Modelling of Groundwater Conditions in Silts and Fine Sands
A study of induced groundwater changes based on laboratory and full-scale field tests

Marius Tremblay
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MARIUS TREMBLAY


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Preface

The present thesis deals with the analysis and modelling of variations in the groundwater conditions in soils influenced by capillary forces. The major part of the study consists in laboratory and full-scale field tests made in silts and fine sands.

The research was carried out at the Department of Geotechnical Engineering of Chalmers University of Technology (CTH), and at the Swedish Geotechnical Institute (SGI), under the supervision of Prof. Göran Sällfors. His guidance and encouragement have been invaluable for this project, and I am very grateful to him.

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Finalement, je voudrais remercier mes parents et toute ma famille pour leur support inconditionnel et leur encouragement dans tout ce que j'entreprends, peu importe la distance qui nous sépare.

Göteborg and Linköping, November 1995

Marius Tremblay
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Summary

Introduction
Every day, the geotechnical engineer encounters problems closely related to the groundwater conditions in the soil that he/she is investigating. These problems may be of varying importance and may require different levels of analysis before a solution is found.

Even though our understanding of groundwater conditions and their variation with time is steadily improved, it is still clearly insufficient in many areas. The purpose of the present study is to increase the knowledge of the interaction between soil and groundwater in silty materials, both as recorded behaviour during different tests and as simulated behaviour using a numerical model.

Measurement of soil matric suction
In order to study and eventually model the behaviour of unsaturated soils, negative pore water pressures must be measured. The quality of the predictions to be made is greatly influenced by the quality of these measurements. Therefore, an investigation of different instruments available on the market was initiated in order to study the behaviour of some of the most commonly used instruments.

For the investigation, two instruments making direct measurement were chosen together with two sensors using indirect techniques. These four instruments were tested in order to control their accuracy and response when measuring soil matric suction under different conditions.

To perform the study, a new laboratory equipment was built. The equipment consisted in a large cylinder with a diameter of 1.0 m, which could reach a height of 1.6 m by assemblage of 0.4 m rings, Figure S.1. A water reservoir situated on the outside of the cylinder was connected to the bottom of the cylinder and the water level in the cylinder could therefore easily be controlled by varying the level of the water reservoir. The instruments can be installed at any
level in the soil specimen and the water level can be changed in different ways in order to study both steady and transient states.

The results from the measurements made with the Soil Moisture tensiometers and the BAT-piezometers are presented in Figure S.2. The instruments were placed at different depths below the soil surface and the results are shown as recorded water level and can be compared with the real water level in the cylinder. As seen from the results, both types of measuring system responded directly and correctly to the fluctuations in the water level, showing a good agreement with each other.

The investigation showed that tensiometers and piezometers are well suited for measurement of variations in the pore pressure in the vadoze zone. The maximum value of negative pressures which can be measured with the piezometer is usually assumed to be as large as for the tensiometer, i.e. about -90 kPa, when
using the available high air-entry-value filter. However, many factors can affect the long term functioning of the piezometer, such as leakage through the membrane and air entry during the measuring procedure.

**Study of the behaviour of partially saturated soils**

A number of tests covering different situations have been made in order to study changes in groundwater conditions resulting from various activities. In the first part of the study, the behaviour of unsaturated soils was studied in laboratory tests performed under fully controlled conditions. In these tests a lowering of the groundwater table in a soil column was generated, in order to obtain a transition from saturated to unsaturated conditions. This also gave the possibility of studying the behaviour of partially saturated soils when submitted to infiltration from above during further stages of the tests.

The equipment used for the laboratory tests consisted of a column with a total height of 3.0 m and a diameter of 150 mm. At the bottom of the column, there was a small chamber which could be filled with water in contact with the soil through a filter plate. A water reservoir, whose level could be freely regulated, was connected to the bottom of the column. When the soil was placed in the column, the water level in the soil could be controlled by moving the reservoir to selected levels.
Two tests were performed, one with fine sand and a second with silt. **Figure S.3** presents the results from the second test, in which the response from the tensiometers and the piezometers was in very good agreement with the actual changes in water level in the column. The measurements made with these four instruments placed inside the soil column at two different levels, 0.7 and 1.7 m below the surface of the soil, confirmed the behaviour observed in the filter tubes located along the cylinder.

![Graph](image)

**Figure S.3.** Comparison between observed and simulated behaviour of a laboratory test.

**Full-scale field test**
A field test consisting in measurements of changes in the groundwater conditions during pumping tests was made at one site located outside Linköping, Sweden. The purpose of the test was to simulate the effect of an excavation under well defined conditions, and to study the changes in the groundwater conditions in the immediate surroundings of the excavation. To obtain a lowering of the water table, a drainage system was installed about 2.5 m below the ground surface. The system consisted of a drainage pipe about 100 m long connected to a well in which the water level could be lowered by pumping, **Figure S.4.** The natural groundwater level in the area was between 0.5 and 1.0 m below the ground surface, which means that a groundwater lowering of about 1.5-2.0 m could be achieved with this installation.
Measurements were made during these pumping tests with a total of forty-seven instruments, including open standpipes, piezometers and tensiometers. These measurements were later used for comparison with the results obtained from simulation with the computer program, see Figure S.5, where measured values are compared with those obtained from the numerical simulations. The tests also gave the opportunity of studying the behaviour of different measuring instruments under field conditions.

Figure S.4. Drainage installation used in field test.

Figure S.5. Comparison between observed and simulated behaviour of a field test.
Conclusions
The investigation performed on four different types of measuring equipment showed that instruments directly measuring the pore pressure, such as tensiometers and piezometers, are well suited for recording negative pore pressures in the range recorded in this study. The measuring principle of these instruments enables measurement of suctions down to about -90 kPa, provided that they are equipped with filter elements with high air entry values. Variations in pore pressure can also be registered with good accuracy using these types of instrument. The accuracy of the measurements is mainly controlled by the type of reading instrument (manometer or pressure transducer) used.

The measurements performed in the laboratory in a silt column showed that the pressure variations in the tension-saturated zone, and at levels immediately above the tension-saturated zone, are directly controlled by the water level in the soil column. These tests also showed that the behaviour of unsaturated soil is not only controlled by the groundwater level in the soil, but also by the water retention characteristics of the soil. A decrease in pore pressure corresponds to the lowering of the water level as long as the pore-water is in full contact with the water in the saturated zone. When this contact is broken due to drainage of water (decrease in water content), the suction reaches a stabilized level, independently of the position of the water table. Consequently, the variations in suction in the unsaturated zone is directly related to the thickness of the capillary zone, as it was observed in the test.

The simulations of variations in pore pressures recorded during the various tests performed in the laboratory and in the field have pointed out some important factors to consider. The results showed that the numerical simulation of groundwater lowering in saturated soil is relatively straightforward, as long as the hydraulic properties of the soil have been determined with good accuracy. The concordance between the simulation and the actual behaviour of the soil depends also on the accuracy to which the stratigraphy of the soil deposit has been established and modelled.
### List of Symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A$</td>
<td>area of the cross-section</td>
</tr>
<tr>
<td>$C$</td>
<td>storage capacity (function)</td>
</tr>
<tr>
<td>$C_p$</td>
<td>coefficient of proportionality</td>
</tr>
<tr>
<td>$d$</td>
<td>diameter</td>
</tr>
<tr>
<td>$d_{16}$</td>
<td>effective grain diameter</td>
</tr>
<tr>
<td>$g$</td>
<td>gravitational acceleration</td>
</tr>
<tr>
<td>$h$</td>
<td>pressure head</td>
</tr>
<tr>
<td>$H$</td>
<td>total head</td>
</tr>
<tr>
<td>$h_a$</td>
<td>air entry pressure or bubbling pressure</td>
</tr>
<tr>
<td>$h_c$</td>
<td>capillary head</td>
</tr>
<tr>
<td>$h_r$</td>
<td>matric soil suction</td>
</tr>
<tr>
<td>$h_{rm}$</td>
<td>matric soil suction</td>
</tr>
<tr>
<td>$h_{so}$</td>
<td>osmotic soil suction</td>
</tr>
<tr>
<td>$h_{nt}$</td>
<td>total soil suction</td>
</tr>
<tr>
<td>$i$</td>
<td>hydraulic gradient</td>
</tr>
<tr>
<td>$K$</td>
<td>hydraulic conductivity or coefficient of permeability</td>
</tr>
<tr>
<td>$k$</td>
<td>specific or intrinsic permeability</td>
</tr>
<tr>
<td>$K_{lab}$</td>
<td>hydraulic conductivity measured in laboratory test</td>
</tr>
<tr>
<td>$K_{in situ}$</td>
<td>hydraulic conductivity measured in field test</td>
</tr>
<tr>
<td>$K_{rel}$</td>
<td>relative hydraulic conductivity</td>
</tr>
<tr>
<td>$K_{sat}$</td>
<td>saturated hydraulic conductivity</td>
</tr>
<tr>
<td>$K_{unsat}$</td>
<td>unsaturated hydraulic conductivity</td>
</tr>
<tr>
<td>$K_S$</td>
<td>measured saturated hydraulic conductivity</td>
</tr>
<tr>
<td>$K_{SC}$</td>
<td>calculated saturated hydraulic conductivity</td>
</tr>
<tr>
<td>$K_x$</td>
<td>hydraulic conductivity in x-direction</td>
</tr>
<tr>
<td>$K_y$</td>
<td>hydraulic conductivity in y-direction</td>
</tr>
<tr>
<td>$K_z$</td>
<td>hydraulic conductivity in z-direction</td>
</tr>
<tr>
<td>$l$</td>
<td>length</td>
</tr>
<tr>
<td>$m_{vG}$</td>
<td>soil parameter in vanGenuchten equation</td>
</tr>
<tr>
<td>$M$</td>
<td>specific moisture capacity</td>
</tr>
<tr>
<td>$n_1$</td>
<td>number of pore classes in Green and Corey’s equation</td>
</tr>
<tr>
<td>$n_2$</td>
<td>number of pore classes in Green and Corey’s equation</td>
</tr>
<tr>
<td>$m_w$</td>
<td>slope of storage curve</td>
</tr>
<tr>
<td>$n$</td>
<td>porosity</td>
</tr>
<tr>
<td>$n_{vG}$</td>
<td>soil parameter in vanGenuchten equation</td>
</tr>
</tbody>
</table>
\( p \) = pressure
\( P_{\text{air}} \) = air pressure
\( p_c \) = capillary pressure
\( P_{\text{GC}} \) = parameter in Green and Corey’s equation
\( P_w \) = water pressure
\( p_{\text{water}} \) = water pressure
\( Q \) = groundwater flow
\( q \) = water flow
\( q_x \) = water flow in x-direction
\( q_y \) = water flow in y-direction
\( q_z \) = water flow in z-direction
\( r_w \) = radius of meniscus
\( S \) = storativity
\( S_r \) = degree of saturation
\( S_{ro} \) = specific retention
\( S_k \) = specific storage
\( S_y \) = specific yield
\( S_{\text{wo}} \) = irreducible degree of saturation
\( u_{\text{air}} \) = air pressure
\( u_{\text{water}} \) = porewater pressure
\( V \) = bulk volume
\( v \) = flow velocity or specific discharge
\( V_{w} \) = water volume
\( z \) = elevation related to a datum

Greek letters
\( \alpha \) = contact angle
\( \alpha_k \) = coefficient in Kozeny’s equation (grain shape)
\( \alpha_{\text{ve}} \) = vanGenutchen coefficient
\( \beta_i \) = coefficient of compressibility
\( \Phi \) = potential
\( \gamma \) = surface tension of water
\( \gamma_w \) = volumetric weight of water
\( \lambda \) = coefficient in Brooks and Corey’s equation
\( \mu \) = dynamic viscosity
\( \theta \) = volumetric water content
\( \theta_o \) = lowest volumetric water content on the pF-curve
\( \theta_r \) = retention volumetric water content
\( \theta_s \) = volumetric water content at full saturation
\( \rho \) = density
\( \rho_w \) = water density
\( \sigma \) = interfacial tension
\( \sigma_{ik} \) = interfacial tension between two substances i and k
\( \nu \) = kinematic viscosity
Chapter 1.
Introduction

1.1 ANALYSIS OF VARIATIONS IN GROUNDWATER IN GEOTECHNICAL ENGINEERING
Every day, the geotechnical engineer encounters problems closely related to the groundwater conditions in the soil that he/she is investigating. These problems may be of varying importance and may require different levels of analysis before a solution is found.

Many times, the solution to such problems is chosen strictly on the basis of previous experience. For instance, the variations in the groundwater conditions in the immediate surrounding of a large excavation or any other construction work resulting in a lowering of the groundwater table, are often estimated using simplified methods. One of the reasons for this is the limited knowledge and use of analytical or numerical methods available for more detailed analysis.

1.2 OBJECTIVE AND SCOPE OF THE STUDY
Even though our understanding regarding the groundwater conditions and their variation with time is steadily improved, it is still clearly insufficient in many areas. The purpose of the present study is to increase the knowledge of the interaction between soil and groundwater in silty materials, both as recorded behaviour during different tests and as simulated behaviour using a numerical model.

The study starts with an investigation of different equipment used for measuring the groundwater conditions in the partially saturated zone, i.e. above the water table. This investigation performed in different conditions, permitted to evaluate the advantages and disadvantages of the different instruments, as well as their accuracy and field of utilization.

In order to acquire a better knowledge of the interaction between the soil and the water in the partially saturated zone, a laboratory investigation was per-
formed, including a number of tests with soils having different properties. These tests were later simulated with a numerical model for comparison with the observed behaviour.

Field tests were also made to induce a lowering of the groundwater table and monitor the changes in the groundwater conditions in the surroundings. A drainage system was installed specially for this purpose in a test field. These tests were also simulated with the numerical model after estimation of the soil properties on the site. Finally, a comparison was made between the measured and computed values.
Chapter 2.
Literature Survey

2.1 INTRODUCTION
In order to study the groundwater conditions in fine-grained materials such as silty soils, it is necessary to have a good knowledge of both the prevailing conditions and the behaviour of groundwater flow in saturated as well as unsaturated soils. The present literature survey mainly deals with the water conditions in partially saturated soils not only because the phenomena involved are more complex, but also due to the fact that knowledge in this field is more limited among geotechnical engineers than knowledge regarding the behaviour of fully saturated soils.

The different phenomena controlling the water distribution and its variations in space and time are well described in many textbooks and will therefore be only briefly presented here. Among the most important factors are the capillary forces acting in partially saturated soils and hysteresis of the water distribution relationships.

One of the most important steps in the modelling process is the determination of the hydraulic characteristics of the soil. Properties such as hydraulic conductivity, water retention relation and storage capacity have to be estimated using appropriate measuring methods. When direct measurement cannot be made, empirical relations can be used.

It is also important to be able to measure the groundwater pressure both in partially and fully saturated soils if the initial groundwater conditions and their variations are to be monitored. Different measuring techniques will be described and evaluated here.

2.2 BASIC DEFINITIONS

2.2.1 Groundwater formation and distribution
The endless circulation of water between oceans, atmosphere and land is called
the *hydrologic cycle*. A complete description of the hydrologic cycle can be found in any textbook on the subject (e.g. Freeze and Cherry, 1979; Fetter, 1980).

The groundwater distribution shows important variations in space and time due to the variability of a large number of parameters such as topography, stratigraphy, climatic conditions, etc. Human activities such as drainage installations, excavations or pumping wells contribute to an increase in these variations at a local level. The study of the horizontal distribution of the groundwater therefore requires a good knowledge of the geology of the area, the flow parameters of the different soil layers and other factors affecting the water conditions, such as the climatic conditions.

