressure and relative density mainly relate to the results from large scale laboratory tests in "calibration chambers". There is a certain uncertainty about the application to naturally deposited soils in the field and far-reaching conclusions should be avoided.

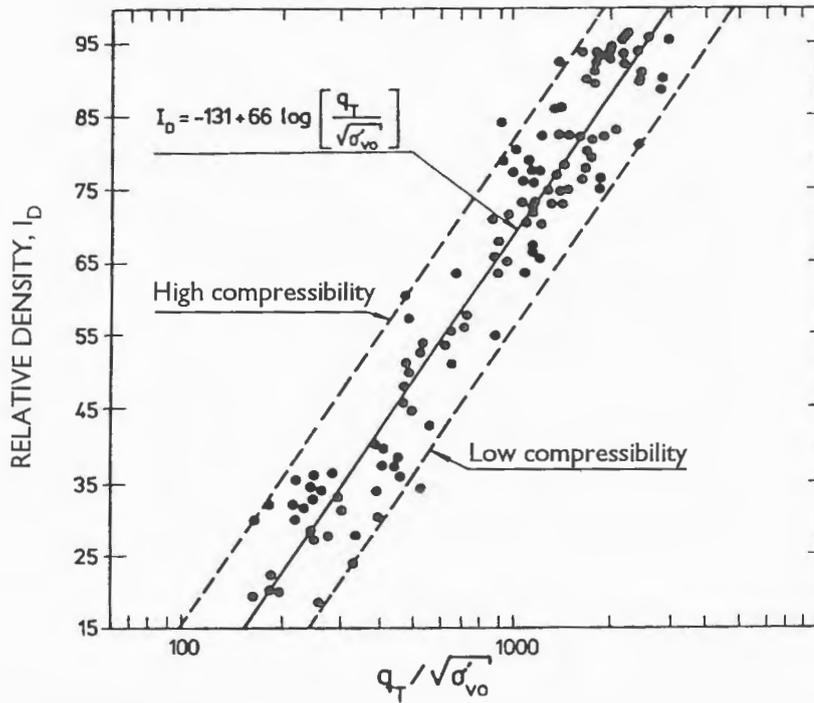


Fig. 32. Relation between relative density, cone resistance and effective overburden pressure in normally consolidated even-graded quartz sand (after Lancelotta 1983).

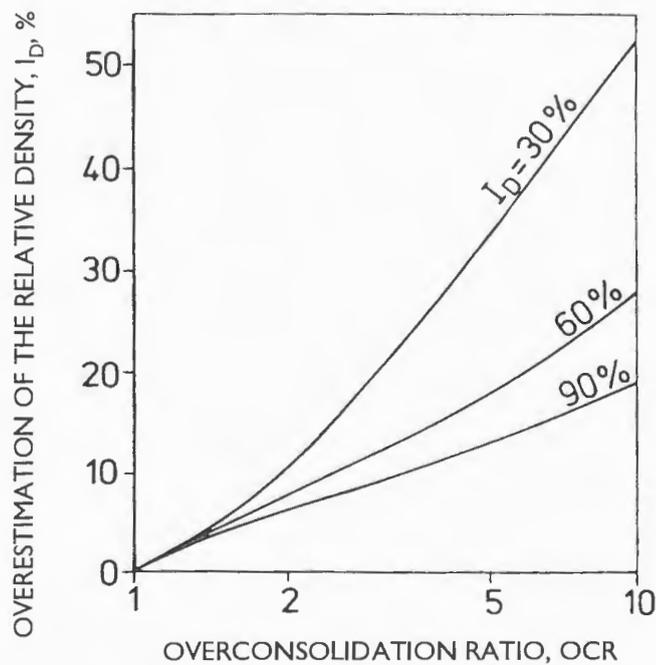


Fig. 33. Overestimation of relative density because of overconsolidation. (After Baldi et al. 1986).

## CHAPTER 11.

# Evaluation of friction angle

Also the friction angle in *friction soil* can be evaluated from the relation between the cone resistance and the in situ effective vertical stress. In reality, it is mainly the horizontal stress that governs the relation and, before the evaluation is made, some kind of estimation of the coefficient of earth pressure should be made.

A standardised estimation of the coefficient of earth pressure can be made in the following way:

- The coefficient of earth pressure at rest in normally consolidated soil  $K_0 \approx 1 - \sin \phi'$
- The coefficient of earth pressure decreases towards the coefficient of active earth pressure  $K_A$  in the upper part (the active zone) of steep slopes
- The coefficient of earth pressure increases gradually with overconsolidation ratio and in passive zones towards the value at fully mobilised passive earth pressure  $K_p$ .

The friction angle can then be evaluated from the diagram in *Figure 34*.

As can be seen in the diagram, there is no great difference in evaluated friction angle on the assumption of active earth pressure or earth pressure at rest respectively, (maximum 0.5°). For heavily overconsolidated soil and at passive earth pressure, the friction angle may be overestimated by a maximum of 10 % if the higher horizontal stresses are not taken into account.

The friction angle is not a constant parameter, but varies with the stress level in the soil, mainly because of crushing of the particles and the compressibility of the soil, and also with the mode of loading (see, for instance, SGI Information 8). The friction angle  $\phi'$  can thus be written

$$\phi' = \phi'_{cv} + \mu \cdot F \cdot I_D \cdot [(Q - \ln p') - 1]$$

where

$\phi'_{cv}$  = Friction angle at constant volume

$\mu$  and  $Q$  = Material parameters

$F = 3$  for the triaxial case and 5 for plain strain

$I_D$  = Relative density (material parameter)

$p'$  = Mean effective stress,  $(\sigma'_1 + \sigma'_2 + \sigma'_3)/3$

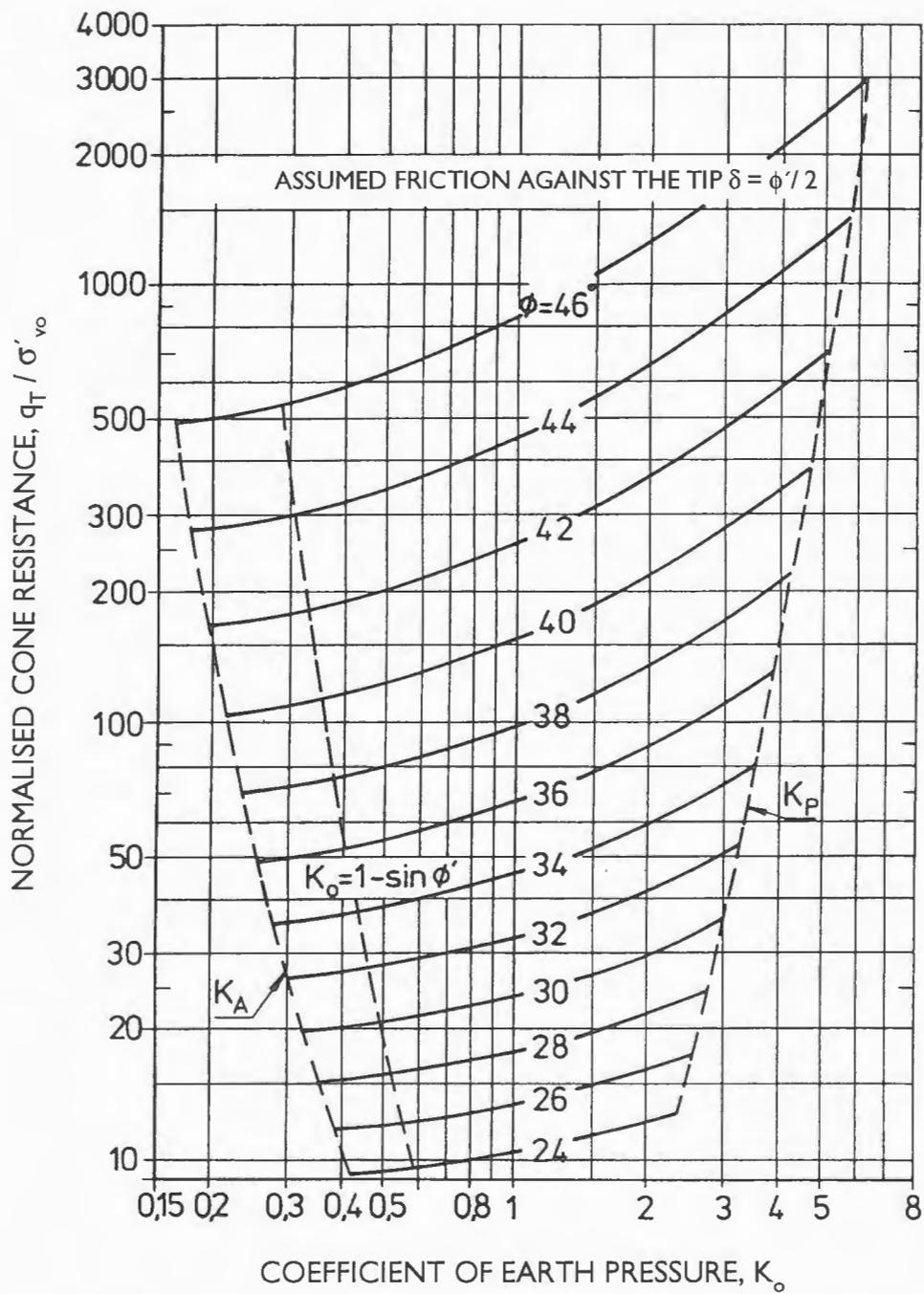
The friction angle evaluated from the CPT test corresponds to the triaxial case and is evaluated for a stress level which varies between about  $0.11 \cdot q_T$  at  $\phi' = 30^\circ$  and about  $0.06 \cdot q_T$  at  $\phi' = 45^\circ$  (Bellotti et al. 1989).