The vertical distribution and its variations are often controlled by the stratigraphy. Essentially, there is a *saturation zone* where all the pores are filled with water (fully saturated soil), and an overlying *aeration zone* where the pores may contain both air and water (partially saturated soil).

By definition, the saturation and aeration zones have the following characteristics (Freeze and Cherry, 1979):

- **saturation zone:**
  - situated below the water table,
  - pore-water pressure is higher than atmospheric,
  - pressure head $h$ is positive,
  - volumetric water content is equal to the porosity,
  - degree of saturation is equal to one.

- **aeration zone:**
  - situated above the water table,
  - pore-water pressure is lower than atmospheric,
  - pressure head $h$ is negative,
  - volumetric water content is lower than the porosity,
  - degree of saturation is lower than one.

There is however a zone often observed in fine-grained materials, to which neither of these definitions apply. This zone, situated immediately above the water table, is characterized by the fact that all the pores are filled with water by capillarity, even though the pore-water pressure is lower than atmospheric. This zone, called the *tension-saturated zone*, shows therefore characteristics...
commonly attributed to both the saturated and the unsaturated zones. It will be described in more detail in section 2.3.3.

The water table is defined as the level where the pore-water pressure is equal to the atmospheric pressure. In coarse materials, it is the limit between the fully saturated and the partially saturated zones. In fine grained materials, the position of the water table must be determined according to the above definition, since the saturated zone can reach higher levels due to capillarity. The groundwater is defined as the water contained in the pores located in the saturated zone, and the soil moisture is the water contained in the pores of the unsaturated zone. In agronomy and agriculture, the term groundwater is sometimes used to denote all water in the soil (fully and partially saturated zones) (Bear, 1979).

The aeration zone may be divided into different sub-zones, see Figure 2.1. The capillary fringe is situated directly above the water table and reaches up to different heights depending on the soil type and its relative density. The pressure in this zone is lower than the atmospheric pressure and can be calculated at any point knowing its position over the water table. The water content in this zone is often close to full saturation.

The soil water zone is located below the ground surface and has a varying moisture content depending on the climatic conditions. During rainy periods, the pores in this zone may be fully saturated. On the other hand, during dry periods, the pores may be completely dried up.

Finally, the intermediate zone or vadose zone is located between the soil water

Figure 2.1. Distribution of the subsurface water. (Bear, 1979)
zone and the capillary zone. The degree of saturation in this zone depends on the percolation from the upper layer and is usually relatively low.

The thickness of these zones varies greatly depending on the soil characteristics (infiltration capacity) and the water conditions in the area (level of the water table, amount of precipitation).

2.2.2 Classes of soil water
The basic division of soil water into three main classes, such as proposed at the end of the last century by Briggs (1897), is still generally accepted as the most logical classification (Childs, 1972). According to this division, the three classes are the hygroscopic, the capillary and the gravitational (or gravimetric) water.

The *hygroscopic water* is the water which always remains in the soil, even in dry conditions, usually as a thin film around the grains. The *capillary water* is the water which is kept in the pores by internal forces acting between the grains and the water; it is observed above the groundwater table. The *gravitational water* is the water which is not retained by capillarity in the soil and forms the permanent groundwater system below the water table.

2.3 CHARACTERISTICS OF GROUNDWATER IN PARTIALLY SATURATED SOILS

2.3.1 Capillarity
When two substances are in contact with each other, the difference between their inward attraction results in an interfacial tension. The interfacial tension \( \kappa_{ik} \) for a pair of substances \( i \) and \( k \) is specific and temperature-dependent. The contact angle \( \alpha \) between two substances and a solid plane surface is the angle measured through the denser substance between the solid surface and the interface, Figure 2.2. When \( \alpha < 90^\circ \), the fluid (L) is said to wet the solid, and is called a wetting fluid. A nonwetting fluid shows a contact angle \( \alpha > 90^\circ \). In an unsaturated soil (air and water filling the pores), water is the wetting phase and air is the nonwetting phase.

The difference in pressure between the two substances (here air and water) is called the capillary pressure \( p_c \):

\[
P_c = P_{\text{air}} - P_{\text{water}}
\]

(2.1)
where \( p_{\text{air}} \) and \( p_{\text{water}} \) are measured near the interface. Since the air pressure is atmospheric \( (p_{\text{air}} = 0) \), the capillary pressure is equal to the pressure in the water:

\[
p_c = -p_w.
\]

(2.2)

To visualize the capillary phenomenon in a soil, a glass tube can be used to represent the channel between the grains, Figure 2.3. The pressure and the pressure head in the water at the interface is obtained from the following equation:

\[
p_c = -\frac{4 \sigma_{a-w} \cos \alpha}{d}
\]

(2.3)

or

\[
h_c = \frac{4 \sigma_{a-w} \cos \alpha}{\rho_w g d} = \frac{2 \sigma_{a-w}}{\rho_w g r_m}
\]

(2.4)

where
- \( p_c \) = capillary pressure;
- \( h_c \) = capillary pressure head or capillary rise;
- \( \sigma_{a-w} \) = interfacial tension between air and water;
- \( d \) = diameter of the capillary tube;
- \( \alpha \) = contact angle between water and the tube;
- \( r_m \) = radius of the meniscus, \( r_m = d/2\cos \alpha \).

The contact angle \( \alpha \) depends on the chemical composition of the tube and on the impurities covering the walls as well as those found in the water (Terzaghi and Peck, 1948). The capillary rise \( h_c \) reaches its highest value when \( \alpha = 0 \) \((\cos \alpha = 1)\) and the radius of the meniscus is equal to the radius of the capillary tube, \( r_m = d/2 \).
The main problem in transferring this concept to a porous media such as soil is that the diameter of the capillary channel is not constant and very difficult to estimate. Many methods have been proposed for the estimation of the capillary pressure head by using some grain diameter taken from the grain size distribution of the soil (see e.g. Beskow, 1930; Terzaghi and Peck, 1948; Andersson, 1976; Holtz and Kovacs, 1981). All these methods try to find a direct relation between the "effective pore diameter" governing the capillary rise and some given grain diameter, e.g. the effective grain diameter, $d_{10}$. However, these approximations are seldom reliable since the grain size distribution, as well as the pore size distribution, may vary considerably between soils having the same $d_{10}$.

Owing to the capillary phenomenon, water can be drawn up from the saturated zone to partially or sometimes fully saturate the pores in the soil zone situated
immediately above the water table. Since the capillary forces are controlled by
the dimension of the pores, the capillary zone may reach different heights in
different soils. This capillary zone is thicker in fine-grained soils where the
pores are smaller than in coarse-grained materials. Table 2.1 presents some
guiding values of the height of the capillary fringe in different materials.

Table 2.1. Capillary fringe for different soil types.
(after Beskow, 1930 and Hansbo, 1960)

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>Capillary fringe, m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse sand</td>
<td>0.03 - 0.15</td>
</tr>
<tr>
<td>Medium sand</td>
<td>0.12 - 0.50</td>
</tr>
<tr>
<td>Fine sand</td>
<td>0.30 - 3.50</td>
</tr>
<tr>
<td>Silt</td>
<td>1.50 - 12.0</td>
</tr>
<tr>
<td>Clay</td>
<td>&gt; 10.0</td>
</tr>
</tbody>
</table>

**Hysteresis**

The simplification made by representing the soil pores using a tube of constant
diameter helps to understand the capillarity phenomenon and define the different
forces involved. However, the soil is in reality formed of a system of channels
having different cross-sections, and variations of the thickness of the capillary
zone are therefore observed in different locations.

Observations have shown that the capillary rise is usually different when the
soil is wetted by a rising water table compared to when it is drained due to a
sinking table (Staple, 1962; Poulouvasilis, 1962; Vachaud, 1970). According to
Bear (1972, 1979), this hysteresis phenomenon can be explained by a combination
of two factors: the “ink-bottle effect” which results from the fact that water
needs more energy to re-enter pores with a smaller diameter, and the “raindrop
effect” which is due to the fact that the contact angle is different depending on
whether the interface is advancing or receding, Figure 2.4. However, Kovacs
(1981) rejects the arguments presented for the “raindrop effect”, claiming that
this effect is present only during water movement and does not affect the water
distribution in static conditions.

The hysteresis phenomenon does not only affect the capillary rise but also in-
fluences the water distribution in the entire unsaturated zone (see section
2.3.3).
2.3.2 Soil suction

As mentioned before, one of the characteristics of the unsaturated and the tension-saturated zones is that the pore-water pressure in these zones is negative when related to the atmospheric pressure. This negative pressure is sometimes called soil suction.

The soil suction is related to relative humidity (RH) as expressed in thermodynamics, Fredlund, 1989. The relationship, presented in Figure 2.5, shows that a slight reduction in RH causes important variations of the suction, especially at higher degrees of saturation.
According to Figure 2.5, it is possible to compute a total suction approaching 10 000 atmospheres by measuring the relative humidity of a near air dried sample (Fredlund, 1991). The physical phenomenon which can explain these high values without water starting to boil is still not clearly understood, but the fact remains that pressures in these ranges may occur. One explanation could be that capillary forces create a very small radius of the air-water interface, resulting in decreased vapour pressure of the water (Fredlund, 1991). This could explain the high values measured in soils with low water content.

The total soil suction $h_{t}$ can be divided into two components (Fredlund, 1989): the matric suction component $h_{m}$ which is due to the reduction in the relative humidity created by a curved air-water interface, and the osmotic component $h_{o}$ which is due to a reduction in relative humidity caused by the presence of salts in the pore fluid.

The matric suction is by definition equal to the difference in pressure in the air and the water phases (see section 2.3.1 on capillarity), and is obtained from the following equation, similar to equation 2.1:

$$h_{m} = u_{air} - u_{water} = u_{a} - u_{w}$$

(2.6)

and can be reduced to

$$h_{m} = -u_{w}$$

(2.7)

if the air pressure is equal to the atmospheric pressure.

The total suction is given by

$$h_{t} = h_{m} + h_{o} = (u_{a} - u_{w}) + h_{o}$$

(2.8)

**Figure 2.6** shows results from an investigation performed by Krahn and Fredlund (1972) where the different components of the total suction have been measured. The results show that the osmotic suction is relatively constant while the matric suction varies resulting in variations in the total suction.

The osmotic component of the soil suction is normally neglected by the engineers dealing with unsaturated soils. Fredlund (1989, 1991) discusses the reasons for this practice and the predominant importance of the matric suction. Osmotic suctions appear even in saturated soils but are neglected in the appli-
Figure 2.6. Total, matric and osmotic suction measured in Regina clay.
(Krahn and Fredlund, 1972 and Fredlund and Rahardjo, 1993)

For 2.7. Total suction (psychrometer) - Matric suction (pressure plate) - Osmotic suction (squeezing technique)
- Osmotic plus matric suction

2.3.3 Soil moisture distribution

- Saturation phases
The distribution of the soil moisture in the unsaturated zone is influenced by many factors such as the weather conditions, the position of the water table, the surface vegetation and the pore size distribution in the soil. Bear (1979) distinguishes between three different stages of saturation, **Figure 2.7**:
- at very low degrees of saturation, the water forms pendular rings between soil particles, Figure 2.7a. These rings are isolated and the water phase is not continuous which means that no flow can occur;

- at higher saturation, the pendular rings expand to eventually form a continuous water phase, Figure 2.7b. At this stage, both the water and the air phase are continuous;

- as water saturation increases further, the air breaks into small individual bubbles in the water and the air phase is no longer continuous, Figure 2.7c. These bubbles can eventually be transported away by water flow and the soil may reach full saturation.

As the water content increases, different things happen which influence the porewater pressure. At higher water content, the radius of the curvature of the air-water interface (meniscus) decreases which results in a decrease in the interfacial tension, leading finally to a decrease in the matric suction. This means that the matric suction is inversely related to the water content in the soil, a phenomenon which is of primary interest for the study of the unsaturated zone.

Obviously, the relationship is similar in the other way, the water content being affected by variations in the matric suction. An increase in the matric suction in a soil layer will result in a decrease in the water content. However, since the pores have different dimensions, all the water will not be removed from the soil.
under the effect of the same suction. The water in the small pores will remain at higher suction than the water filling larger pores due to the fact that they can sustain higher radii of curvature of the meniscus and therefore reach higher capillary rise.

Another important observation is that there will always remain a certain amount of water in the soil even at very high suction level. This water will remain as a thin film around the particles or as pendular rings between two particles. The lowest value reached for the volumetric water content is called the *irreducible water content* $\theta_{wo}$ and results in an *irreducible saturation degree* $S_{wo}$.

### Soil moisture retention curve

The relationship between the matric suction and the water content $h_f = h_f(\theta)$ is called the *soil moisture retention curve* or the *characteristic water retention curve* of a soil. The facts presented above show that this relationship is a function of the pore size distribution in the soil. Since the pore size distribution is closely related to the grain size distribution (Åberg, 1992), the shape of the soil moisture retention curve can also be characterized by the grain size distribution of the soil (Andersson and Wiklert, 1972, van Genuchten, 1980, Jonasson, 1991) (see Figure 2.8).

The influence of the grain size distribution of the soil on the shape of the retention curve can be confirmed also by the influence of the gradation of the soil, a well graded soil showing a smoother relation than a poorly graded one, Figure 2.9.

As mentioned previously, there is, in fine-grained soils, a *tension-saturated zone* situated immediately above the groundwater table. According to Fetter (1980), the negative pressure in this zone is too low to desaturate the pores filled with water. In fact, all the pores in this zone are so small that they are fully saturated with water by capillarity up to the limit of the tension-saturated zone. The suction at which some pores cannot hold the porewater is called the *bubbling pressure* or the *air entry pressure* $h_a$ of the material. This pressure is related to the size of the pores; the smaller the pores, the higher the bubbling pressure, and the thicker the tension-saturated zone. This zone is very well defined in the characteristic curve as shown schematically in Figure 2.10.
Figure 2.8. Example of soil moisture retention curves for different soil types. (After Andersson and Wiklert, 1972)

Figure 2.9. Influence of the gradation on the soil moisture retention curve. (Bear, 1979)
Since the matric suction in the unsaturated zone may cover a large range of values, a logarithmic representation is sometimes used. In this representation, the matric suction is transformed in the following way

\[ pF = \log_{10} h_t \quad (h_t \text{ in cm of water}) \]  

(2.9)

The representation of the other axis remains unchanged, Figure 2.11.

---

Figure 2.10. Schematic characteristic curves including tension-saturated zone. a) uniform sand; b) silty sand; c) silty clay. (after Freeze and Cherry, 1979)

Figure 2.11. Logarithmic representation of a soil moisture retention curve. (also called pF-curve). (Lindström and McAffey, 1987, and Jonasson, 1991)
Hysteresis of the soil moisture retention curve

As explained previously, the hysteresis phenomenon does not only affect the capillary rise but also the water conditions in the whole unsaturated zone. Its influence on the water content will result in different soil moisture retention curves depending on the process (wetting or draining) occurring during the measurement.

For example, starting the drainage of a saturated sample will give a drainage curve as shown in Figure 2.12. The water content decreases gradually until it reaches the irreducible water saturation. Rewetting the sample will give a curve which differs from the drainage curve due to the phenomenon of hysteresis.

![Diagram of soil moisture retention curve](image)

**Figure 2.12.** Hysteresis of the soil moisture retention curve. (after Freeze and Cherry, 1979)

The occurrence of the hysteresis is mainly due to two different phenomena: the ink-bottle effect and the raindrop effect discussed earlier. There is, however, another factor increasing the difference between the rewetting (or imbibition) curve and the drainage (or drying) curve: the air entrapment. During the rewetting, some air will be entrapped in the pores, resulting in a final water content lower than that at the beginning. **Figure 2.13.** This entrapped air may sometimes be removed by flow of water. According to Bear (1979), an effect similar to the air entrapment can be caused in fine-grained soils by subsidence and shrinkage.
The hysteresis loop can be repeated as long as the soil sample is not subjected to any structural change due to, for example, consolidation.

Reversing the process of drainage and wetting leads to the determination of internal branches between the drainage and imbibition curves (Bear, 1979; Vachaud and Thony, 1971; Kovacs, 1981; Poulavassilis, 1962; Staple, 1962).

This means that the relationship between the water content and the matric suction depends on the history of wetting-drying of the sample. The water content corresponding to a given matric suction is always higher in a drying soil than in a soil under imbibition. Measuring only one parameter, it is virtually impossible to determine the other with accuracy unless the exact situation of the sample is known.
Soil moisture distribution vs water movement

The soil moisture retention curve is generally determined in static conditions and represents therefore the situation when no flow occurs in the soil.

In the upper zone of the soil, i.e. immediately below the ground surface, the water conditions are often affected by phenomena creating a hydraulic gradient and resulting in flow of water.

Evaporation from this zone will produce an upward flow, a diminution of the water content and an increase in the matric suction, Figure 2.14a. On the other hand, precipitation will create an increase in the water content and a decrease in the matric suction, Figure 2.14b. The excess of water will induce a downward flow to re-establish the equilibrium in the zone.

![Figure 2.14. Soil moisture configurations in different flow conditions.](image-url)
Distribution of soil moisture in layered soils

It was established in the previous sections that the relation between water content and matric suction depends on the soil type and that it takes different forms for different grain size distributions. Inversely, each soil type has a specific soil moisture retention curve. This means that for a given matric suction, the water content differs from one soil to another.

In homogeneous soil profiles, the water content at different elevations above the water table can be obtained directly from the soil moisture retention curve, provided that a no-flow situation is prevailing.

In a layered soil profile, the water content at a given elevation has to be taken from the curve which is characteristic of the soil encountered at this elevation. This may result in abrupt variations in the water content profile at the limit between two layers even though the pressure profile is uniform, Figure 2.15.

This phenomenon creates problems when studying the flow in a layered soil profile and will be discussed later.

Figure 2.15. Soil moisture conditions in layered soils.
Determination of the soil moisture retention curve

The soil moisture retention curve can be determined in laboratory or from field investigations or estimated by using correlations based on different characteristics of the soil such as the grain size distribution. To determine the retention curve, both the soil suction and the water content must be measured at different steps covering the range to be studied.

Laboratory measurement

Basically, the laboratory determination is performed by changing the matric suction gradually, measuring the water content after equilibrium at each step.

The simplest way to obtain the water retention curve is to use a capillary column which consists of a soil in contact with free water at its bottom. The water content is determined after equilibrium is reached at different levels in the capillary zone using gamma transmission. However, this method is disadvantaged by a long period of time required for stabilization (Jonasson, 1991).

The most commonly used apparatus is the porous plate which can be subjected to rather high negative pressures (up to 85 kPa) without losing its saturation. A soil sample is placed on the plate and the water pressure is decreased gradually using a hanging column or vacuum, Figure 2.16. The pressure is changed stepwise and the water content is allowed to reach equilibrium in the sample. Thereafter, the water content is determined by gravimetry or by recording the change of volume in the specimen.

Figure 2.16. Different types of pressure plate apparatus. (Jonasson, 1991)
Field measurement

The field determination is usually performed by adding water to the soil gradually, measuring both the water content and the soil suction at different time intervals.

The most common method is a combination of tensiometers for suction measurement and neutron probes for water content determination. Measurements performed simultaneously at the same level with these instruments give the corresponding relation between the water content and the matric suction.

The main disadvantage of this type of measurement is that the water content measured is a mean value over the hole length of the neutron probe, which means that the limit between two layers having very different water contents is impossible to determine with accuracy, Figure 2.17.

Figure 2.17. Field determination of the water content using a neutron probe. (after Jonasson, 1991)
Determination based on predictive models

Jonasson (1991) presents a comprehensive review of the different methods which can be used for the determination of the soil moisture retention curve. Mainly, the models are based either on the particle size distribution or simple analytical expressions used to describe the soil moisture retention curve.

The most commonly used method for transforming the grain size distribution curve to a soil moisture retention curve is the model proposed by Ayra and Paris (1981). The model uses a quasiphysical approach in which the soil is treated as a number of capillary tubes of different diameters, each tube corresponding to a soil particle size.

Other approaches based on grain size distribution use multiple regression to compute the soil water retention curve (see e.g. Hall et al., 1977; Gysta and Larsson, 1979; Rawls and Brackenski, 1982).

Analytical expressions have also been proposed for the prediction of the soil moisture curve. Two of these expressions are particularly well-known: the Brooks and Corey (1964) and the van Genuchten (1980) expressions.

The Brooks and Corey (BC) equation is an exponential curve which represents the soil moisture retention curve above the air entry pressure $h_a$:

\[
S_r = \left( \frac{h_a}{h_t} \right)^\lambda \quad \text{for } h_t \geq h_a
\]
\[
S_r = 1.0 \quad \text{for } h_t < h_a
\]
\[
S_r = \frac{(\theta - \theta_r)}{(\theta_s - \theta_r)}
\]

where

- $S_r$ = degree of saturation
- $h_a$ = air entry pressure
- $h_t$ = matric suction
- $\lambda$ = BC-coefficient
- $\theta$ = volumetric water content
- $\theta_r$ = retention water content
- $\theta_s$ = water content at full saturation

The van Genuchten equation is an S-shaped expression:
\[ S_r = \frac{1}{\left(1 + \left(\alpha_{vG}\right)^{n_{vG}}\right)^{m_{vG}}} \]

\[ S_r = \frac{(\theta - \theta_r)}{(\theta_s - \theta_r)} \]  

(2.11)

where

- \( S_r \) = degree of saturation
- \( \alpha_{vG} \) = van Genuchten coefficient
- \( n_{vG} \) = soil parameter
- \( m_{vG} \) = soil parameter
- \( \theta \) = volumetric water content
- \( \theta_r \) = retention water content
- \( \theta_s \) = water content at full saturation

Jonasson (1991) developed a new method for estimating the different parameters used in the above equation based on a two-point characterisation of the grain size distribution curve of the soil. The van Genuchten equation is an S-shaped expression which is continuously differentiable and therefore advantageous to use in numerical models (Jonasson, 1991).

2.4 FUNDAMENTALS OF GROUNDWATER FLOW IN FULLY AND PARTIALLY SATURATED SOILS

2.4.1 General approach

In order to be able to study the flow of fluid in porous media, it is necessary to go through different levels of simplification. The first level is reached by representing the medium as a continuum in order to pass from a microscopic to a macroscopic analysis. This is called the continuum approach. The porous medium is then considered as homogeneous, i.e. the pores and particles are distributed evenly in the volume studied, and the properties of the medium can be more readily determined.

The second level of simplification is reached in passing from a three-dimensional to a two-dimensional flow by assuming that the vertical component of the flow is negligible. This is called the hydraulic approach.
Continuum approach

The study of the porosity of a medium cannot be performed at a microscopic level due to the fact that the porosity shows large variations depending on where the study is performed (in a pore or in a particle). Moreover, the porosity is practically impossible to describe mathematically due to the complexity of the geometry of the pores and the solid surfaces. Therefore, the study of the porosity has to abandon the microscopic level and instead use a fictitious average of the porous volume.

Hubbert (1940, 1956) discussed this problem of macroscopic analysis, proposing to study the porosity by defining a volume that is representative of the medium. The volume chosen must include a sufficient number of pores to be a meaningful statistical representation of the medium. Figure 2.18 shows the solution presented by Hubbert for the determination of the representative volume for soil porosity. In an infinitesimal volume, the porosity may be one or zero depending on whether the volume studied is centered in a pore or on the solid matrix. When increasing the investigated volume, the total porosity will show decreasing oscillations around a mean value. After reaching a certain volume, the average porosity is constant. This volume is the limit between the domain of microscopic effects and the domain of porous medium. It is called the Representative Elementary Volume (REV) (Bear, 1972) and represents the minimum volume to be used for a correct evaluation of the average porosity of the medium. The REV is useful when passing from the microscopic level, where the study is made inside a single pore, to the macroscopic level of a

![Figure 2.18. Method of defining Representative Elementary Volume (REV) for macroscopic properties (illustrated with porosity). (Hubbert, 1956; Bear, 1979)](image-url)
continuum where only average properties are considered. The concept of the REV is useful in studying not only the porosity but even other properties of the medium related to porosity, such as permeability, hydraulic conductivity and compressibility.

The concept of a continuum using the REV is simple and easy to understand physically. However, it has two important shortcomings (Kovacs, 1981, Marsily, 1986). First, it is based on the assumption that the average value of the property studied shows a flattening as explained before. If this is not the case, the size of the REV may remain quite arbitrary. Second, the concept can be used only if the medium is homogeneous in a volume at least as large as the REV. If the volume is increased, reaching a zone with different characteristics, the mean value measured will be different and therefore not representative of the medium, Figure 2.19. This creates problems when representing boundaries in heterogeneous media.

![Graph showing effect of heterogeneity on the Representative Elementary Volume](image)

**Figure 2.19. Effect of heterogeneity on the Representative Elementary Volume (Kovacs, 1981)**

There are other approaches which can be used to pass from the microscopic to the macroscopic level without having to define a minimum reference volume. Some of these methods are based on mathematical analyses (weighting functions) or stochastic analyses (random functions) (Marsily, 1986). Other approaches, such as analysis of composite materials, can also be applied to porous media (Beran, 1968; Dagan, 1979).
Hydraulic approach
The second level of simplification is based on the assumption that the flow is essentially horizontal. This means that the groundwater flow $q$ has only two horizontal components ($q_x$ and $q_y$) and that the vertical component $q_z$ is neglected. This simplification, known as the hydraulic approach to flow in aquifers, can be made in many cases since the geometry of the aquifers is often such that they are relatively thin compared to their horizontal dimensions, Figure 2.20.

The assumptions on which this approach is based are often referred to as Dupuit's assumptions. They are true in a horizontal, homogeneous, isotropic, confined aquifer of constant thickness. According to Bear (1979), the approximation is still acceptable if the variations in the aquifer thickness are small com-

---

Figure 2.20. Examples of the hydraulic approach for flow in aquifers. (Bear, 1979)
pared to the average thickness, Figure 2.20a. The approximation fails only when the flow has an important vertical component as is the case near partially penetrating wells, Figure 2.20b. It is also false near outlets such as rivers or springs, Figure 2.20c. However, the approximation can be justified at a distance from the well of about 1.5 to 2 times the thickness of the aquifer (Bear, 1979). Even the flow in a leaky aquifer may be considered as horizontal if some specific conditions are fulfilled (Bear, 1972, 1979), Figure 2.20d.

The assumption of a horizontal flow can readily be accepted in normal situations where the flow is mainly directed along the different soil layers. However, it becomes more inappropriate in the cases mentioned above, where the flow is three-dimensional, and should preferably be treated as such. Nevertheless, the complexity of the exact solutions and the minor errors normally resulting from the simplified assumption contribute to the extended use of the hydraulic approach.

2.4.2 Hydraulic head and potential

According to elementary fluid mechanics, the potential at a certain point in a fluid is given by three different terms:
- the dynamic term (flow velocity),
- the static term (position),
- the pressure term (fluid pressure).

However, since the water flow velocity in soils is usually small, the first term may be neglected. Assuming that the fluid is incompressible, the potential at one point takes the following form:

\[ \Phi = gz + \frac{p}{\rho} \]  

(2.12)

where \( z \) is the elevation of the point related to a datum (\( z = 0 \)) and \( p \) is the pressure at this point.

The hydraulic head calculated for the same point is defined as the sum of the elevation head \( z \) and the pressure head \( h \), Figure 2.21:
\[ H = z + h = z + \frac{p}{\rho g} \quad (2.13) \]

(a) laboratory manometer  
(b) field piezometer

**Figure 2.21. Representation of the elevation head \( z \), the pressure head \( h \) and the total hydraulic head \( H \). (Freeze and Cherry, 1979)**

A comparison of the two sets of equations shows that the relation between the potential and the hydraulic head is constant:

\[ \Phi = gH. \quad (2.14) \]

**Figure 2.22** shows the physical representation of the parameters mentioned above both in the saturated and the unsaturated zones.

The main difference affects the pressure head, which changes sign when passing from one zone to the other. Usually, the pressure head is represented as a negative value in the unsaturated zone. When the contribution of the pressure head in the hydraulic head is large, the latter may also become negative.

Since the fluid pressure is a function of the water content of the soil, the pressure head and the hydraulic head are also dependent on this parameter. Equation 2.13 becomes:

\[ H = h(\theta) + z \quad (2.15) \]

where \( h(\theta) \) is the pressure head corresponding to the volumetric moisture content \( \theta \) on the moisture retention curve (see e.g. Figure 2.12).
Figure 2.22. Groundwater conditions near the ground surface. (after Freeze and Cherry, 1979)

(a) saturated and unsaturated zones
(b) profile of moisture content vs depth
(c) pressure head and hydraulic head
(d) profile of pressure head vs depth
(e) profile of hydraulic head vs depth

2.4.3 Darcy’s law

In 1856, Henry Darcy published a report on the resistance to flow of sand filters (Darcy, 1856). According to many authors (Freeze and Cherry, 1979; Wang and Anderson, 1982; Freeze, 1994), this report marked the birth of groundwater hydrology as a quantitative science.

Darcy defined the specific discharge (or the seepage velocity) through the filters using the following relation:

\[
\frac{Q}{A} = v = -K \frac{H_2 - H_1}{\Delta l} = -K \frac{\Delta H}{\Delta l}
\]  

(2.16)

where \(K\) is a constant of proportionality called the hydraulic conductivity or coefficient of permeability and \(v\) is the Darcy velocity or the specific discharge. The other symbols are defined in Figure 2.23. The hydraulic conductivity has dimensions [L/T] and is usually given in m/s or cm/s. The relation described in Equation 2.16 is commonly known as Darcy’s law. It describes the flow through porous media at a macroscopic level.
The hydraulic conductivity is defined in Darcy's law as the coefficient of proportionality between the specific discharge \( v = Q / A \) and the hydraulic gradient \( i = \Delta H / \Delta l \). As mentioned before, the approach which leads to Darcy's law is macroscopic, i.e. the study of the flow is valid for an average of the soil. Therefore, the hydraulic conductivity can be regarded as a macroscopic parameter. A microscopic approach shows that the hydraulic conductivity is in reality a function of the matrix and the fluid properties (Hubbert, 1940). Experiments performed with ideal porous media consisting of uniform glass beads of diameter \( d \), and fluids of different density and dynamic viscosity \( \mu \) helped to separate the effect of these different properties on the specific discharge (Freeze and Cherry, 1979). The following relationships of proportionality were observed:

\[
v \propto d^2; \quad v \propto \rho_w g \quad \text{and} \quad v \propto \frac{1}{\mu} \quad (2.17)
\]

Adding the relationships already observed by Darcy,

\[
v \propto \Delta H \quad \text{and} \quad v \propto \frac{1}{\Delta l} \quad (2.18)
\]

we obtain a new expression of equation 2.16:

\[
v = \frac{C_p d^2 \rho_w g \Delta H}{\mu \Delta l} \quad (2.19)
\]

where \( C_p \) is a new coefficient of proportionality taking into account some characteristics of the soil such as the grain size distribution and grain shape.
Since the term $C_p d^2$ is a function of the medium alone, the following parameter can be defined:

$$k = C_p d^2$$  \hspace{1cm} (2.20)

where $k$ is called the specific or intrinsic permeability. The intrinsic permeability has dimensions [L2] and is often given in $m^2$ or $cm^2$. Sometimes, the unit Darcy is used which is approximately equal to $10^{-8} cm^2$.