For ordinary sand, with a mixture of quartz and feldspar,  $\phi'_{cv}$  can be assumed to be about  $34^\circ$  and the parameter  $Q \approx 10.5$ . The difference between  $\phi'$  and  $\phi'_{cv}$  is equivalent to the combined effects of the parameters  $\mu$ ,  $F$  and  $I_D$  and the relative stress level  $[(Q - \ln p') - 1]$ . The three parameters  $\mu$ ,  $F$  and  $I_D$  and their product can in this context be considered as material constants for the particular soil, unless this is to be compacted or re-excavated.

When  $\phi'$  and the related  $p'$  in the test are known, the product  $\mu \cdot F \cdot I_D$  can be calculated and the friction angle recalculated for any stress level of interest. This is shown graphically in *Figure 35*.

A prerequisite for the application of this type of evaluation is that the soil consists of a fairly normal quartz-feldspar sand.

The uncertainty increases with increasing value of  $\phi'$  but, on the other hand, also the safety margin often increases since in most cases it is the friction angle at plane strain that is relevant rather than the friction angle in the triaxial case. It is possible to convert the evaluated friction angle to the friction angle at plane strain by using the given formulas but, with consideration to the uncertainties mentioned above, this is not recommended for normal applications of the results.

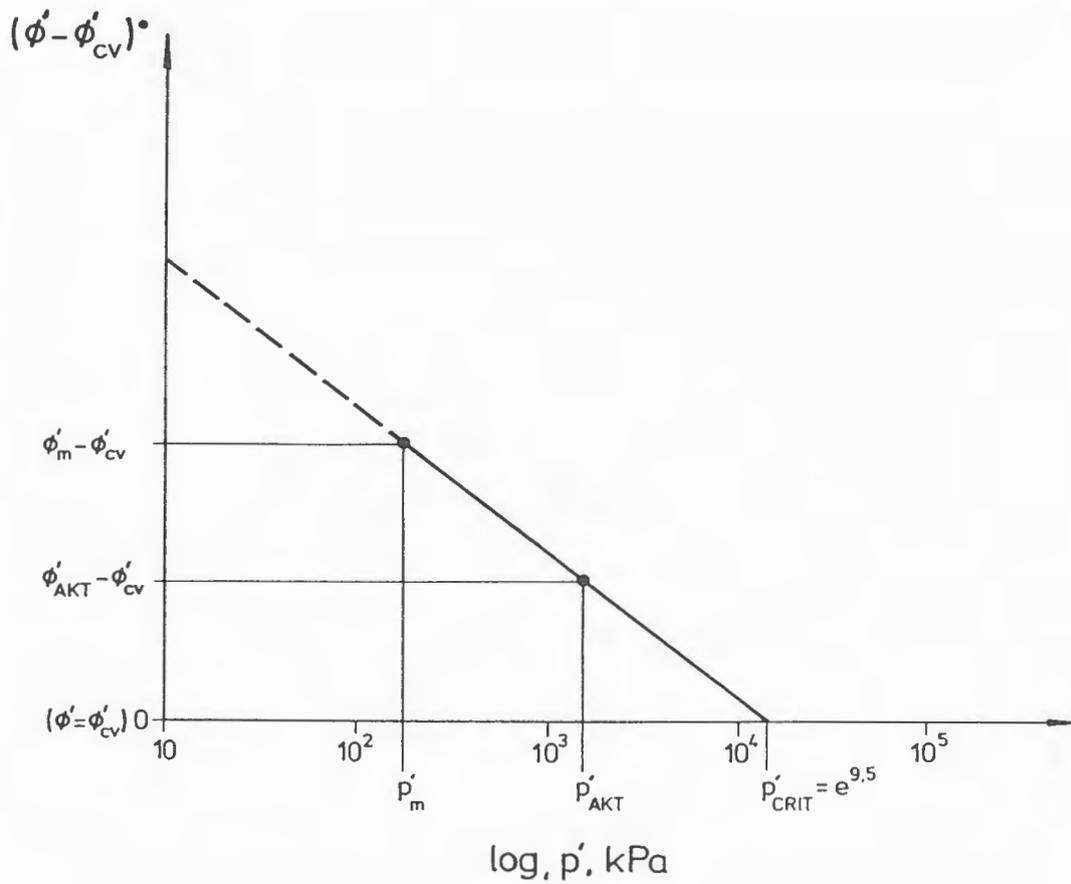


$\sigma'_{vo}$  = effective overburden pressure

$K_A$  = coefficient of earth pressure at active pressure

$K_P$  = coefficient of earth pressure at passive pressure

Fig. 34. Diagram for evaluation of friction angle from results of CPT tests. (After Marchetti 1985).



**Fig. 35. Modification of evaluated friction angle with regard to stress level.**

$\phi'$  = evaluated friction angle at stress level  $p'_m$

$\phi'_{\text{OF INTEREST}}$  = sought friction angle at stress level  $p'_{\text{OF INTEREST}}$

## Evaluation of deformation properties

The evaluation of deformation properties is normally not made directly from the CPT test results. In *friction soils*, instead some of the well established empirical calculation methods specially produced for the CPT test are used, (see, for example, SGI Report No 22). However, these methods are only suited for calculation of settlements and bearing capacity of normal cases of shallow foundations on sand.

For other cases of loading on sand, compression moduli and moduli of elasticity may be estimated from *Figure 36 a* and *b*. The values of in situ effective vertical stress and cone resistance for the particular level are entered into the diagrams and the compression modulus or modulus of elasticity respectively is read off. The modulus of elasticity is not a constant but decreases with increasing level of shear stress. Two moduli,  $E_{25}$  and  $E_{50}$ , are given and correspond to the secant modulus at 25 % and 50 % of the failure stress respectively. At normal factors of safety,  $E_{25}$  can be considered as most relevant.

The compression modulus for normally consolidated sand can be calculated from the relation

$$M \approx 14.48 \cdot q_T \cdot \left( \frac{\sigma'_{v0} (1 + 2K_0)}{300} \right)^{-0.116} \cdot e^{-1.123I_D}$$

where  $\sigma'_{v0}$  is expressed in kPa,  $K_0 \approx 0.45$ ,  $I_D$  is estimated from the CPT test results and  $e$  is the base of the natural logarithm, (Jamiolkowski et al. 1988).

For overconsolidated soil, this modulus should be multiplied by a factor of about  $OCR^{0.313}$ , but neither the overconsolidation ratio,  $K_0$  or  $I_D$  in overconsolidated soil can be estimated from the results of the CPT test.

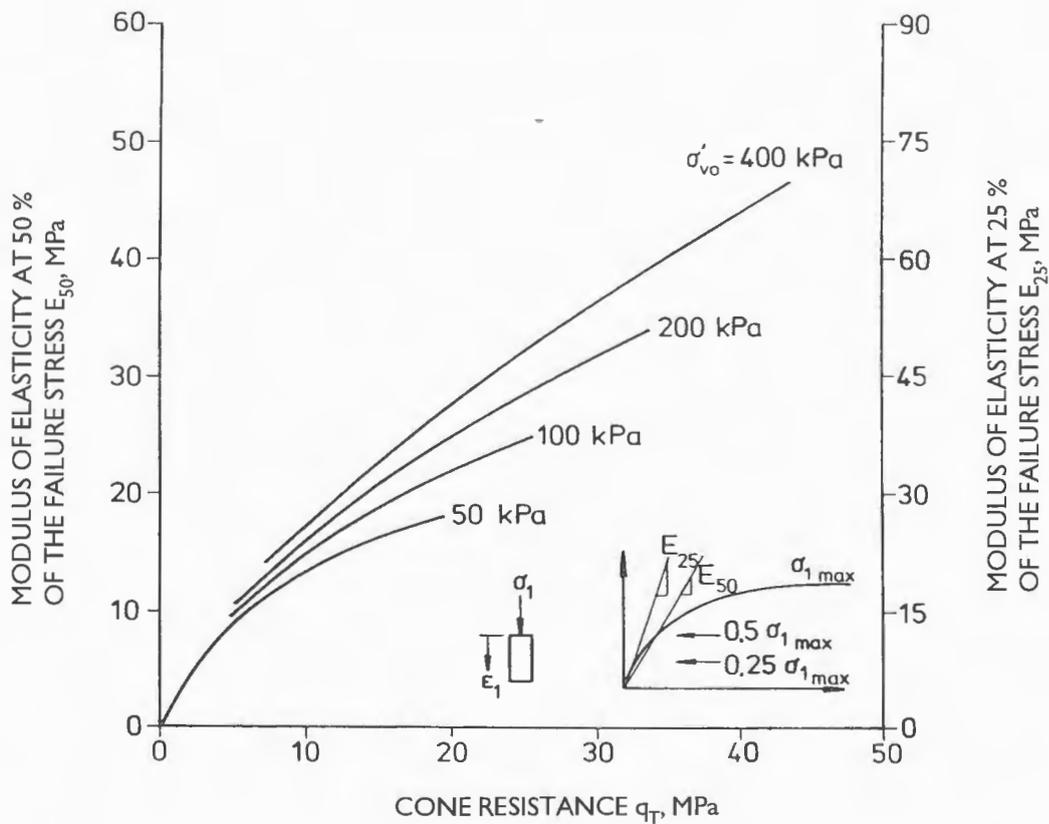
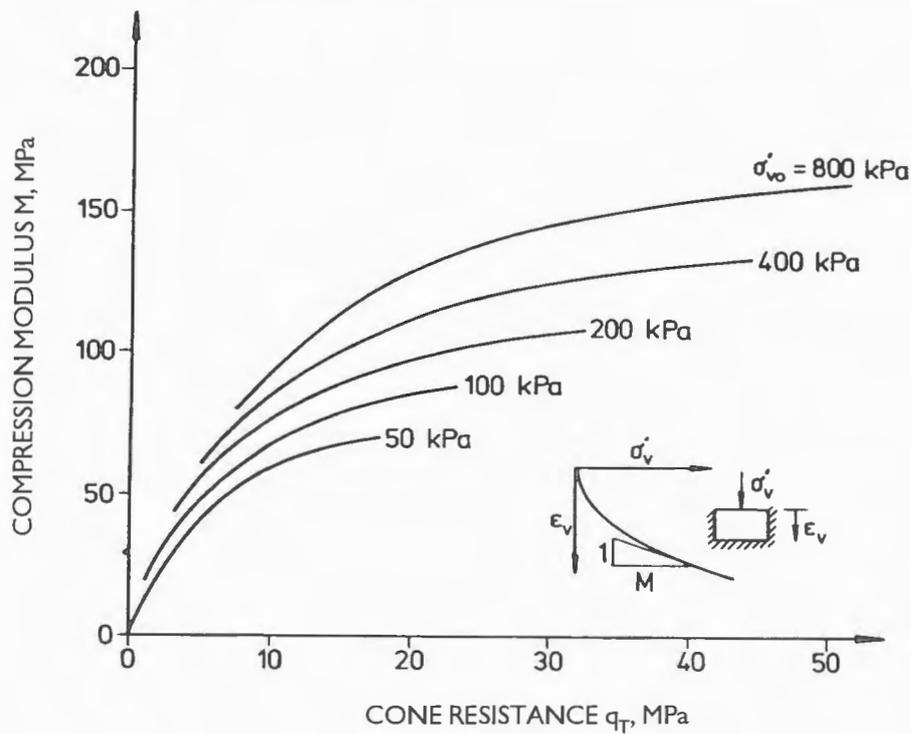
The calculation methods and moduli mentioned above are also mainly applicable in **normally consolidated sand**. Overconsolidation and compaction increase the stiffness of the soil considerably but this is reflected only to a minor degree in the CPT test results.