Combining the above expressions, the hydraulic conductivity may be re-defined as:

$$K = \frac{k p_w g}{\mu} = \frac{k \gamma_w}{\mu} = \frac{k \gamma}{\nu}$$  \hspace{1cm} (2.21)

where $\gamma_w$ is the volumetric weight of the water and $\nu$ is the kinematic viscosity of the fluid. This equation clearly shows that the hydraulic conductivity is directly influenced by the fluid properties (density and viscosity). Since these properties vary with temperature (see Figure 2.24), the hydraulic conductivity will also depend on the temperature of the fluid. The variations in viscosity are especially important and should be taken into account when the flow occurs in the upper part of a soil deposit where significant variations in temperature may take place. It is also important to take this into account when using values of hydraulic conductivity measured in conditions other than those prevailing at the location studied. For instance, the values estimated from laboratory tests made at 20 °C must be corrected before using them to predict the soil behaviour in field conditions, which for Swedish soils are at about 7 °C:

$$K_{\text{in situ}} = \frac{K_{\text{lab}}}{1,4}$$  \hspace{1cm} (2.22)

Table 2.2 presents some typical values of permeability and hydraulic conductivity for different soil types with water at 20 °C. For these conditions, the following relation between the two parameters is valid:

$$1 \text{ m/s} \iff 1,02 \cdot 10^{-3} \text{ cm}^2.$$
Figure 2.24. Water viscosity as a function of temperature. (Marsily, 1986)

Table 2.2. Typical values of permeability and hydraulic conductivity. (Freeze and Cherry, 1979)
Darcy's law is one of the most important concepts when studying groundwater flow in porous media. It can be generalized beyond the conditions under which it was initially found to a three dimensional field:

$$v = -K \text{ grad } H$$  \hspace{1cm} (2.23)

where $K$ is the three dimensional tensor of the hydraulic conductivity.

The conclusions drawn by Darcy were based on a continuum approach considering the medium as homogeneous without taking into account the microscopic aspects, i.e. neglecting the fact that the soil consists of pores and particles. The velocity calculated with the above equation is called the Darcy velocity (or specific discharge), and the concept is appropriate when dealing with flow through porous media. However, the Darcy velocity is not a representation of the flow velocity in the pores and should therefore not be used in studying sediments or contaminant transport. For this purpose, one has to calculate the effective velocity, which takes into account the effective porosity of the medium (Bear, 1972; Kovacs, 1981).

### Hydraulic conductivity in unsaturated soils

The resistance to the flux of an unsaturated porous matrix is greater than the resistance of the same matrix in saturated conditions. This can be explained by two phenomena. First, a decrease in the water content results in a reduction of the cross-sectional area available for water flow. Second, drainage occurs first in the larger pores so that the flow in unsaturated conditions is mainly confined to the small pores, resulting in an increase in the tortuosity of the flow paths (Bear, 1979). This means that the hydraulic conductivity of a soil is decreasing as the water content decreases.

The value of the hydraulic conductivity in unsaturated soils can be measured directly or expressed as a function of other soil properties. In the latter alternative, it is often related to the value measured in the saturated state. The general expression for these relationships can take two different forms

$$K_{\text{unsat}} = K_{\text{sat}} \cdot f(\text{soil conditions}) \hspace{1cm} (2.24)$$

or
\[ K_{rel} = \left[ \frac{K_{\text{unsat}}}{K_{\text{sat}}} \right] = f(\text{soil conditions}) \]  
(2.25)

where \( K_{rel} \) is called the relative hydraulic conductivity. (Since the hydraulic conductivity is directly proportional to the specific permeability, many authors prefer to use \( k \) instead of \( K \) in the equations presented above.)

The function \( f \) appearing in these equations is usually determined experimentally and can therefore take different forms depending on the parameters studied in the investigation. Richards (1931) extended Darcy’s law to the unsaturated zone and suggested a relationship between the hydraulic conductivity and the capillary pressure. Wyckoff and Botsett (1936) studied the relationship between the hydraulic properties of a soil and the degree of saturation. Their investigations performed on the value of permeability at different levels of water viscosity show the relationship between the relative permeability and the saturation.

Since the suction (or capillary pressure) is related to the degree of saturation (or water content), it is obvious that both relationships \( K=K(h) \) and \( K=K(\theta) \) are valid. Irmay (1954) made a theoretical analysis of these relationships based on a development similar to that of Richards. He came to the conclusion that the relationship between the permeability (or hydraulic conductivity) and the degree of saturation is more useful than any other.

### Hydraulic conductivity and hysteresis

In the preceding sections, the suction was shown to be a function of the water content measured in the unsaturated zone, \( h_i = h_i(\theta) \). This relationship is however not unique, being affected by hysteresis. Therefore, the proportionality between the hydraulic conductivity and these two parameters should be studied with regard to this phenomenon.

Since the hydraulic conductivity is influenced by the amount of water in the cross-section, it is obvious that the relationship between these two parameters should be unique, or at least show a low degree of hysteresis. However, the hysteresis shown in the relationship \( h_t = h_i(\theta) \) should be reflected in the value of \( K(h) \). Investigations presented in the literature confirm these statements, showing that the relationship between the hydraulic conductivity and the soil moisture is affected by a certain amount of hysteresis (Figure 2.25a), which
however is negligible compared to the hysteresis shown in the relationship $K(h)$, Figure 2.25b. This confirms also the conclusions made by Irmay (1954) that the former relationship is preferable to the latter, because it is less affected by hysteresis.

![Graph](image)

**Figure 2.25.** Hysteresis of the different relationships established for the hydraulic conductivity.

- a) related to degree of saturation (after Bear, 1979)
- b) related to pressure head (after Freeze and Cherry, 1980)

**Determination of hydraulic conductivity in saturated soils**

*Determination based on predictive models*

Many attempts have been made to correlate the specific permeability or the hydraulic conductivity values with different soil characteristics. All investigations have confirmed the dependence on the grain diameter as described in Equation 2.19.
Using the relation proposed by Kozeny (1953) for the effect of porosity, Ko­vacs (1981) presents the following equation for the calculation of the hydraulic conductivity:

\[ K = \frac{1}{5} \frac{\rho g}{\mu} \frac{n^3}{(1-n)^2} \left( \frac{d_{10}}{\alpha_K} \right)^2 \]  

(2.26)

which takes into account the influence of the parameters discussed above:

- behaviour of fluid (\( \rho g/\mu \)),
- characteristics of grains (\( d_{10}/\alpha_K \)),
- effect of porosity (\( n^3/(1-n)^2 \)).

Kovacs (1981) also presents a thorough discussion of a large number of investigations found in the literature focusing on the influence of different parameters on the hydraulic conductivity. Table 2.3 summarizes the relationships obtained from these investigations. As can be seen from this table, all the investigations proved the above statement regarding the relation between \( K \) and the grain size (\( d^2 \)). Some relations take into account other parameters discussed above, such as temperature (Hazen), porosity (Zauberei), and viscosity (Lindquist, Veronese, Romer, Bear).

### Determination of hydraulic conductivity in unsaturated soils

Many different methods exist to determine the hydraulic conductivity of partially saturated soils. A detailed presentation of the different methods used for laboratory or field measurement can be found in Klute (1972) and Olson and Daniel (1981). Although often similar regarding their accuracy, these methods are based on different principles and performed under different boundary conditions.

#### Laboratory measurement

There are three main groups of methods for measurement in the laboratory: (1) steady-state methods, (2) instantaneous profile methods and (3) pressure-plate outflow tests.

In the **steady-state methods**, the measurements are performed on samples installed in cells with porous elements at both ends, Figure 2.26a. The test is performed by applying a constant pressure difference between the two ends in
### Table 2.3. Some relationships for the calculation of hydraulic conductivity. (Kovacs, 1981)

<table>
<thead>
<tr>
<th>Author</th>
<th>Form of the formula</th>
<th>Temperature used or assumed in the formula or sworn to by the interpretation [°C]</th>
<th>The given or measured validity range of the coefficient of conductivity</th>
<th>The multiplying factor valid in the case of the use of effective diameter</th>
<th>The recalculated shape coefficient, assuming $n = 0.38$</th>
<th>The investigated range of diameter [cm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kröber</td>
<td>$K_D = 41 D_1^2$</td>
<td>10</td>
<td>$\sim 1.2$</td>
<td>43</td>
<td>7.16</td>
<td>0.054-0.210</td>
</tr>
<tr>
<td>Žechtirn</td>
<td>$K_D = 37 D_1^2$</td>
<td>10</td>
<td>$\sim 1.8$</td>
<td>40</td>
<td>7.30</td>
<td>0.016-0.008</td>
</tr>
<tr>
<td>Hugon</td>
<td>$K_D = 36 D_1^2$</td>
<td>10</td>
<td>$\sim 1.5$</td>
<td>39</td>
<td>7.63</td>
<td>0.028</td>
</tr>
<tr>
<td>Hazer</td>
<td>$K_D = 115 D_1^{1.2}$</td>
<td>10</td>
<td>1.5-2.5</td>
<td>46</td>
<td>6.92</td>
<td></td>
</tr>
<tr>
<td>Jaky</td>
<td>$K_D = 100 D_1^2$</td>
<td>10</td>
<td>$\sim 2.0$</td>
<td>39</td>
<td>7.63</td>
<td></td>
</tr>
<tr>
<td>Karádi</td>
<td>$K_D = 160 D_1^2$ (lead balls)</td>
<td>10</td>
<td>0.6-1.0</td>
<td>100</td>
<td>6.10</td>
<td>0.25-1.59</td>
</tr>
<tr>
<td>Teranghi</td>
<td>$K_D = 200 D_1^2$</td>
<td>20</td>
<td>$\sim 0.4$</td>
<td>10</td>
<td>7.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Lindquist</td>
<td>$K_D = 2 - 2500 \frac{P}{D}$ (lead balls)</td>
<td>10</td>
<td>0.4</td>
<td>61</td>
<td>5.00</td>
<td></td>
</tr>
<tr>
<td>Chaudesbellis</td>
<td>$K_D = 265 D_1^2$ (lead balls)</td>
<td>10</td>
<td>0.5</td>
<td>63</td>
<td>7.55</td>
<td>0.015-0.005</td>
</tr>
<tr>
<td>Karádi</td>
<td>$K_D = 140 D_1^2$</td>
<td>10</td>
<td>0.5</td>
<td>63</td>
<td>7.15</td>
<td>0.025-0.050</td>
</tr>
<tr>
<td>Teranghi</td>
<td>$K_D = 160 D_1^2$</td>
<td>10</td>
<td>0.6-1.0</td>
<td>10</td>
<td>6.92</td>
<td>0.002-0.193</td>
</tr>
<tr>
<td>Teranghi</td>
<td>$K_D = 200 D_1^2$</td>
<td>20</td>
<td>$\sim 0.5$</td>
<td>10</td>
<td>7.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Zaaßbecl</td>
<td>$K_D = 230 - 350 m^2$</td>
<td>18</td>
<td>0.6-4.0</td>
<td>47</td>
<td>7.37-7.62</td>
<td>0.40-10.8</td>
</tr>
<tr>
<td>Veronezèse</td>
<td>$K_D = 230 - 350 m^2$</td>
<td>18</td>
<td>0.6-4.0</td>
<td>47</td>
<td>7.37-7.62</td>
<td>0.40-10.8</td>
</tr>
<tr>
<td>Romer</td>
<td>$K_D = 6.54 \times 10^{-1} \frac{P}{D}$</td>
<td>10</td>
<td>$\sim 1.0$</td>
<td>49</td>
<td>0.24</td>
<td>0.01-0.05</td>
</tr>
<tr>
<td>Bear</td>
<td>$K_D = 0.10 \times 10^{-1} \frac{P}{D}$</td>
<td>10</td>
<td>$\sim 1.0$</td>
<td>49</td>
<td>0.89</td>
<td>0.006-0.11</td>
</tr>
</tbody>
</table>

Note: $n = 0.43$

Controlling the pressure at these two locations. The flow of water injected into the sample and the variations in suction are measured when the steady-state is reached. At this moment the water content is measured. To obtain the relationship between the hydraulic conductivity and the water content, the test must be repeated at different pressures. One of the main disadvantages of this type of test is that the equation used for the interpretation is a relationship between the hydraulic conductivity and the pressure (or suction). Since this relationship is affected by hysteresis, the value of $K$ measured is not unique but is dependent on the process in progress during the test (wetting or drainage).

In the instantaneous profile methods, the soil sample is placed in a cell with inflow and/or outflow facility at one end only, Figure 2.26b. The test is performed by creating an alteration of the hydraulic conditions at one end. The alteration may be the application of a constant or time-dependent pressure change, or a constant or time-dependent inflow. The same principle is applied.
To Constant Head Water Supply

a) steady-state method

To Air Supply

b) instantaneous profile method

Soil Specimen

Outflow

Tensiometers

Soil

"Far End"

"Near End"

Inflow or Outflow

Suction Probes

"Near End" “Far End”

To Air Supply

Interconnected Grooves for Flushing

Air

Soil

Flush Line

To Outflow Measurement System

c) pressure plate outflow test

Figure 2.26. Apparatus for measurement of hydraulic conductivity in the laboratory. (Olson and Daniel, 1981)
for the interpretation of the instantaneous profile test as for the steady-state
test, leading to the same non-uniqueness of the value of $K$ due to hysteresis in
the relation $h_i(\theta)$.

The pressure-plate outflow test uses a laboratory instrument similar to the one
used for the measurement of the suction in a soil sample with a null type pres-
sure plate (see section 2.5.2). The instrument is however modified in order to
permit measurement of the outflow from the sample, Figure 2.26c. The test is
performed by applying different air pressures in the cell, measuring the volume
of water flowing from the sample. Once again the interpretation is non-unique
due to the hysteresis of the relation used.

**Field measurement**

Field measurements are mostly performed according to two different princi-
pies: the instantaneous profile method and the infiltration test through an im-
peding layer.

The instantaneous profile method used in the laboratory can also be applied to
field conditions. The test is performed by building a small dike around an area
where the measurement is to be made. The area is then soaked with a control-
led amount of water and covered to avoid evaporation. Measurements of the
suction are performed at different levels during the infiltration of the water
through the soil. The interpretation is performed the same way as for the labo-
rary test. These tests are sometimes performed without flooding a given area
but by waiting instead for a heavy rain. Another variant is to let evaporation
occur during the test, taking this factor into account during the interpretation.
These variations sometimes make it difficult to interpret the measurement with
the accuracy required.

Measuring infiltration through an impeding layer is another type of test which
can be performed in field conditions. The area to study is first limited by push-
ing down an impervious tube of given dimensions and with open ends (suitable
diameter and length are 26 and 30 cm, Baker, 1977). The soil is thereafter cov-
ered with a permeable layer working as a filter. A constant water pressure is
then applied by pouring water onto this impeding layer. According to the de-
velopers of the method, the gradient is equal to one and the hydraulic conduc-
tivity is equal to the flow when steady-state is reached (Olson and Daniel,
**Determination based on predictive models**

To avoid the problems related to direct measurement of the hydraulic conductivity of unsaturated soils, many expressions have been developed based on different soil properties. These expressions are mostly based on three different principles: (1) constitutive (or predictive) models based on pore size distribution, (2) constitutive (or predictive) models based on particle size distribution, and (3) inverse analysis of initial-boundary value problem.

The most common approach based on the pore size distribution was proposed by Childs and Collins-George (1950). The model is based on the assumption that the pore-size distribution is easily characterized by measurement of the relationship between moisture content and suction, \( h(\theta) \). Modifications of the Childs and Collins-George model has been suggested by many authors. Among the most important contributions are those of Millington and Quirk (1961) who improved the method by taking into consideration the pore interaction, and Jackson et al. (1965) who suggested the use of a matching factor to "calibrate" the equation at a given degree of saturation (usually at full saturation).

The first step in this method is to divide the curve moisture content-suction into \( m_2 \) equal water content increments. Thereafter, the suction \( h \) at the midpoint of each increment is determined and the hydraulic conductivity \( K(\theta) \) is calculated according to the following equation (Green and Corey, 1971):

\[
K(\theta)_i = \frac{K_s}{K_{sc}} \frac{30 \gamma^2}{\rho_w g \mu} \frac{m_{PC}}{m_i^2} \sum_{j=i}^{m_2} [(2j + 1 - 2i) h_j^{-2}]
\]  

\( i = 1, 2, ..., m_2 \)

where:

- \( K(\theta)_i \) = calculated conductivity for a specified moisture content corresponding to the \( i \)th increment, cm/min
- \( \theta \) = volumetric moisture content, cm\(^3\)/cm\(^3\)  
  \( (\theta_o = \text{lowest moisture content on the experimental } h(\theta) \text{ curve,} \) and \( \theta_s = \text{saturated moisture content}) \)
- \( i \) = last water-content increment on the wet end, e.g. \( i=1 \) identifies the pore class corresponding to \( \theta_o \), and \( i=m_2 \) identifies the pore class corresponding to \( \theta_s \)
\( K_s/K_{sc} = \) matching factor (measured saturated conductivity/calculated saturated conductivity)  
\( \gamma = \) surface tension of water, N/cm  
\( \rho_w = \) water-density, g/cm\(^3\)  
\( g = \) gravitational constant, cm/s\(^2\)  
\( \mu = \) kinematic viscosity of water, cm\(^2\)/s  
\( n = \) water-saturated porosity, cm\(^3\)/cm\(^3\)  
\( p_{GC} = \) parameter that accounts for interaction of pore classes  
\( m_1 = \) total number of pore classes between \( \theta = 0 \) and \( \theta_s \)  
\( m_2 = \) total number of pore classes between \( \theta_o \) and \( \theta_s \)  
\( h_j = \) suction for a given class of water-filled pore, cm H\(_2\)O

Green and Corey (1971) pointed out the distinction between \( m_1 \) and \( m_2 \), which is often neglected in the literature (Elzeftawy and Cartwright, 1981). There are \( m_1 \) moisture content classes between zero water content and full saturation (between 0 and \( \theta_s \)), but the calculation is performed with only \( m_2 \) water classes covering the experimental moisture-content curve (between \( \theta_o \) and \( \theta_s \)). The relationship between \( m_1 \) and \( m_2 \) is as follows:

\[
m_1 = m_2 \left[ \frac{\theta_s}{\theta_s - \theta_o} \right] \tag{2.28}
\]

In summary, \( m_1 \) is greater than \( m_2 \) (\( m_1 > m_2 \)) except when \( \theta_o = 0 \) in which case they are equals (\( m_1 = m_2 \)).

The importance of the choice of the parameter \( p_{GC} \) to be used in the equation has been studied by different authors. Values of 2.0 (Marshall, 1958), 1.33 (Millington and Quirk, 1959), and 1.0 (Kunze et al., 1968) have been proposed. Green and Corey (1971) studied the effect of the choice of \( p_{GC} \) on the results, concluding that calculated values show little discrepancy when 1.0 < \( p_{GC} \) < 2.0.

Since the textural characteristics of a soil are more easily obtained than the hydraulic data, another estimation approach based on the particle size distribution has been proposed (Mishra et al., 1989; Mishra and Parker, 1989). This approach proposes to use the Ayra and Paris (1981) model to transform the particle size distribution data into a soil water retention curve and to use the expressions proposed by Brooks and Corey (1964) and van Genuchten (1980) to analytically represent this curve by parameter-fitting. The hydraulic conduc-
tivity can thereafter be calculated using the different equations proposed in these models.

The inverse analysis of initial-boundary value problem is presented by Kool et al. (1985, 1987) and Mishra and Parker (1989). In this approach, the parameters used in the Brooks and Corey and van Genuchten models are determined by using a dynamic experiment (i.e. infiltration) in a study of the inverse initial-boundary problem.

2.4.4 Storage or retention capacity of an aquifer

The storage capacity of an aquifer is a characteristic which is essential in the study of the flow. It may be described in different ways depending on the type of aquifer studied and on the type of flow occurring.

The storativity $S$ of an aquifer with constant thickness is the volume of water released per unit of surface area of the aquifer due to a unit decline of the piezometric head. In a confined aquifer, the storativity may be defined as follows:

$$S = \frac{\Delta V_w}{A \Delta h} \quad (2.29)$$

where $\Delta V_w$ is the change in water volume in the aquifer, $A$ is the surface area and $\Delta h$ is the change in piezometric head, see Figure 2.27a.

For an unconfined aquifer, the storativity may be defined using a similar equation:

$$S = \frac{\Delta V}{A \Delta h'} \quad (2.30)$$

where $\Delta h'$ represents the decline in the phreatic surface, Figure 2.27b.

The storativity of an unconfined aquifer is sometimes called the specific yield $S_y$. It is always smaller than the porosity, $n$, of the soil due to the fact that some water is retained between the particles by capillary forces. The amount of water retained is called the specific retention $S_{ro}$ and depends on the grain size distribution. The specific yield $S_y$ can therefore be defined as follows:

$$S_y = n - S_{ro} \quad (2.31)$$
Figure 2.27 - Definition sketch for storativity of aquifers. (Bear, 1979)

As pointed out by Bear (1979), it is very important to realize that, despite the similarity of Equations 2.29 and 2.30, the storativity of the two types of aquifers is controlled by different parameters. In confined aquifers, the storativity is due to the water and matrix compressibility, while in unconfined aquifers it is due to drainage of the water from the pores between the two positions of the water table.
The concept of storativity has been introduced using the assumption of a constant thickness of the aquifer and a two-dimensional flow. If the groundwater movement is characterized by a three-dimensional flow, the specific storage $S_s$ should be used instead. The specific storage $S_s$ is defined as the volume of water released by a unit volume of an aquifer due to a unit decline in the piezometric head (confined aquifer) or in the phreatic surface (unconfined aquifer):

$$S_s = \frac{\Delta V_w}{V \Delta h}$$

where $V$ is the bulk volume of the medium. The specific storage is controlled by changes in the volume of the fluid and in the porous medium (grains and skeleton). Assuming that the compressibility of the fluid is significantly smaller than the compressibility of the porous medium and can therefore be neglected, equation 2.32 can be rewritten as follows (see e.g. Bear (1979) and Marsily (1986) for complete development of the equation):

$$S_s = \rho g \beta_s = \frac{d n}{d h}$$

where $\beta_s$ is the coefficient of compressibility of the soil skeleton, neglecting the compressibility of the grains:
\[
\beta_s = \frac{d \text{ porosity}}{d \text{ pressure}} = \frac{d n}{d \theta}
\]  

(2.34)

The specific storage \( S_s \) is a representation of the storage capacity for fully saturated soils. In the unsaturated zone, the amount of water released or accumulated during variation of the water conditions is controlled by the relation between the soil moisture and the matric suction. The storage capacity of an unsaturated soil can therefore be estimated from its characteristic curve, \((h_t)\). The specific moisture capacity \( M \) is defined as

\[
M(h_t) = \frac{d \theta}{dh_t}.
\]  

(2.35)

The value of \( M(h_t) \) is the slope of the characteristic curve. Since \( \theta(h_t) \) is non-linear and affected by hysteresis, the value of \( M(h_t) \) is not constant but hysteretic.

Using the expressions in Equations 2.33 and 2.35, it is possible to find a general expression \( C(h) \) for the storage capacity of a soil in all conditions (saturated and unsaturated). In the partially saturated zone, the value of \( C(h) \) is equal to \(-M(h_t)\). Below the level where \( h_t = h_s \) and in the saturated zone, the soil moisture is equal to the porosity \( n \), i.e. \( C(h) = (dn/dh) = (d\theta/dh) \). This means that the general expression for the storage capacity is as follows:

\[
C(h) = \frac{d \theta}{dh}
\]  

(2.36)

### 2.5 MEASUREMENT OF GROUNDWATER CONDITIONS

#### 2.5.1 Measurement of pore pressure in fully saturated soils

Groundwater conditions and their variations with time can be recorded in cohesionless materials by using open standpipe piezometers or perforated pipes. In cohesive soils, the variations in pore pressure distribution can be recorded with closed piezometers. Detailed presentations of the methods used for measuring pore pressures in saturated soils can be found in Dunnicliff (1988) and Tremblay (1989).
2.5.2 Measurement of soil suction in partially saturated soils

There are many different instruments which have been developed for the measurement of the total suction and its two components: osmotic and matric suction. Many of the instruments perform indirect measurement of the suction, i.e. the parameter measured is not the suction itself but a parameter related to it. The suction is thereafter obtained from a calibration curve.

Many authors have studied one or several instruments. Fredlund and Rahardjo (1988) and Fredlund (1989) give a comprehensive and detailed presentation of the advantages and disadvantages of different methods, their field of application and limitations.

- **Null type pressure plate**

The null type pressure plate is a laboratory method used for measuring the matric suction in soil specimens. It is based on the axis-translation technique developed in the late 1940’s (Hilf, 1956).

To perform a measurement, the soil specimen is put in contact with a water compartment through a porous disc, Figure 2.29. Since the water is at first at atmospheric pressure and the specimen has an intrinsic suction, the water starts moving from the water compartment to the soil, whereby the pressure in the water decreases. A pressure is then applied around the soil specimen until an equilibrium is reached and the pressure is stabilized both in the pressure cell and the water compartment. When this equilibrium is reached, the difference between these two pressures is equal to the initial matric suction in the soil specimen.

![Figure 2.29. Schematic of a null type pressure plate. (Fredlund and Rahardjo, 1993)](image-url)
The null type pressure plate can be used over a wide range of suction values (up to 500 kPa) (Fredlund, 1989). However, this range depends on the air-entry value of the porous disc used, a disc with higher air entry value (or bubbling pressure) permitting measurement of higher suction.

**Tensiometer**

The tensiometer is an instrument developed for field measurement of the negative pore pressure (Gardner and Richards, 1936). In this instrument, the pressure is measured directly in a water column, which is in contact with the pore-water through a porous element. If the pore-air pressure is equal to the atmospheric pressure, the value measured with tensiometers is equal to the matric suction.

**Figure 2.30** presents schematically the tensiometer with different reading systems. The tensiometer is filled with water before installation of the porous cup into the soil where the suction is to be measured. The pore-water in the soil comes in contact with the water in the system through the porous cup. Water flows from the cup to the soil where the pressure is negative until an equilibrium is reached. The negative pressure measured in the system at this moment corresponds to the pore-water pressure. Mercury manometers, vacuum gauges and electrical transducers are commonly used to measure the pressure in the system.

The range of suction which can be measured with tensiometers is relatively low and limited by the cavitation phenomenon in the system. This gives an upper limit of about 90 kPa. Problems with diffusion of air into the porous cup are one limitation of the tensiometers (Fredlund, 1989). However, this problem may be diminished by using porous cups with a higher air-entry value (Jonasson, 1989) or reference solutions with low osmotic potential (Peck and Rabidge, 1969; Stannard, 1990).

**Thermal conductivity sensor**

The thermal conductivity sensors measure the suction indirectly, using the relation between the thermal conductivity, the water content and the suction. This method is used to measure the matric suction both in the field and in the laboratory.

**Figure 2.31** presents the cross-section of a thermal conductivity sensor. A porous block containing such a sensor is installed in the soil where measurement is to be made. Water movement is immediately initiated due to the difference
Figure 2.30. Schematic lay-out of a tensiometer with different reading systems. (Stannard, 1990)

Figure 2.31. Cross-sectional diagram of a thermal conductivity sensor. (Phene et al. 1971; Fredlund and Rahardjo, 1993)
in pressure. This movement occurs from the soil to the porous block if the latter is installed dry, or in the opposite direction if the installation is made with saturated blocks. When equilibrium is reached between the block and the soil, a controlled amount of heat is generated in the sensor and the temperature is read out at the same point. The increase in temperature depends on the water content in the block which corresponds to the pore water pressure in the soil, since pressure equilibrium prevails.

The value of the matric suction can be evaluated from a calibration curve obtained before installation and in which the relation between the temperature increase and the matric suction in the block is represented. Calibration curves are usually provided by the manufacturer, but large deviations from these curves have been observed during calibration controls performed in a modified pressure plate apparatus (Sattler and Fredlund, 1989).

Thermal conductivity sensors can be used in a suction range up to 400 kPa (Fredlund, 1989). They can therefore be used for many applications both in situ and in the laboratory.

**Electrical conductivity sensor**

The electrical conductivity sensor is used to measure the suction based on a similar relation to the one used with the thermal conductivity sensors, in which the thermal conductivity is replaced by the electrical conductivity.

This type of sensor is made of a porous block, usually a gypsum block, in which two coaxial electrodes are incorporated, Figure 2.32. The porous block is installed in the soil in a dry or saturated state. After equilibrium of the pressure between the block and the surrounding soil, a current is sent to one of the electrodes and the resistivity between the two electrodes is measured.

The value of the matric suction can be evaluated from a calibration curve, usually provided by the manufacturer in which the relation between the electrical resistivity and the matric suction in the block is represented. According to the calibration chart, the electrical conductivity sensor can be used in a suction range up to 2000 kPa (Soil Moisture, 1989) and can therefore be used in many application fields both in situ and in the laboratory.

**Thermocouple psychrometer**

The thermocouple psychrometer indirectly gives a value of the total suction by measuring the relative humidity. As mentioned above, the total suction has a
unique relation to the relative humidity as previously described in Figure 2.5. This principle is used in the measurement made with thermocouple psychrometers.

To perform a measurement, the psychrometer is placed in the same environment as the soil until equilibrium is reached between the air near the psychrometer and the pore-air in the soil. The relative humidity can then be registered on the psychrometer. **Figure 2.33** shows schematically the design of a laboratory equipment.
The useful range of the thermocouple psychrometer is limited upwards by the minimum relative humidity which can be measured with this type of instrument, that is about 95% corresponding to a suction of 80 atm (8000 kPa) (Daniel et al., 1981). The lower limit is established by the precision which can be obtained at high relative humidity, at which small variations in temperature will result in important changes in water vapour and suction values (see Figure 2.5).

A commonly accepted lower limit is 100 kPa below which the precision of the thermocouple psychrometer is subjected to substantial uncertainty (Krahn and Fredlund, 1972; Daniel et al., 1981). The limiting effect of the influence of temperature variations makes the thermocouple psychrometer a reliable instrument for laboratory tests where the environment can be controlled easily. However, it should not be used for field measurements where temperature variations can be considerable.
Chapter 3.
Groundwater Modelling

3.1 INTRODUCTION
To model groundwater conditions, it is necessary to describe the phenomena involved and the different characteristics of the area for which modelling is performed.