When these improved properties are to be fully utilised, another test method normally has to be applied.

Compression characteristics in *cohesive soil* should not be evaluated directly from the results of CPT tests, which in this type of soil are performed under almost fully undrained conditions. For evaluation of these properties, undisturbed sampling and oedometer tests are required.

However, moduli for calculation of settlements in overconsolidated cohesive soil and for calculation of initial elastic settlements are often estimated empirically from the undrained shear strength. For this purpose, also the undrained shear strength evaluated from the CPT test can be used.

More accurate measurements of the elastic properties of the soil, especially at small strains, can be made if the probe is equipped with an accelerometer and seismic tests are performed.



**Fig. 36. Diagrams for estimation of deformation properties in normally consolidated sand. (After Robertson and Campanella 1988).**

- a) Compression modulus
- b) Modulus of elasticity

## CHAPTER 13.

# Comparison with other empirical experience and test methods

For checking of the evaluated parameters and for comparison with parameters evaluated from other test methods, guiding values of relative density, friction angle and modulus of elasticity can be used, *Table 3*.

The given values are averages of the used correlations and do not include any built-in corrections or safety factors. The values are mainly applicable to normal quartz-feldspar sand and for the friction angle

corrections are made for silt and gravel according to the footnote below the table.

The relative density is given in terms of "relative stiffness", which cannot be translated directly into relative density.

The given moduli are guiding values for the approximate size and can only be used for very general estimates of settlements.

**Table 3. Comparison between results from different test methods in sand and their relations to relative density, friction angle and modulus of elasticity, (From SGI Information 2.)**

Relative stiffness	CPT test Cone resistance $q_T$ MPa	The cone resistance to the left normally correspond to the following values for $V_{im}$ and $H_{fa_{net}}$ respectively		$\phi'$ degrees *)	E MPa
		$V_{im}$ ht / 0.2 m	$H_{fa_{net}}$ blows / 0.2 m		
Very loose	< 2,5	< 10	< 5	29 - 32	< 10
Loose	2,5 - 5,0	10 - 30	5 - 10	32 - 35	10 - 20
Medium stiff	5,0 - 10	20 - 60	7 - 15	35 - 38	20 - 30
Stiff	10 - 20	40 - 100	10 - 40	38 - 42	30 - 60
Very stiff	> 20	> 80	> 30	> 42	> 60

\*) Given values apply to sand. For silt, a reduction of  $2^\circ$  is made by and for gravel  $2^\circ$  is added.

## CHAPTER 14.

# Examples

The results of CPT tests are normally presented and evaluated with the aid of a computer programme. Such a programme, CONRAD, has been produced at SGI. This programme is based on the procedures described in this Information. The results are mainly presented in terms of standardised plots and a diagram of evaluated soil types and soil properties.

In the latter diagram, the evaluated soil type is presented together with sub-designations regarding estimated stiffness and for cohesive soils, when appropriate, also with sub-designations regarding overconsolidation, siltiness and sensitivity. Furthermore, the estimated undrained shear strength in cohesive soil and friction angle and moduli in sand and silt and relative density in sand are presented respectively.

If a more elaborate evaluation has been made by using determined liquid limits, an additional diagram presenting the evaluated stress condition in cohesive soil can be plotted. When desired, a complete printout of all measured and evaluated parameters can also be made.

As typical examples of results from CPT tests, measured and evaluated parameters from a profile with soft and partly organic clay and a profile with sand, silt and stiff clay are shown.

### *Example 1:*

#### **PROFILE WITH SOFT CLAY**

The profile with soft clay and organic clay is from Tuve on the island of Hisingen on the border between the regions of Bohuslän and Västergötland. The test site is located within the municipality of Gothenburg just north of the area of the large landslide in 1977. The soil conditions are described in detail in SGI Reports Nos. 18 and 42.

The upper 6 metres of the profile consist of organic clay with infusions of plant remnants and shells. The dry crust is only half a metre thick. Below 6 metres depth, the organic content is insignificant but the shell

content increases. Between 6 and 8 metres depth, there is a pronounced change in the soil properties. First, there is a zone with infusion of thin silt seams and then follows a clay with higher values of both liquid limit, water content, undrained shear strength and preconsolidation pressure than the adjacent overlying clay layers. This clay layer continues down to about 23 metres depth, where the clay becomes more silty and layered with silt. Firm bottom in terms of a thin layer of sand on top of bedrock is found at 24 to 25 metres depth.

The liquid limit of the clay decreases from over 110 % just below the dry crust to about 95 % at 6 m depth. It goes down to about 80 % in the layers with a higher content of silt and shells and after the sudden jump in properties it again increases to about 95 % at 9 metres depth. It then gradually decreases to about 50 % at 23 metres depth. The water content is about 10 % higher than the liquid limit throughout the profile.

The density is about 1.4 t/m<sup>3</sup> in the organic clay, about 1.5 t/m<sup>3</sup> between 7 and 10 metres depth and then gradually increases to 1.7 - 1.8 t/m<sup>3</sup> in the bottom layers.

The free ground water level is located very high. At the time of the test, it was located about 0.4 m below the ground level but the area is seasonally flooded. The test site is located at the bottom of a valley and water infiltrates into the sand layer on top of the bedrock at the sides of the valley; the water pressure in the bottom layers is therefore artesian. At the time of the test, it corresponded to a free water surface located 3.25 metres above the ground surface.

Because of the pore pressure situation, the effective stresses in the ground are relatively low. The preconsolidation pressures and the undrained shear strength are also low but still correspond to an overconsolidation ratio of 1.5 - 2 in most of the profile.

Results from a CPT test performed according to test class 3 and evaluated by using of known values of the liquid limit are shown in *diagrams 1:1 - 1:4*

together with the relevant report.

Special tests with excess pore pressure dissipation at stops in the penetration showed that the clay at 6 and 8 metres depth can be classified as non-draining according to *Figure 26*, while the clay at 7 metres depth fell just below the limit for non-draining and can thereby be considered as partly draining.

*Example 2:*

**PROFILE WITH SAND, SILT AND MEDIUM STIFF TO STIFF CLAY**

This example comes from the investigations for a new railway bridge over Hybo canal on the line between Bollnäs and Ljusdal in Hälsingland. The tests were performed from the ice during wintertime.

Below 1 m water depth, the soil consists, according to unanimous results of dilatometer tests and CPT tests, of loose sand - silty sand down to a depth of 8 metres. Between 8 and 12 metres depth, there is a dense silt layer. Below this, there is a 4 - 5 metre thick medium stiff to stiff clay layer and at 15 - 17 metres there is a transition to dense silt and sand. The tests have not been continued further down because this has not been required for the problem at hand and would have required a more elaborate anchoring of the pushing equipment.

The pore pressures in the upper sand layer and in the dense bottom layers are hydrostatic from the water surface and this can be assumed to be the case also in the fine-grained soil layers.