The first step in modelling is to establish the basic equations governing the groundwater movement in the soil for the specific case to be studied. Starting from the same basic equations, it is often possible to simplify the analysis for different cases mainly depending on the variations in the model characteristics in time and/or space. Thereafter, the area for which modelling is to be performed must be delimited, and the soil stratification and the properties of each layer must be defined. All soil layers and other elements (lakes, rivers, wells, etc.) susceptible to influence the groundwater conditions must be incorporated in the model. It is, however, important to avoid using unnecessary elements such as impermeable layers, which would increase the time necessary for the simulation without improving the results.

When the geometry of the area and the properties of the different layers have been specified, it is necessary to define the groundwater conditions at the boundaries of the model for the whole period to be simulated. Water conditions can also be incorporated in the model in other specific locations, if they are well defined and play an important role for the simulation, i.e. the water level in lakes or rivers and pumping from wells. When variations in groundwater conditions are to be modelled, the initial conditions in the whole area have to be defined and applied to the model; the simulation of the variation is thereafter initiated by changing the value at some given location(s).

This chapter deals with the description of the different steps leading to a reliable model and includes the presentation of different types of models which can be used. The computer model used in this study is presented at the end of the chapter.
3.2 CONTINUITY AND DIFFUSION EQUATIONS

In order to establish the diffusion equations for the flow of a fluid in porous media, it is necessary to introduce the mass conservation equation. Considering a unit volume of a porous medium, the law of mass conservation within this volume takes the following form:

\[ \text{div}(\rho \mathbf{v}) + \frac{\partial}{\partial t}(\rho n) + \rho q = 0 \]  
\[(3.1)\]

where \( \rho \) = density of the water
\( \mathbf{v} \) = specific discharge (vector)
\( n \) = porosity of the medium
\( q \) = volumetric flow rate

The different terms of Equation 3.1 represent (1) the fluid mass flow through the volume, (2) the change in fluid mass in the volume, and (3) the addition of fluid by external sources.

The mass conservation equation can also be written as:

\[ -\text{div}(\rho \mathbf{v}) = \frac{\partial}{\partial t}(\rho n) + \rho q \]  
\[(3.2)\]

The left-hand term represents the net mass flow into the element. If the fluid is incompressible (\( \rho = \text{constant} \)), the mass conservation equation becomes

\[ -\text{div} \mathbf{v} = \frac{\partial n}{\partial t} + q \]  
\[(3.3)\]

Introducing Darcy’s law for flow in aquifers (Equation 2.16) and the definition of the storage capacity of the soil \( C \) (Equations 2.33 and 2.36) in the mass conservation equation, we obtain

\[ \text{div} (K \grad H) = C \frac{\partial H}{\partial t} + q \]  
\[(3.4)\]
which is the general expression of the diffusion equation for the transient flow in soils.

The term \( q = q(x,y,z,t) \) represents the external contribution to the mass balance into the aquifer. This term may represent (1) distributed sources or sinks, (2) point sources or sinks located at the upper boundary of the aquifer, or (3) point sources or sinks located inside the flow domain. The formulation of the function \( q \) varies depending on what types of source or sink are considered. This term will be omitted in the following development of the general equations, i.e. an assumption is made that there are no such sinks or sources.

Equation 3.4 is linear if \( K, C \) and \( q \) are independent of \( H \). It is non-linear if any of these three terms depends on the value of \( H \) during the solution process. The general partial differential equation for the forecasting of the flow behaviour may take different forms depending on different parameters: (1) the hydraulic properties of the aquifer in which the flow occurs, (2) the type of aquifer, and (3) the state at which the flow is to be forecasted. Many times, a good judgement in the choice of parameters will lead to a simplification of the equation.

Making the assumption that the vertical axis is one of the principal directions of the hydraulic conductivity tensor and the x- and y-axis are parallel to the two principal directions of anisotropy, the general equation for flow in saturated soils takes the following form:

\[
\frac{\partial}{\partial x} \left( K_x \frac{\partial H}{\partial x} \right) + \frac{\partial}{\partial y} \left( K_y \frac{\partial H}{\partial y} \right) + \frac{\partial}{\partial z} \left( K_z \frac{\partial H}{\partial z} \right) = C \frac{\partial H}{\partial t} \tag{3.5}
\]

The flow properties of an aquifer are controlled by the value that the hydraulic conductivity can take in different directions and locations. The four main different combinations of \( K \) are:

a) heterogeneous anisotropic aquifer
\[ \Rightarrow K_x = K_x(x,y,z); \quad K_y = K_y(x,y,z); \quad K_z = K_z(x,y,z) \]

b) heterogeneous isotropic aquifer
\[ \Rightarrow K_x = K_y = K_z = K(x,y) \]
c) homogeneous anisotropic aquifer
   \[ K_x, K_y, \text{ and } K_z \text{ are constant but different from each other} \]
   \[ (K_x \neq K_y, K_x \neq K_z, K_y \neq K_z) \]

d) homogeneous isotropic aquifer
   \[ K_x = K_y = K_z = K = \text{constant} \]

Two main types of aquifer may be encountered in fully saturated soils: confined and phreatic aquifers. In the case of confined aquifers, the flow of water directly depends on changes in the pressure head, and the equation to be solved is linear. For phreatic aquifer, the storage capacity of the soil also depends on the position of the water table, which results in a non-linear equation.

In unsaturated soils, both the storage capacity and the hydraulic conductivity are dependent on the pressure head \( h_r \). Using the definition of the hydraulic head in unsaturated soil, \( H = h_r + z \), the general equation for flow in unsaturated soil can be rewritten in the following form:

\[
\frac{\partial}{\partial x} \left( K_x(h_r) \frac{\partial h_r}{\partial x} \right) + \frac{\partial}{\partial y} \left( K_y(h_r) \frac{\partial h_r}{\partial y} \right) + \frac{\partial}{\partial z} \left( K_z(h_r) \left( \frac{\partial h_r}{\partial z} + 1 \right) \right) = C(h_r) \frac{\partial h_r}{\partial t} \tag{3.6}
\]

This equation is often called Richards' Equation, in honour of the soil scientist who first developed it (Richards, 1931). The equation is non-linear due to the definition of \( C \) and \( K \) in partially saturated soils.

The last parameter controlling the choice of the governing equation is the time factor. The flow can be studied in two different states: the transient state in which the hydraulic head varies with time \( (\partial h_r/\partial t \neq 0) \), and the steady state where no variations occur in time \( (\partial h_r/\partial t = 0) \).

### 3.3 ESTIMATION OF BOUNDARY AND INITIAL CONDITIONS

The differential equations presented in section 3.2 describe different classes of flows according to the physical properties of the soil and the fluid. Each equation has an infinite number of solutions and the appropriate solution is obtained after providing some supplementary information relevant to the specific prob-
lem studied and depending on the conditions prevailing.

Bear (1979) gives the following list of supplementary information necessary to describe a specific problem:

a) the geometry of the domain where the phenomenon is taking place
b) the parameters describing the porous medium and the fluid involved (k, n, S,...)
c) the boundary conditions which define the interaction between the domain studied and its surroundings
d) the initial conditions which define the initial state of the fluid in the domain.

The boundary and initial conditions specify the form of dependent variables at a given position and/or time in the differential equation. Solving the problem means solving the governing partial differential equation in the flow domain while, at the same time, satisfying the specified boundary and initial conditions.

A detailed discussion regarding the choice of the boundary and initial conditions is presented in Bear (1979) and in Franke et al. (1987).

3.3.1 Boundary conditions

The choice of boundary conditions to be satisfied on the domain is a critical stage in the modelling of the groundwater flow. The use of improper boundary conditions will be reflected on the results obtained from the model as they will not correspond to any physical response in the real system.

Selecting the boundary conditions usually involves a simplification of the actual domain. The assumptions made for this selection should always be understood and their effect controlled. The location of the lateral limits given to the model is critical and should always be selected on the basis of a sensitivity analysis (Franke et al., 1987), Figure 3.1a (line AF, EG) and Figure 3.1b (line AH, GI). The description of the boundary conditions inside the domain studied may also sometimes necessitate such an analysis.

In this section, the five most common boundary conditions encountered will be described and discussed.

□ Prescribed-head boundary (Dirichlet)

The first type of boundary conditions is the prescribed-head boundary which occurs when the head along a boundary surface can be described using a spe-
cific function. The most general case is encountered when the head is a function of both the position on the boundary and the time

\[ H = f(x,y,z,t). \]

The most common case of this type of boundary occurs when the head is constant both in time and space \((H = \text{constant})\), such as the head at the bottom of a free water body, Figure 3.1a (lines AB, DE) and Figure 3.1b (lines ABC, EFG).

The specified-head boundary should be used with precaution since it provides an inexhaustible source of water (Franke et al., 1987). Before using this type of boundary, one should consider if the real system can provide the necessary amount of water to keep the specified head. The evaluation of the results should always include a close look at this characteristic.

■ Prescribed-flux boundary (Neumann)

A second type of boundary conditions can be described when the flux through a surface is specified as a function of the position of the points on the boundary and the time.

The most general case occurs when the flux varies both in space and time

\[ q = f(x,y,z,t). \]

The most elementary case of this type of boundary is the impermeable surface where the flux is zero

\[ q = 0 \]

which is also called the no-flow boundary, Figure 3.1a (lines BCD and FG) and Figure 3.1b (line HI).

The prescribed flux may also take a constant value when the flow is constant with time and uniform in space

\[ q = \text{constant} \]

such as in the area where an artificial recharge is performed, or in the vicinity of a pumping well.
Figure 3.1. Different examples of boundary conditions. (after Franke et al., 1987)
**Head-dependent flux (semipervious) boundary (Cauchy)**

Sometimes, the flux crossing the boundary surface changes as variations of the head in the surroundings occur. In such cases, the boundary condition is described as a function of the flux depending on the head difference between the two sides of the boundary.

Figure 3.1c shows an example of such a situation where a semipervious layer B separates a confined aquifer C from a free water body A. The flow through the line XY can be described as follows

\[
q = -K \frac{H_A - H_C}{b} \quad (H_C > z_{XY})
\]  

(3.7)

where
- \( q \) = flux through the semipervious boundary,
- \( K \) = hydraulic conductivity of the leaky aquifer,
- \( b \) = thickness of the leaky aquifer B,
- \( H_A \) = hydraulic head in the aquifer A,
- \( H_C \) = hydraulic head in the aquifer C,
- \( z_{XY} \) = position of the lower limit of the semipervious layer B.

Assuming that the water level in the free water body A remains unchanged, the variations in the flux passing through the semipervious boundary XY is strictly dependent on variations in the head in the lower aquifer C. As seen from the above equation, the flow will be directed upwards if the head in aquifer C is higher than the water level in the free water body. As the head decreases, the flow will decrease and eventually be directed downwards when \( H_C \) becomes smaller than \( H_A \).

Usually, this type of boundary also involves a practical limit beyond which the changes in head in the aquifer have no influence on the flux. In this example, the flow remains unchanged when the hydraulic head in the aquifer C becomes lower than \( z_{XY} \):

\[
q = -K \frac{H_A - z_{XY}}{b}
\]  

(3.8)

Any further drawdown in the aquifer C will have no effect on the flow which is then controlled only by the water level in A and the position of the bottom of the aquifer \( z_{XY} \).
**Free-surface (phreatic) boundary**

The most common free-surface boundary is the water table which is defined as the groundwater surface where the pressure is atmospheric \((p = 0)\), Figure 3.1b (line CD). The head can be defined on the surface as equal to the elevation head since the pressure head is equal to zero \((h = p/\gamma = 0)\):

\[
H = z.
\]

When searching the position of the free-surface boundary in a steady-state problem, it is necessary to proceed by iterations since the solution of the problem is influenced by the position assumed and vice versa.

Since the position of the water table is influenced by the head and flux acting at the boundaries, any variation in these parameters will lead to a change of this position. This relationship is non-linear and plays an important role in the solution of groundwater systems with phreatic conditions.

**Seepage-face boundary**

The seepage-surface boundary is used to define a surface along which groundwater discharges, Figure 3.1b (line DE). This surface is therefore always at atmospheric pressure and can be described using the same equation as for the free-surface boundary

\[
H = z.
\]

The seepage-face boundary has a known geometry since it coincides with the limit of the domain studied. Its length is, however, dependent on other boundaries which is why the seepage-face boundary is always associated with the free-surface boundary. The location of the junction point between the two boundaries is usually determined during the solution of the problem.

### 3.3.2 Initial conditions

Before studying the transient state of a groundwater system, it is necessary to define the initial conditions in the system at a reference time. These conditions are usually defined by specifying the head distribution at a time \(t = 0\):

\[
H = f(x,y,z,t=0).
\]
The choice of initial conditions is very important for the reliability of the first steps of the simulation, since the changes induced immediately after the insertion of new boundary conditions in a model are representative only if the initial conditions are correctly defined.

There are different ways of obtaining the initial conditions. The most common method is to use measurements performed at different locations in the area studied. However, measurements cannot always be used directly in the model, since they may have been made at a time when the system was non-steady and therefore represent a groundwater situation which is not at equilibrium. Moreover, the measurements represent the real groundwater situation at a given time and, since the model is only a simplified representation of this area, it is not certain that the characteristics of the model and the measurements performed are consistent. Using measurements which are not consistent with the model would result in false variations in the first step of the simulation due to adjustments of the measurements and the model characteristics. According to Franke et al. (1987), the initial conditions used in a transient-state problem should be determined through a simulation of the flow at steady-state. The initial conditions should be defined by modifying the characteristics of the model until a steady-state analysis gives results showing a satisfying fit with the measurements performed when the groundwater conditions were at equilibrium. The results of the simulation can thereafter be used as initial conditions.

3.4 METHODS FOR SOLVING THE GROUNDWATER EQUATION

In the preceding sections, the formulation of different groundwater flow problems was presented. To obtain the values of hydraulic heads $H=H(x,y,z,t)$, it is necessary to solve the partial differential equation subject to specified boundary and initial conditions.

There are different methods which can be used for solving these problems. These methods can be grouped into three classes:
- analytical methods,
- analogous methods (models),
- numerical methods.

The choice of method to be used for solving a specific problem should be made with regard to the complexity of the problem, the economic and time constraints, the availability of manpower and finally, the use of the results and the required accuracy.
3.4.1 Analytical methods

The best way to solve the equations describing the problem to be studied is, if possible, to find an analytical expression of the solution. The analytical methods are superior to other methods if the exact solution is obtained without further simplification of the real problem other than the necessary approximations made during the formulation of the governing equation and the boundary and initial conditions. Analytical solutions permit the study of the influence of different parameters on the results.

Analytical solutions can be found in textbooks treating the heat conduction equation (Carslaw and Jaeger, 1959). Bear (1972) presents some solutions applied to flow in aquifers.

Analytical methods are suitable for solving simple, one-dimensional groundwater flow problems. They are sometimes suitable for a two-dimensional flow in the vertical plane (hydraulic approach) where different approximations can be done in the formulation of the problem. They are often used for well hydraulics. However, they may be less suitable when a lag of time is observed in the release of storage water.

Analytical solutions are not applicable in the following cases (Bear, 1979):
- irregularity of the shape of the aquifer’s boundary,
- type of boundary conditions changes along the boundary,
- spatial distribution of aquifer characteristics (K and S) or the initial conditions cannot be described using analytical expressions,
- phreatic aquifers described using non-linear partial differential equations.

Analytical methods are seldom applied in practice for the solution of relatively complex problems since the formulation requires too much simplification and approximation. Nevertheless, analytical solutions provide information on the dependence of different properties for idealized systems and permit a comparison and a control of analogous and numerical methods.

3.4.2 Analogous methods

Analogs and models are usually constructed to solve a particular problem. However, they can also be used for studying other problems with different inputs and outputs if the geometry and the parameters describing the aquifer are the same.

Analogous methods can be used if the dynamic and kinematic behaviour of the
real system can be reproduced in a prototype system using similar fundamental equations. Analogy requires a one-to-one correspondence between the elements of the two systems, as well as elements with similar excitation-response relationship.

When the prototype system has the same physical characteristics as the real system, it is called a model, e.g. sand box model. When the prototype system is constructed in such a way that its governing equations describe the same basic principles of conservation and movement that are observed in the real system, it is called an analog, e.g. electric analog.

Table 3.1 presents some models and analogs together with their main features and fields of application.

Calibration of analogs and models is necessary to fill the lack of information especially regarding some physical parameters. The first step of the construction is the reproduction of every relevant aspect of the aquifer including the interaction between various parts of the system. The prototype can then be used for forecasting the behaviour of the system under certain conditions (e.g. pumping, recharge, drainage, ...).

Table 3.1. Applicability of models and analogs. (Bear, 1979)

<table>
<thead>
<tr>
<th>Feature</th>
<th>Sand box model</th>
<th>Hele-Shaw analog</th>
<th>Electric analogs</th>
<th>Membrane analog</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Vertical</td>
<td>Horizontal</td>
<td>Electrolytic</td>
<td>RC-network</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>two or three</td>
<td>two or three</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>steady</td>
<td>steady</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>both</td>
<td>both</td>
</tr>
<tr>
<td>Dimensions of field</td>
<td>two or three</td>
<td>two</td>
<td>two</td>
<td>two</td>
</tr>
<tr>
<td>Steady or unsteady flow</td>
<td>both</td>
<td>both</td>
<td>both</td>
<td>both</td>
</tr>
<tr>
<td>Simulation of elastic storage</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>Simulation of capillary fringe and capillary pressure</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>Simulation of phreatic surface</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>Simulation of anisotropic media</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>Simulation of medium inhomogeneity</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>Simulation of leaky formation</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>Simulation of accretion</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>Flow of two liquids with an abrupt interface</td>
<td>approximately</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>Hydodynamic dispersion</td>
<td>yes</td>
<td>no</td>
<td>no</td>
<td>no</td>
</tr>
<tr>
<td>Simultaneous flow of two immiscible fluids</td>
<td>yes</td>
<td>no</td>
<td>no</td>
<td>no</td>
</tr>
<tr>
<td>Observation of streamlines and pathlines</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
</tr>
</tbody>
</table>

Subject to restrictions because of the presence of a capillary fringe. ^3^ By trial and error for steady flow. ^4^ By scale distortion in all cases except for the RC-network and sometimes the Hele-Shaw analog where the hydraulic conductivity of the analog can be made anisotropic. ^5^ With certain constraints. ^6^ For a stationary interface by trial and error. After Bear, 1972.
Analogs and models are obviously not compatible with computer analysis. The relatively high cost of construction and calibration of the prototype, combined with its limited use, makes these methods unattractive for practical applications.

### 3.4.3 Numerical methods

Numerical methods used in computers are the major tools available for solving forecasting problems of groundwater flow. Several methods have been developed for the numerical solution of partial differential equations and applied to the equations governing the flow in porous media. These methods allow the scientist to replace complex partial differential equation problems with more tractable problems (Gray, 1984).

Numerical methods allow the treatment of a large number of problems which would otherwise be impossible to consider. Problems involving major complexities and described above as unsolvable with analytical methods can become straightforward exercises with numerical methods.

**Finite Difference Method (FDM)**

Finite Difference Methods are based on replacing partial differential equations by algebraic difference equations. This is done by replacing the continuous variable \( h(x, y, z, t) \) in the equation with a discrete variable. By solving the set of finite difference equations, a number of discrete approximations are obtained at investigator-selected points forming a grid.

The approximation at a given point depends on the grid points located in the immediate neighbourhood. The Finite Difference Method therefore gives a number of local approximations.

The domain of interest is discretized by selecting a number of points (or nodes) where the approximations are to be developed. The location of the points can theoretically be chosen anywhere in the domain. However, in order to simplify and accelerate the calculations, the grid is often chosen to form a network of squares with constant spacing. If a variable grid spacing is required, Trescott et al. (1976) suggested that one should use the criterion \( (x_i/x_{i-1}) < 1.5 \) to avoid problems with convergence of the solution and large truncation errors.

Shoup (1978) and Gray (1984) concluded that the Finite Difference Method is advantageous when the problem to be studied can be represented as a simple and regular grid. If the grid has to be irregular, the Finite Element Method is preferable.
The difference approximation of a function and its derivatives can be performed using different methods, of which the Taylor series expansion method is the most commonly used. By applying this method to the area around the point where the approximation is to be made, a difference equation is obtained. This equation consists of a term containing the values of the function known at given points and a truncation term giving an estimate of the error made by the approximation (see Bear, 1979).

Depending on the method chosen for the calculation, three different approximation patterns can be used: forward, backward and central difference approximations. Figure 3.2 shows the use of these three methods for the determination of the first derivative of a function.

![Figure 3.2](image)

**Figure 3.2. Representation of the different approximation methods to be used for finite difference analysis. (Bear, 1979)**

Three different schemes are commonly used in numerical simulations on computer: the explicit, the implicit and the Crank-Nicolson schemes using the forward, backward and central difference simulations respectively. To be stable, the explicit forward scheme requires a restrictive relationship between the grid dimensions and the time step (Rosenberg, 1969). The implicit backward and the Crank-Nicolson central schemes are unconditionally stable (Bear, 1979).

Other finite difference schemes have been developed for different applications (Peaceman and Rachford, 1955; Trescott et al., 1976; Bear, 1979; Gray, 1984; Narashimhan, 1984). These schemes are however less commonly used than those mentioned above.
Finite Element Method (FEM)

Finite Element Methods are based on replacing the partial differential equation by an integral equation (Zienkiewicz, 1977; Desai, 1979). The solution is expressed as an infinite sum involving polynomial functions. Each function has a coefficient and the solution is obtained by solving the set of equations for these coefficients.

The requirement on the solution is imposed on the average error over the entire domain. The Finite Element Method therefore treats a problem from a global point of view.

As for the Finite Difference Method, the domain studied is discretized into a number of sub-areas delimited by nodes. The shape and size of these sub-areas may vary depending on the shape of the domain or the degree of interest for a specific area. For example, the area near a highly solicited surface, e.g. the corner of a rectangular block, may be discretized into smaller sub-domains than the surrounding area. The difference in shape and size does not affect the calculation process in the Finite Element Method, which makes it a more flexible and powerful method than the Finite Difference Method (Gray, 1984). However, the use of elements of different form and shape is not recommended when the medium is not homogeneous and isotropic, since the choice of the mesh may, in such cases, influence the final solution obtained with the computer simulation.

The numerical integration performed in the Finite Element Method can be performed using different methods. The Weighted Residual Methods are certainly the most commonly used in practice. Among these methods, the Galerkin Method is usually preferred (Bear, 1979). The Galerkin Method uses a weighting function which is made equal to the shape function of the different elements of the domain. The integration is thereafter performed independently for each element and summed up to obtain the total contribution.

Among the advantages of the Finite Element Method given in the literature, the most important are as follows:
- FEM can handle any boundary shape and conditions,
- the size of the elements can vary (small elements can be used when rapid changes are expected and detailed results are required),
- FEM can handle deformable media and moving boundaries, even though with certain difficulties,
- FEM can handle flow in anisotropic domains (and in some special cases, in inhomogeneous systems).
Many examples of application of the Finite Element Method to groundwater flow problems can be found in the literature (Lewis and Robert, 1984; Cividini and Gioda, 1983; Aalto, 1984, 1985; Humbert, 1984). Table 3.2 presents the conclusions from a comparison of the Finite Difference and the Finite Element Methods made by Gray (1984).

**Boundary Element Method (BEM)**

The Boundary Element Method is an integration method using nodes located only on the boundaries of the domain to be studied (Ligget, 1977; Brebbia, 1978; Banerjee and Butterfield, 1981). This method is similar to the Finite Element Method using line elements only, Figure 3.3.

![Figure 3.3. Comparison of FEM and BEM meshes. (a) Finite Element mesh (b) Boundary Element mesh. (Chang, 1988)](image)

In the Boundary Element Method, the calculations are performed on the boundaries. This diminishes the number of equations to be solved simultaneously compared to the “domain” methods, such as the Finite Difference Method and the Finite Element Method. The Boundary Element Method is therefore advantageous to use for problems involving free surfaces where the position of the surface is sought and the groundwater situation in the whole region is of less interest. Therefore, most of the applications found in the literature are related to problems involving unconfined aquifers (Chang, 1981, 1988; Baker, 1981).

The Boundary Element Method requires less input preparation and computer capacity than the other numerical methods and could therefore be used in practice for solving simple seepage problems at lower cost.
## Table 3.2. List of elementary differences between the Finite Difference and the Finite Element Method. (Gray, 1984)

<table>
<thead>
<tr>
<th><strong>FINITE DIFFERENCE</strong></th>
<th><strong>FINITE ELEMENT</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>Approximates derivatives of functions directly.</td>
<td>Approximates the functions themselves.</td>
</tr>
<tr>
<td>Provides approximations at points which borrow information from neighbouring points to whatever degree desired.</td>
<td>Provides global approximations which are restricted to neighbourhoods of the grid to whatever degree desired.</td>
</tr>
<tr>
<td>Traditionally uses a regular mesh.</td>
<td>Structure of the method readily lends itself to irregular grid.</td>
</tr>
<tr>
<td>Approximations developed via curve fitting or Taylor series expansions.</td>
<td>Approximations developed from families of basis functions and integration over the grid.</td>
</tr>
<tr>
<td>Resultant difference expression explicitly provided by the programmer to the code. Approximation may be &quot;seen&quot;.</td>
<td>Resultant difference expression provided implicitly to the code which performs the integrations. Approximation is buried in the integrals.</td>
</tr>
<tr>
<td>Complete freedom to select nodes in a discrete approximation provided.</td>
<td>Nodes included in discrete approximation determined by basis functions, integration rule, and weighting function.</td>
</tr>
<tr>
<td>On a regular grid, similar approximations are made at different nodes.</td>
<td>Even on a regular grid, different approximations may be made at different nodes.</td>
</tr>
<tr>
<td>Difference approximations are generally in terms of function values only.</td>
<td>Difference expressions are often written in terms of function values and derivatives of the functions.</td>
</tr>
<tr>
<td>Though not typically done, derivatives with respect to one independent variable may be weighted over another variable.</td>
<td>Though typically weighted over different coordinates, derivatives may sometimes be lumped by selection of an appropriate nodal integration formula.</td>
</tr>
<tr>
<td>Commonly used ADI requiring a tridiagonal matrix solver.</td>
<td>Commonly uses banded, frontal, or sparse matrix solver.</td>
</tr>
<tr>
<td>Staggered grid easily implemented because of regular triangular nodal placement.</td>
<td>Staggered grid virtually impossible because of grid irregularities. However, mixed interpolation is a partially staggered grid.</td>
</tr>
<tr>
<td>Second or third type boundary conditions applied by altering the basic computational molecule.</td>
<td>Second or third type of boundary conditions are applied through the boundary integral obtained after application of Green's theorem.</td>
</tr>
<tr>
<td>Moving boundary problems are traditionally difficult.</td>
<td>Moving boundary problems are conceptually straightforward.</td>
</tr>
<tr>
<td>Large gradients require fine mesh for resolution.</td>
<td>Functional implanting may minimize need for fine mesh near steep gradients.</td>
</tr>
<tr>
<td>For a regular mesh, a zone requiring high resolution determines grid spacing.</td>
<td>Localized refinement of computational grid is easily accomplished.</td>
</tr>
<tr>
<td>Data input may be simple because of regularity.</td>
<td>Data input is complex and undiscovered data errors can be a cause of trouble.</td>
</tr>
<tr>
<td>Regular grid interfaces easily with computer graphics.</td>
<td>Graphics interface may be complicated by an irregular mesh and isoparametric elements.</td>
</tr>
</tbody>
</table>
3.5 COMPUTER PROGRAM SEEP/W

General features
SEEP/W is a computer software using the finite element method to solve problems related to porewater distribution and groundwater flow in porous media. The software may be used to solve simple or complex seepage problems. It operates in the Microsoft Windows environment.

The software presents the following technical features (GEOSLOPE, 1992):
- saturated and unsaturated flow analysis
- steady-state and transient conditions
- confined and unconfined problems
- multiple soil types
- anisotropic hydraulic conductivity coefficients
- transient boundary conditions

Flow in unsaturated soil is assumed to follow Darcy’s law as in saturated soil, which means that the flow at a given time is dependent upon the hydraulic head and the hydraulic conductivity at this time. The hydraulic conductivity function must be completely defined for the analysis. Since the hydraulic conductivity in unsaturated soils varies with the hydraulic head, the equations are non-linear and solved by an iterative process.

For transient analysis, the relation between the pressure and the water content (storage function) must be defined.

Basic equation
The governing equation used in SEEP/W is:

\[
\frac{\partial}{\partial x} \left( K_x \frac{\partial H}{\partial x} \right) + \frac{\partial}{\partial y} \left( K_y \frac{\partial H}{\partial y} \right) + Q = \frac{\partial \theta}{\partial t}
\]  

(3.9)

where:  
- \( H \) = total head  
- \( K_x \) = hydraulic conductivity in the x-direction  
- \( K_y \) = hydraulic conductivity in the y-direction  
- \( Q \) = applied boundary flux  
- \( \theta \) = volumetric water content  
- \( t \) = time
The term which represents the change in the volumetric water content can be related to the pore-water pressure through the storage curve, see Figure 3.4. The slope of the storage curve ($m_w$) represents the rate of change in volumetric water due to a change in pore pressure:

$$\frac{\partial \theta}{\partial u_x} = m_w \quad \text{or} \quad \partial \theta = m_w \partial u_x \quad (3.10)$$

Since changes in pore-water pressure are caused by changes in hydraulic head, equation 3.10 can be rearranged as:

$$\frac{\partial \theta}{\partial H} = m_w \gamma_w \quad \text{or} \quad \partial \theta = m_w \gamma_w \partial H \quad (3.11)$$

This gives the following governing equation used in SEEP/W:

$$\frac{\partial}{\partial x} \left( K_x \frac{\partial H}{\partial x} \right) + \frac{\partial}{\partial y} \left( K_y \frac{\partial H}{\partial y} \right) + Q = m_w \gamma_w \frac{\partial H}{\partial t} \quad (3.12)$$

In steady-state conditions, the right-hand term of the above equation is equal to zero since the flow entering the elements is the same as the flow leaving them which results in the following simplified equation:

$$\frac{\partial}{\partial x} \left( K_x \frac{\partial H}{\partial x} \right) + \frac{\partial}{\partial y} \left( K_y \frac{\partial H}{\partial y} \right) + Q = 0 \quad (3.13)$$
Solving principle
SEEP/W uses isoparametric triangular and quadrilateral elements. Triangular elements may have up to three secondary nodes (total of six nodes), while quadrilateral elements may have up to four secondary nodes (total of eight nodes). The element characteristics matrices are formulated using Gauss numerical integration. When no secondary nodes are defined in the triangular elements, the number of integration points or integration order is usually set to one, which implies that the gradient is constant throughout the element. For quadrilateral elements without secondary nodes, the integration function is linear and the integration order used is four. For both types of element in which secondary nodes are defined, the integration functions are non-linear and a higher integration order is required (see GEOSLOPE, 1992).