Results from one of the CPT tests performed according to test class 2 and the relevant report are shown in *diagrams 2:1 - 2:4*.

A preliminary classification by using the programme indicated a few sand layers in the interval between 9.8 and 10.6 metres. With guidance from the measured pore pressures and calculated pore pressure ratio, the entire interval between 9.1 and 11.4 m has been designated as silt and the programme has thereafter been re-run.

# CPT test with measured parameters

Location/level:	N 200, +2,12	Ground water level:	0.40 m u my.	Equipment:	Geotech
Reference level:	markyta	Predrilling depth:	1 m	Stops in sounding:	24,57
P pres. measurement:	Yes	Start depth:	0.65 m	Observations:	artesiskt vottentryck
Hydrostatic P. pres.:	No	Stop depth:	24.66 m		
Filename:	exempel3.cpt	Predrilled material:	Torrskorpa		

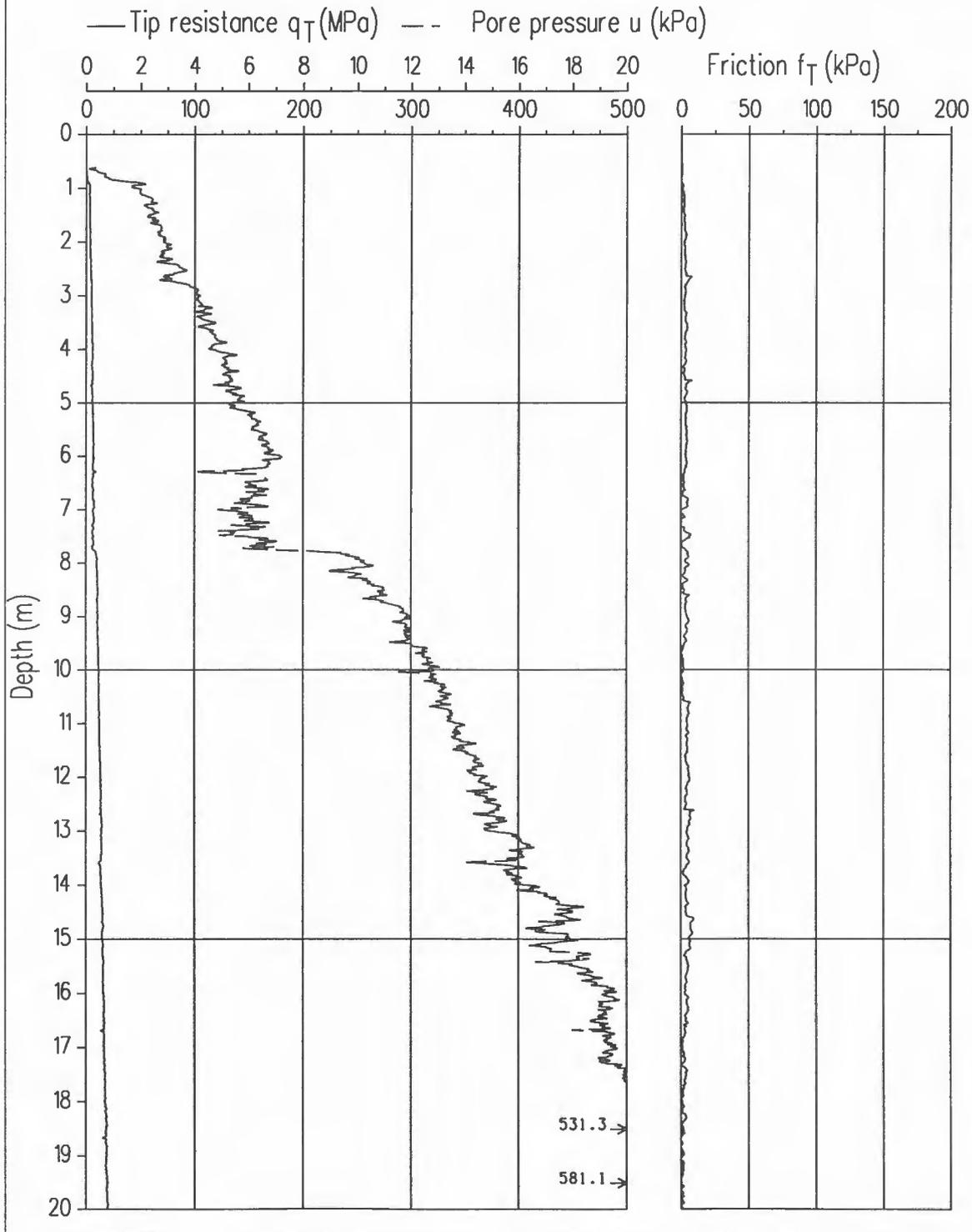


Diagram I:I. Standard presentation.

CPT-test

File name : EXEMPEL3.CPT

Test area : Tuve  
 Operator : K. Hidsjö  
 Date : 90-09-10  
 Designation : Hål 2  
 Location/level: N 200, +2,12  
 Ref. level : markyta

Ground water level : 0.40 m. b. markyta  
 Predrilling depth : 1 m. b. markyta  
 Start depth : 0.65 m. b. markyta  
 Stop depth : 24.66 m. b. markyta  
 Predrilled material : Torrskorpa  
 Equipment : Geotech  
 Stops in sounding : 24,57

Observations : artesiskt vattentryck

Geometry : Normal filter position  
 P Press. measurement : Yes  
 P Press. at zero reading : 3.00 kPa  
 Fluid in P Press. system : Glycerin

Pore pressure observations : 2 levels

Level [m]	Press. [kPa]
24.50	272.20

Calibration data cone 3065 Calibrated 90-09-01

Area factor	a:	0.580	b:	0.014
Internal friction	Oc:	8.000	Of:	0.500
Cross talk	c1:	0.000	c2:	0.000

Scale factors and measuring ranges [MPa]

Pore pressure		Friction		Tip resistance	
Range	Factor	Range	Factor	Range	Factor
0.50	4291	0.05	1209	0.50	791

Zeor reading	Before	After	Diff	Correction
Pore pressure	127.86	127.86	0.00 [kPa]	No
Friction	54.48	54.38	-0.10 [kPa]	No
Tip resistance	478.21	479.75	1.54 [kPa]	No

Depth correction : No

Sampling : 14 levels

From	To	Density	L limit	Classification
0.00	1.00	1.60	0.00	Crust
1.00	1.50	0.00	1.20	
1.50	2.50	0.00	1.15	
2.50	3.50	0.00	1.10	
3.50	4.50	0.00	1.02	
4.50	5.50	0.00	1.02	
5.50	6.50	0.00	1.02	
6.50	7.50	0.00	0.72	
7.50	8.50	0.00	0.82	
8.50	9.50	0.00	0.93	
9.50	10.50	0.00	0.88	
10.50	16.00	0.00	0.80	
16.00	20.00	0.00	0.70	
20.00	24.00	0.00	0.50	

Boundaries between layers : 6 levels

2.50	2.75	6.25	7.75	22.90	24.00
------	------	------	------	-------	-------

Irrelevant measurements : not given

**Diagram 1:2. Report.**

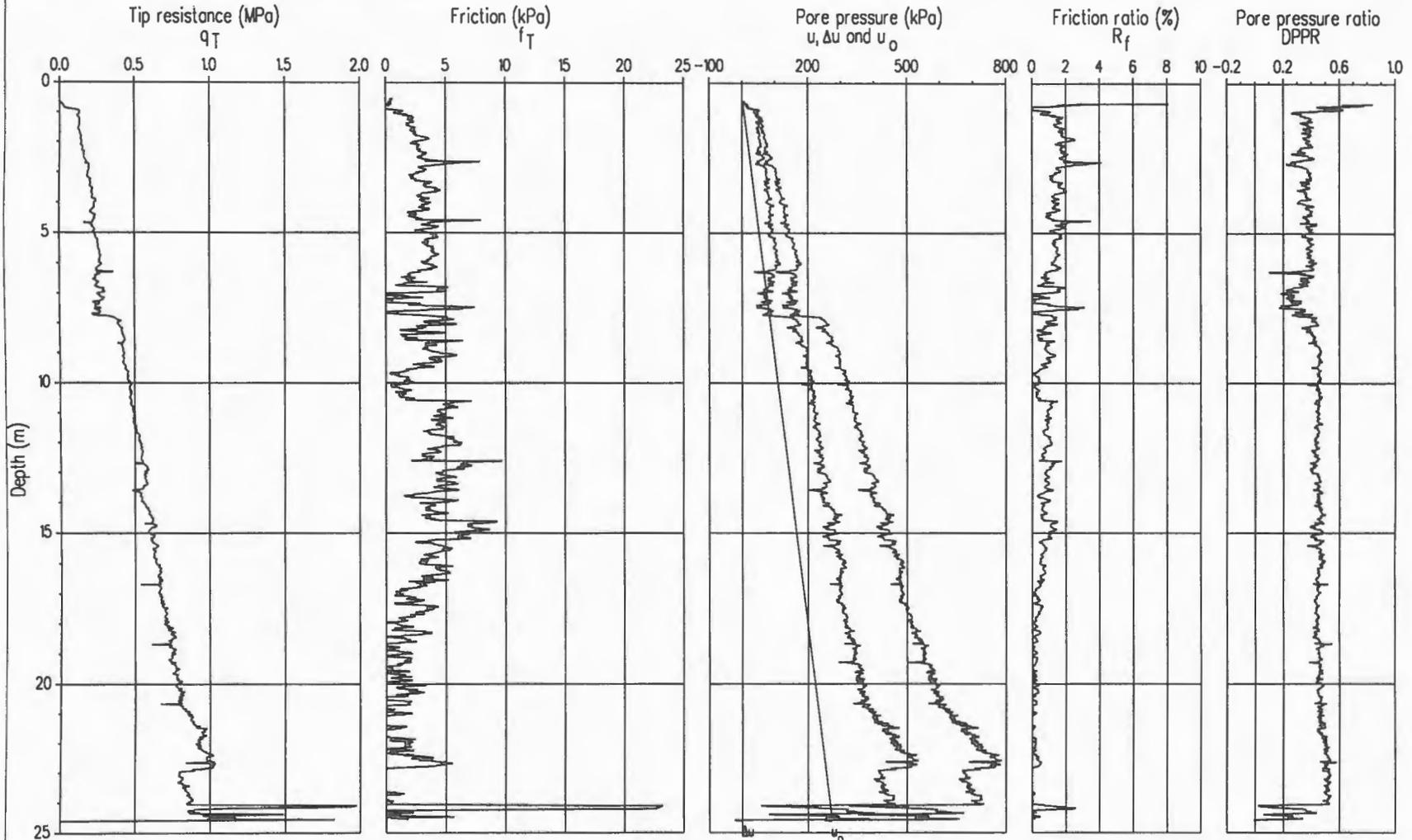
### CPT test with measured parameters

Location/level:	N 200, +2,12	Ground water level:	0.40 m u. m.	Equipment:	Geotech
Reference level:	markyta	Predrilling depth:	1 m	Stops in sounding:	24,57
P pres. measurement:	Yes	Start depth:	0.65 m	Observations:	ortesiskt vattentryck
Hydrostatic P. pres.:	No	Stop depth:	24.66 m		
Filename:	exempel3.cpt	Predrilled material:	Tarrskarpo		



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Designations, see SGI Information 15

Diagram I:3. Presentation in selected scales.

### CPT test evaluated according to SGI Info 15

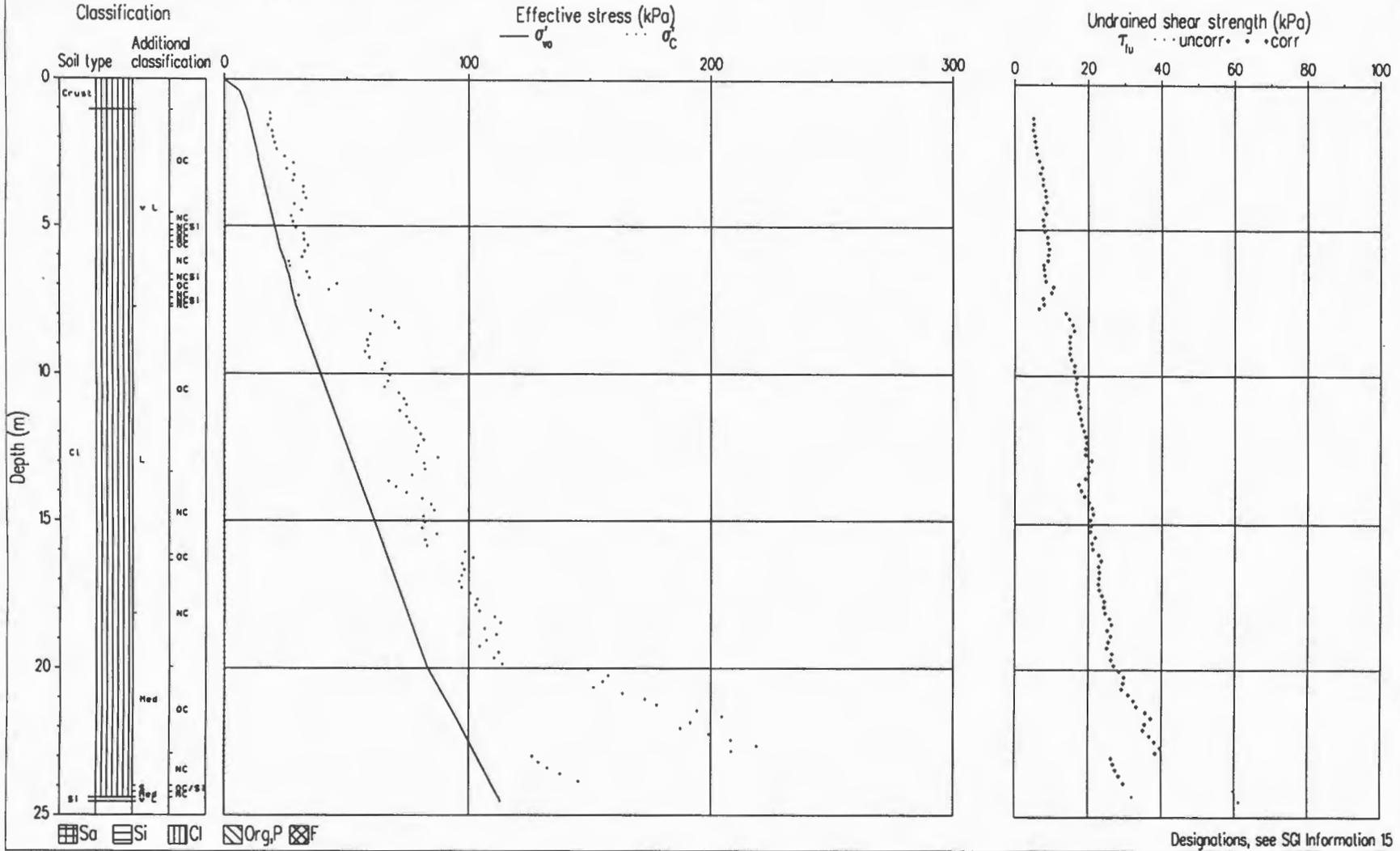
Location/level:	N 200, +2,12	Ground water level:	0.40 m u my.	Equipment:	Geotech
Reference level:	markyta	Predrilling depth:	1 m	Stops in sounding:	24,57
P pres. measurement:	Yes	Start depth:	0.65 m	Observations:	artesiskt vattentryck
Hydrostatic P. pres.:	No	Stop depth:	24.66 m		
Filename:	exempel3.cpt	Predrilled material:	Torrskorpa		



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Diagram 1:4. Soil classification and evaluated properties.



# CPT test with measured parameters

Location/level:	+0,53	Ground water level:	0.00 m u my.	Equipment:	Geotech
Reference level:	vattenyta	Predrilling depth:	0.9 m	Stops in sounding:	
P pres. measurement:	Yes	Start depth:	0.90 m	Observations:	
Hydrostatic P. pres.:	Yes	Stop depth:	17.45 m		
Filename:	exempel2.cpt	Predrilled material:	Vatten		

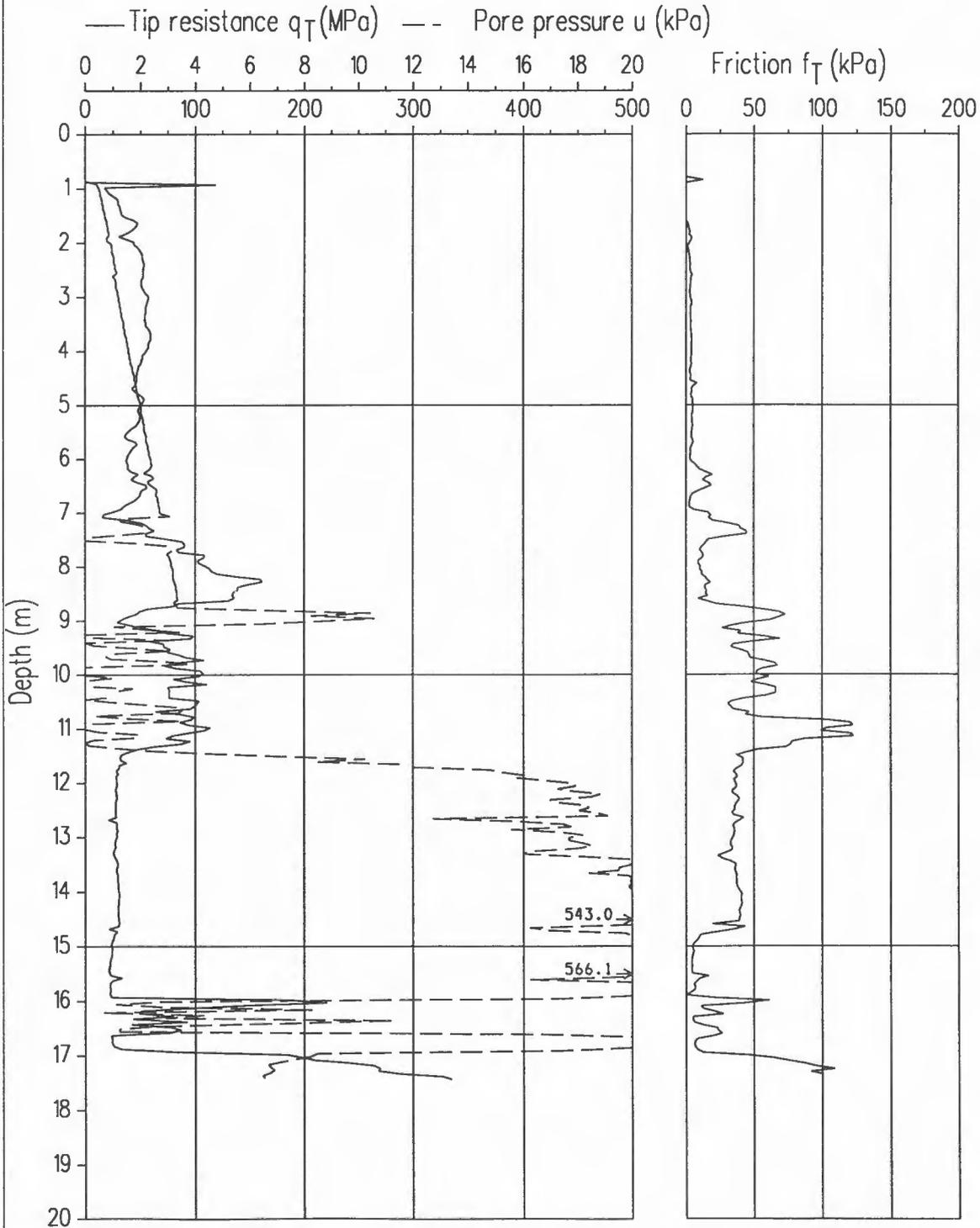


Diagram 2:1. Standard presentation.

CPT-test

File name : EXEMPEL2.CPT

Test area : Hybo Banverket  
 Operator : K. Hidsjö  
 Date : 92-01-19  
 Designation : 199H5  
 Location/level: +0,53  
 Ref. level : vattenyta

Ground water level : 0.00 m. b. vattenyta  
 Predrilling depth : 0.9 m. b. vattenyta  
 Start depth : 0.90 m. b. vattenyta  
 Stop depth : 17.45 m. b. vattenyta  
 Predrilled material : Vatten  
 Equipment : Geotech  
 Stops in sounding :

Observations :

Geometry : Normal filter position  
 P Press. measurement : Yes  
 P Press. at zero reading : 0.00 kPa  
 Fluid in P Press. system : Glycerin

Pore pressure observations : Hydrostatic from G.W.L

Calibration data cone 3087 Calibrated 89-10-02  
 Area factor a: 0.580 b: 0.014  
 Internal friction Oc: 8.000 Of: 1.000  
 Cross talk c1: 0.001 c2: 0.001

Scale factors and measuring ranges [MPa]  

Pore pressure		Friction		Tip resistance	
Range	Factor	Range	Factor	Range	Factor
1.00	2756	0.10	1148	20.00	3150

Zeor reading	Before	After	Diff	Correction
Pore pressure	112.50	112.50	0.00 [kPa]	No
Friction	27.01	27.43	0.43 [kPa]	Average before/after
Tip resistance	1968.63	1984.13	15.50 [kPa]	Average before/after

Depth correction : No

Sampling : 2 levels		Density	L limit	Classification
From	To			
0.00	0.90	1.00	0.00	Exc
0.90	1.10	2.00	0.00	Sa Med

Boundaries between layers : 11 levels  
 6.05 6.70 7.00 7.45 8.80 9.10 11.40 14.80  
 16.00 16.50 17.00

Irrelevant measurements : not given

**Diagram 2:2. Report.**

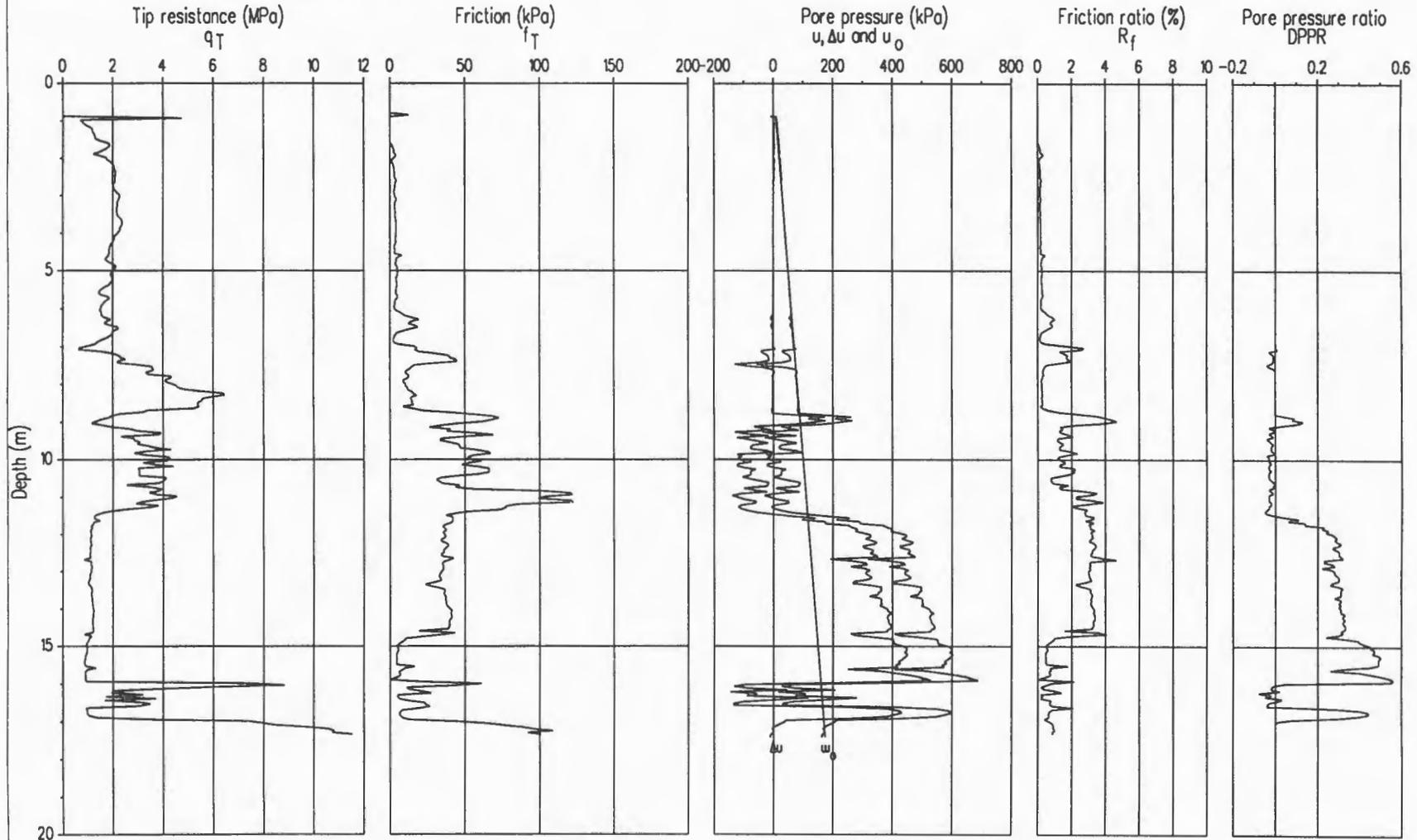
### CPT test with measured parameters

Location/level:	+0,53	Ground water level:	0.00 m u my.	Equipment:	Geotech
Reference level:	vallentyta	Predrilling depth:	0.9 m	Stops in sounding:	
P pres. measurment:	Yes	Start depth:	0.90 m	Observations:	
Hydrostatic P. pres.:	Yes	Stop depth:	17.45 m		
Filename:	exempel2.cpt	Predrilled material:	Vallen		



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Diagram 2.3. Presentation in selected scales.

### CPT test evaluated according to SGI Info 15

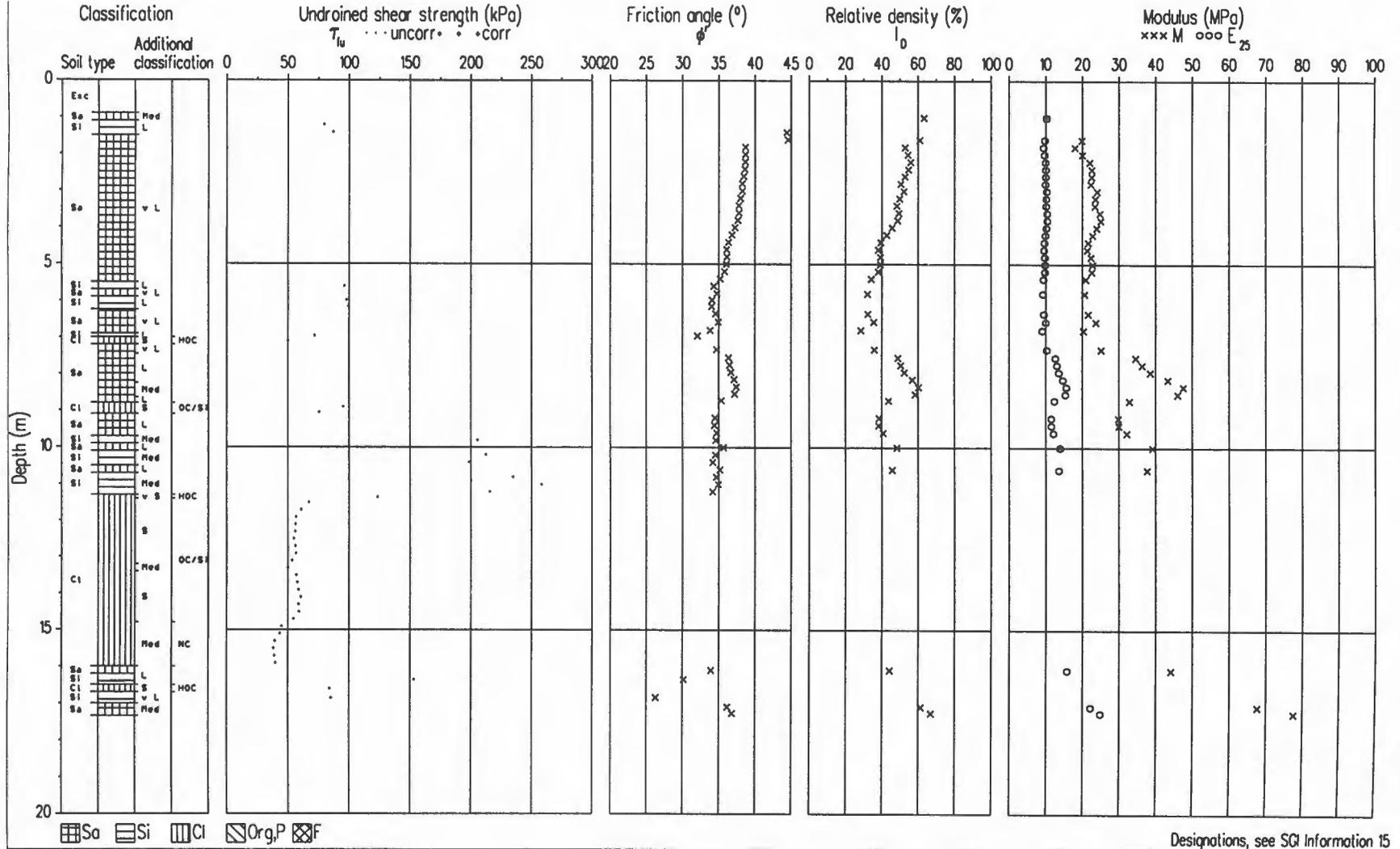
Location/level: +0,53      Ground water level: 0.00 m u my.      Equipment: Geotech  
 Reference level: vattenyta      Predrilling depth: 0.9 m      Stops in sounding:  
 P pres. measurement: Yes      Start depth: 0.90 m  
 Hydrostatic P. pres.: Yes      Stop depth: 17.45 m      Observations:  
 Filename: exempel2.cpt      Predrilled material: Vatten



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 Approved:

Diagram 2.4. Soil classification and evaluated properties.



Designations, see SGI Information 15

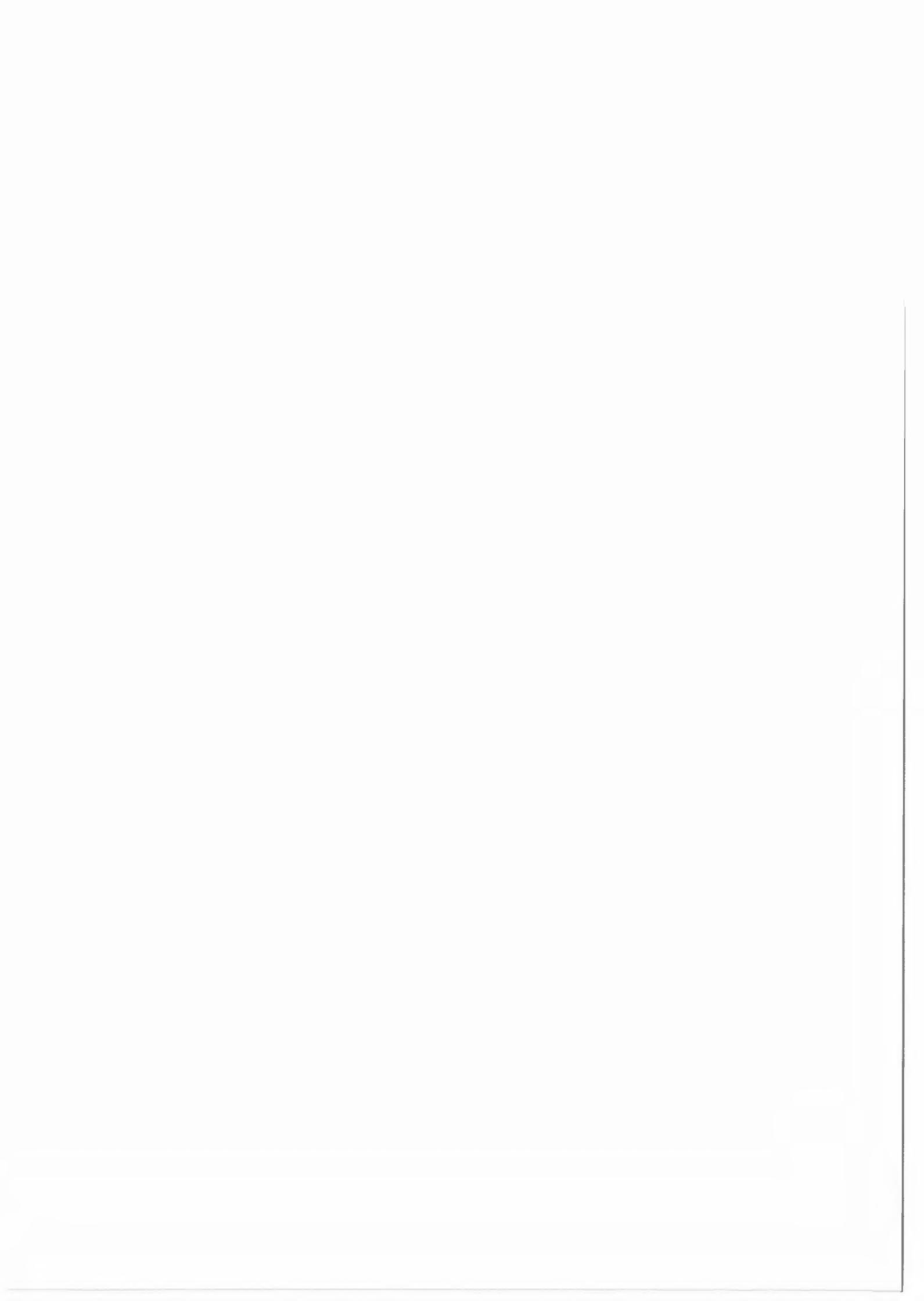
## CHAPTER 15.

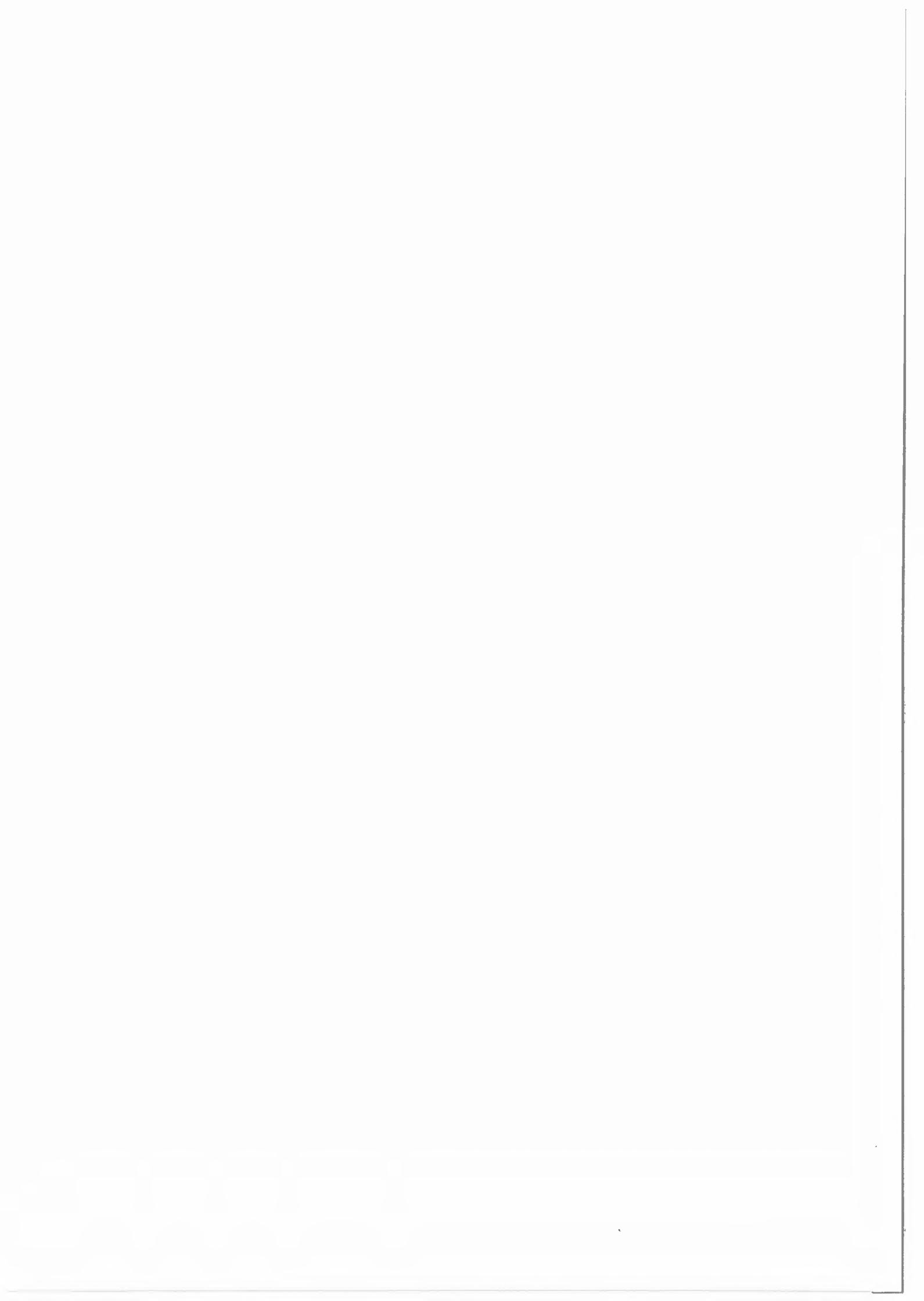
# Literature

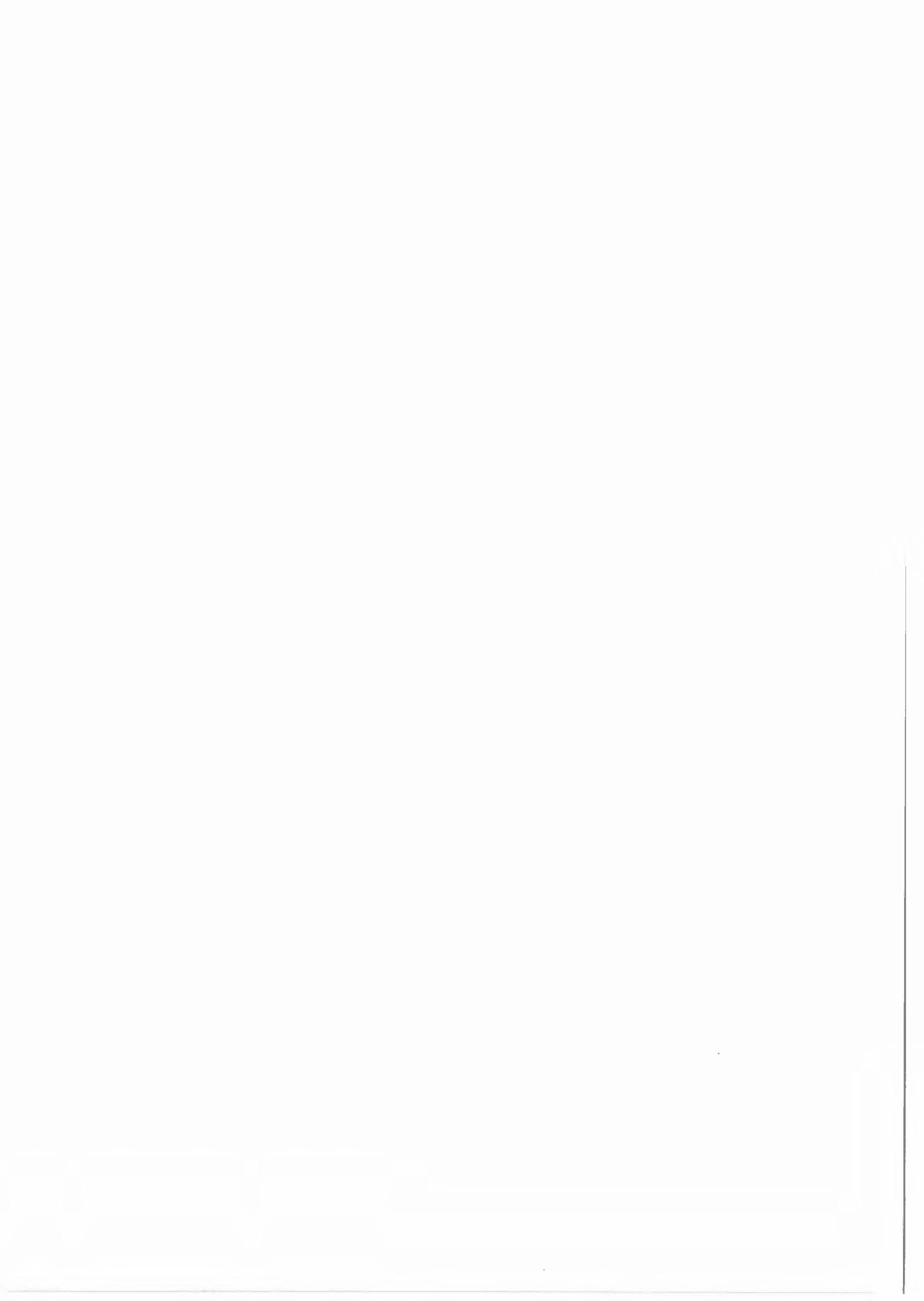
- Baldi, G., Bellotti, R., Ghionna, V. N. Jamiolkowski, M. and Pasqualini, E. (1986).** Interpretation of CPT's and CPTU's, Second part: Drained Penetration of Sands. Proceedings of the Fourth International Geotechnical Seminar. Singapore.
- Bellotti, R., Ghionna, V. N., Jamiolkowski, M., Lancelotta, R. and Robertson, P. K. (1989).** Shear strength of sand from CPT. Proceedings, 12th International Conference on Soil Mechanics and Foundation Engineering, Rio de Janeiro, Vol. 1, pp. 179-184.
- Bergdahl, U (1984).** Geotekniska undersökningar i fält. Statens geotekniska institut, Information 2, Linköping.
- Bergdahl, U. och Eriksson, U. (1983).** Bestämning av jordegenskaper med sondering - en litteraturstudie. Statens geotekniska institut, Rapport No 22, Linköping.
- International Society for Soil Mechanics and Foundation Engineering ISSMFE (1989).** Report of the ISSMFE Technical Committee on Penetration Testing of Soils -TC 16 with Reference Test Procedures, CPT - SPT - DP - WST. Statens geotekniska institut, Information 7, Linköping.
- Jamiolkowski, M., Ghionna, V. N., Lancelotta, R. and Pasqualini, E. (1988).** New correlations of penetration tests for design practice. Proceedings of the First International Symposium on Penetration Testing, ISOPT-1, Orlando Florida, Vol. 1, pp. 263-296.
- Lancelotta, R. (1983).** Analisi di Affidabilità in Ingegneria Geotecnica. Atti Istituto Scienza Costruzioni. No. 625, Politecnico di Torino.
- Larsson, R. and Mulabdic, M. (1991).** Piezocone Tests in Clay. Statens geotekniska institut, Rapport No 42, Linköping.
- Larsson, R. och Sällfors, G. (1985).** Nyare in situ metoder för bedömning av lagerföljd och egenskaper i jord. Statens geotekniska institut, Information 5, Linköping.
- Lunne, T. and Christoffersen, J. P. (1983).** Interpretation of Cone Penetrometer Data for Off-shore Sands. Proceedings of the Fifteenth Annual Off-shore Technology Conference, Houston Texas. Också i Publikasjon Nr. 156, Norges Geotekniske Institutt, Oslo.
- Marchetti, S. (1985).** On the field determination of  $K_0$  in sand. Panel discussion, 11th International Conference on Soil Mechanics and Foundation Engineering, San Francisco, Session 2 A.
- Meigh, A. C. (1988).** Cone Penetration Testing - Methods and Interpretation. CIRIA, Ground Engineering Report: In Situ Testing.
- Mulabdic, M., Eskilson, S. och Larsson, R. (1990).** Calibration of Piezocones for Investigations in Soft Soils and Demands for Accuracy of the Equipments. Statens geotekniska institut, Varia 270, Linköping.
- Robertson, P. K. (1988).** Soil classification using the cone penetration test. Discussion in Penetration testing in the UK, pp. 177-179, Thomas Telford, London. Also in Canadian Geotechnical Journal, Vol. 12, No. 1, 1990, pp. 151-158.
- Robertson, P. K. and Campanella, R. G. (1988).** Guidelines for using the CPT, CPTU and Marchetti DMT for geotechnical design. U.S. Department of Transportation. Report No. FHWA-PA-87-022+-84-24. Vol. 2.
- Svenska Geotekniska Föreningen - SGF (1992).** Rekommenderad standard för kombinerad spetsportryckssondering, CPT. Stockholm.

## **Previous publications related to the project “In situ methods”:**

- Larsson, R. och Sällfors, G. (1985).** Nyare In-Situ metoder för bedömning av lagerföljd och egenskaper i jord. Statens geotekniska institut, Information 5, Linköping.
- Larsson, R. och Eskilson, S. (1988).** Kalibrering av kombinerade spetstryck- portrycksonder i laboratorium. Statens geotekniska institut, Varia 223, Linköping.
- Larsson, R. och Eskilson, S. (1989a).** Dilatometerförsök i lera. Statens geotekniska institut, Varia 243, Linköping.
- Larsson, R. och Eskilson, S. (1989b).** Dilatometerförsök i organisk jord. Statens geotekniska institut, Varia 258, Linköping.
- Larsson, R. (1989).** Dilatometerförsök; En in situ metod för bestämning av lagerföljd och egenskaper i jord; Utförande och utvärdering. Statens geotekniska institut, Information 10, Linköping.
- Mulabdic, M., Eskilson, S. och Larsson, R. (1990).** Calibration of Piezocones for Investigations in Soft Soils and Demands for Accuracy of the Equipments. Statens geotekniska institut, Varia 270, Linköping.
- Larsson, R. and Mulabdic, M. (1991).** Shear Moduli in Scandinavian Clays. Statens geotekniska institut, Rapport No 40, Linköping.
- Larsson, R. and Mulabdic, M. (1991).** Piezocone Tests in Clay. Statens geotekniska institut, Rapport No 42, Linköping.







# **SGL Information**

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