The solving process in SEEP/W is based on two assumptions:

- the total stresses are constant (no loading or unloading of the soil skeleton)
- the pore-air pressure remains at the atmospheric pressure in the transient state.

Consequently, changes in volumetric water content are strictly dependent on changes in pore-water pressure variations.

Main area of utilization
SEEP/W can be used to analyse the water flow and compute the pore pressure distribution in porous media such as soil and rock. Various applications in geotechnical engineering can be found, but the main area of interest is the study of expected variations in groundwater conditions around excavations and road cuts. The effect of drainage in different situations such as stabilization of natural slopes is another area where the model can be used.

SEEP/W is developed for studying any hydraulic problem with darcian flow in one or two dimensions. It can also be used for computing the potential surface changes due to water flow in soils in confined aquifers where the pore pressure is positive and the hydraulic conductivity shows small changes due to pressure change.
Special topics

Definition of hydraulic conductivity and storage functions
The material functions representing the hydraulic conductivity and the storage capacity of the soil can be fully described in SEEP/W. The storage capacity function is usually described by the user based on results from laboratory tests or some prediction method. The hydraulic conductivity function may also be described manually if it is determined in laboratory or field tests or estimated in some other way. If the function is unknown, it may be estimated in SEEP/W using a calculation routine based on the equation proposed by Green and Corey (1971), (see Chapter 2).

To obtain smooth and continuous curves, SEEP/W uses spline integration on the data points describing the different functions. The method used is called “best-fit spline” and may be applied at any chosen degree on the data points in order to obtain a smooth curve which is representative of the test results, Figure 3.5.

Figure 3.5. Use of best-fit spline in SEEP/W. (GEOSLOPE, 1992)
Definition of boundary conditions
The boundary conditions sometimes vary during the period over which the modelling is performed. SEEP/W includes the possibility of representing the boundary condition at any node using functions defined by the user. These functions may be used to describe the head or flow variations at specific locations, e.g. drainage of a dam reservoir or pumping test.

Another possibility available in SEEP/W is the automatic review of the boundary conditions at nodes located on the free-seepage surface (see section 3.3.1). To find the exit point on the seepage surface, different conditions may be imposed on a number of nodes (called review nodes) which will change the boundary condition applied depending on the results obtained during the different iterations. The conditions imposed may be based on the pressure computed at the nodes or the elevation of the nodes. Figure 3.6 shows an example of the use of review nodes for which a zero-flux condition will be applied as soon as the total head at the node becomes lower than its elevation during the computation.

Infinite elements
The boundary conditions are often known at some distance from the point of engineering interest (excavation, pumping well, etc.). Representing the area including the boundary may require a scale which will result in insufficient precision for the problem studied. To be able to use a suitable scale near the area of interest (near field) and impose a boundary condition located far from this area (far field), SEEP/W includes elements, called infinite elements, which have an infinite dimension in one direction, Figure 3.7. These elements enable the representation of boundary conditions prevailing at some distance without affecting the area near the point to be studied and without being forced to work with uncomfortable scales.
Convergence of the solution

The convergence of the solution may be controlled in SEEP/W by two different parameters: the number of iterations and the variations between two iterations. In the first case, the computation is automatically stopped when the number of iterations reaches the number specified.

In order to specify the criteria for acceptable convergence of the solution based on the changes between two iterations, SEEP/W computes the norm of the pressure head vector calculated as follows (GEOSLOPE; 1992):

$$||h|| = \left( \sum_{j=1}^{n} |h_j|^2 \right)^{1/2} + i0$$

(3.14)

where:
- $||h||$ = norm of the pressure head vector
- $h_j$ = pressure head at node $j$
- $n$ = number of nodes.

The convergence criteria is set as an acceptable difference in the norm of the vector between two iterations. If the difference calculated is larger than the criteria specified, the process continues until this criteria becomes fulfilled or the maximum number of iterations is reached. The convergence progress may be followed during the analysis (see example in Figure 3.8) and the calculation may be stopped if the solution does not seem to reach convergence.
Some problems with convergence may be encountered in coarse material due to the relatively steep hydraulic conductivity function, Figure 3.9. During the computation process, a small variation in pressure may result in a large calculated variation in hydraulic conductivity creating an important decrease of the flow and destabilizing the system. In such cases, the solution may oscillate between two values and convergence may never occur. To avoid this problem, a third criterion may be introduced in SEEP/W which will control and limit the changes in hydraulic conductivity between two iterations.

Figure 3.8. Evolution of the norm of the vector during computation (example). (GEOSLOPE, 1992)

Figure 3.9. Example of a steep hydraulic conductivity function. (GEOSLOPE, 1992)
Presentation of the results

Presentation of the results computed by SEEP/W may be done in different ways. The most common presentation is the contours of the parameters computed. This can be done on a large number of parameters, such as the total head, the pressure and the pressure head, the velocity, the gradient, the conductivity and the water content, Figure 3.10a. Another form of presentation is plotting the values computed at different time intervals at specified nodes, Figure 3.10b.

SEEP/W allows, therefore, the presentation of the results both with regard to the whole section studied or at any chosen nodes. This makes it easier to study the results and compare them to any type of measurement performed in the laboratory or in the field.

Figure 3.10. Presentation of computed hydraulic head (a) contours; (b) function of time. (GEO-SLOPE, 1992)
Chapter 4.

Measurement of Soil Matric Suction

4.1 PURPOSE OF THE STUDY

In order to study and eventually model the behaviour of unsaturated soils, negative pore water pressures must be measured. The quality of the predictions to be made is greatly influenced by the quality of these measurements. Therefore, an investigation of different instruments available on the market was initiated in order to study the behaviour of some of the most commonly used instruments.

As mentioned in section 2.5, many different instruments have been developed for the measurement of matric suction. Some of these instruments perform direct measurement of the matric suction using a filter element to maintain contact between the pore water and the water in a chamber in which the pressure is recorded with a pressure transducer or a manometer. Other instruments perform indirect measurement of the matric suction, i.e. the measurements are not made on the suction itself, but on a parameter which is directly related to it, such as the water content or the electrical resistivity of a defined media in contact with the soil. The suction is thereafter obtained from a calibration curve.

Instruments making indirect measurements are often used to measure high matric suction values, but are considered less reliable for measuring small negative pressures. On the other hand, instruments making direct measurements are limited to suction values of about -90 kPa. This means that both types of instrument have to be used together in order to cover the whole pressure range in a situation when the conditions fluctuate from wet to very dry soil. In such a case, direct measuring instruments will stop functioning during a dry period and will most likely require a new installation when the conditions are getting back to relatively wet with low matric suction. It is therefore of interest to closely study the behaviour of indirect measuring instruments at low pressures in order to get an idea of their accuracy and the possibility of obtaining reliable recordings also in wet conditions.
4.2 DESCRIPTION OF THE INSTRUMENTS STUDIED
For the investigation, two instruments making direct measurement were chosen together with two sensors using indirect techniques. These four instruments were tested in order to control their accuracy and response when measuring soil matric suction under different conditions.

Tensiometer
The first instrument is a tensiometer which makes direct measurement of the soil matric suction. In this type of instrument, the pore water pressure is directly transmitted to the water in the system through a ceramic porous element which enables water movement in both directions but does not permit air to enter the system. The water pressure is transmitted to the ground surface by a water column inside a plastic pipe. A readout instrument is located at (or above) the ground level. Any change in the pore water pressure due to movement of the water table, or drying or wetting of the soil around the tip is recorded by the readout instrument. If the pore air pressure is equal to the atmospheric pressure, the recording made with the tensiometer corresponds to the soil matric suction at the level of the porous element. The pressure registered is always negative relative to the atmospheric pressure, even if the soil becomes fully saturated or the groundwater level rises above the filter tip. The only situation in which the readings might become positive is when the groundwater table reaches a level higher than the readout instrument; this case should however be avoided since the instrument is then likely to stop functioning.

The tensiometer used in the study is a Soil Moisture tensiometer, Figure 4.1. It can be equipped with a manometer or a pressure transducer as read-out instrument. The pipes are made of plastic and different lengths can be combined to install the filter tip at the required depth.

Piezometer
The second instrument studied is a BAT-piezometer, Figure 4.2. This closed system piezometer comprises a filter tip, a pressure transducer and a readout unit. The filter tip has a porous element with a height of 40 mm and a diameter of 30 mm. To prevent air bubbles to enter the water chamber, a special porous element with high air entry value is used for installations in partially saturated soils. The water chamber is closed on top by a rubber disk. To perform the measurements, the pressure transducer is lowered onto the tip, and a hypodermic needle connected to the transducer penetrates the rubber disk, opening communication between the transducer and the water chamber. The pressure can then be read at ground level with the read-out unit.
Figure 4.1. Tensiometer with manometer and pressure transducer.

Figure 4.2. BAT-piezometer with read-out instrument and pressure transducer.
As for the tensiometer, the value measured with this instrument is equal to the matric suction at the level of the porous element, provided the pore-air pressure is at atmospheric pressure. The pressure registered may be negative or positive relative to the atmospheric pressure depending on whether the groundwater level rises above the filter tip. This means that the transducer used should be able to measure both positive and negative pressures and therefore be calibrated for both measuring ranges.

**Thermal conductivity sensor**
The third instrument used is a thermal conductivity sensor from Agwatronics, Figure 4.3. The sensor consists of a porous block containing a temperature-sensing element and a miniature heater (see Chapter 2).

![Figure 4.3. Thermal conductivity sensor.](image)

The thermal conductivity sensor measures indirectly the matric suction using the relation between soil suction and water content combined with the thermal conductivity of the porous element at different water contents. To perform the measurements, the temperature in the porous block is first recorded, after which a controlled amount of heat is generated at the centre of the block and a new temperature recording is made after a given time interval. The temperature rise depends on the heat dissipation rate in the porous block, which is inversely proportional to the water content in the block. Since there is a relation between the water content in the block and the matric suction in the surrounding soil, the temperature rise can be converted into soil matric suction. Calibration in any pressure range is usually provided by the manufacturer.
Electrical resistivity sensor

The last instrument used is an electrical resistivity sensor, commonly called gypsum block, Figure 4.4. It comprises a porous block of gypsum in which coaxial electrodes are incorporated (see Chapter 2).

![Electrical resistivity sensor (gypsum block).](image)

The electrical resistivity sensor measures indirectly the matric suction using the relation between the soil matric suction and the water content combined with the electrical resistivity of the gypsum block at different water contents.

To perform the measurement, an electric signal of a given intensity is sent to the block and the electrical resistivity is measured. The resistivity being directly related to the water content in the block, the matric suction can be determined using a calibration curve, usually provided by the manufacturer (Figure 4.5), in which the relation between the soil matric suction and the electrical resistance is established.
4.3 LARGE SCALE SOIL MODEL

To perform the study, a new laboratory equipment was built. The equipment consists of a large cylinder with a diameter of 1.0 m, which can reach a height of 1.6 m by assemblage of 0.4 m rings, Figure 4.6. A water reservoir situated on the outside of the cylinder is connected to the bottom of the cylinder through three porous plates 100 mm in diameter. The water level in the cylinder can therefore easily be controlled by varying the level of the water reservoir.

Figure 4.5. Calibration curve for gypsum blocks from Soil Moisture.
The cylinder can be filled with any type of soil to perform different studies. The instruments can be installed at any level in the soil specimen and the water level can be adjusted to a given level to study the properties of the instruments in steady-state conditions. The water level can also be altered more or less rapidly to perform similar analyses of the instruments’ properties at transient states.

The soil used in the tests was a fairly uniform fine sand with a capillary height of about 0.8 m. This soil was chosen in order to avoid long periods of stabilization to reach steady-state conditions after changing the water level in the cylinder. A fairly homogeneous density in the whole soil specimen was obtained by gradually increasing the water level during the preparation so that the soil was continuously placed under about 10 cm of water. The soil specimen was built up to a total height of 1.2 m.
A total of eleven instruments were installed and monitored during the test. One instrument of each type described above was installed at two different levels for comparison. In addition, two gypsum blocks with electrical resistivity sensors were installed in the upper and the middle parts of the cylinder, and one BAT-piezometer was installed close to the bottom of the soil. The location of the instruments is presented in Figure 4.7.

To perform the automatic reading and recording of the signals from the instruments, a data acquisition system was used, Figure 4.8.
**Test performed**

After build up of the soil specimen and installation of the instruments, the wa­
ter level was kept at the top of the cylinder for a certain time in order to obtain
a reliable measurement of the initial value. Thereafter, the water level was rap­
idly lowered about 0.25 m and kept at this level for about one day; this opera­
tion was repeated twice for a total lowering of 0.75 m. Measurements were
performed every 30 minutes on all instruments during the entire period.

The results from the measurements made with the Soil Moisture tensiometers
and the BAT-piezometers are presented in Figure 4.9. The results are shown as
recorded water level and can be compared with the real water level in the cylin­
der. Since the transducers used can record pressures both in the positive and the
negative fields, the level could be recorded during the whole test. As seen from
the results, both types of measuring system respond directly and correctly to the
fluctuations in the water level, showing a good agreement with each other.

![Figure 4.9. Measurements made with tensiometers and piezometers.](image)

The response from the thermal and electrical sensors has to be studied separate­
ly for each sensor since they will react differently during the test. Since the sen­
sors can only measure negative pressures, no change should be recorded as long
as the water level is above the sensor. This means that the sensors located in the
upper part of the soil should show some reaction during the last part of the test,
while the ones located in the lower part should not register any variation.
The signals registered from the thermal conductivity and the electrical resistivity sensors were, in a first step, directly transformed using the calibration curves provided by the manufacturers. However, the results from this transformation indicated that some corrections were required before a complete analysis could be made.

The measurements made with the electrical resistivity sensors clearly showed that the "zero-pressure" (the value registered when the sensor is placed in fully saturated soil) obtained after transformation using the manufacturer's calibration was not satisfactory. In order to eliminate the error caused by uncertain initial values, the calibration parameters of these sensors were adjusted so that a correct pressure was recorded in the first part of the test, i.e. when the soil in the cylinder was completely submerged. Thereafter, the interpretation of the measurements was made according to the manufacturer's calibration curves. The results of the measurements after correction of the "zero-value" are presented in Figure 4.10.

![Figure 4.10. Measurements made with electrical resistivity sensors - after correction of zero-value.](image)

Figure 4.10 shows that despite correction of the zero-value, the recording made with the electrical resistivity sensors does not correspond to the changes in water level induced in the cylinder during the test. This will be discussed further in Section 4.6 after investigation of the calibration parameters (see Section 4.5).
The measurements made with the thermal conductivity sensors show the same problem concerning the "zero-pressure" value as for the electrical sensors. They were therefore adjusted in a similar way. The measurements also show significant fluctuations. These fluctuations are due to the limits of accuracy of the measuring unit. The precision of the temperature-sensing device is about 0.05°C, which is relatively important compared to the small variations in temperature necessary to register different pressure levels. In order to reduce the effect of this low accuracy, the mean value of a number of measurements was used instead of a single measurement. The results after correction for the "zero-value" and using mean values of fifty (50) recordings are presented in Figure 4.11.

The corrected measurements shown in Figure 4.11 still present a large scattering. However, the sensor AGW-2 located in the upper part of the cylinder shows some signs of reaction to the water level variation in the last part of the test, even though the variation registered is smaller than the actual change in water level.

Figure 4.11. Measurements made with thermal conductivity sensors - after correction of zero-value and calculation of mean-value.
Preliminary conclusions
As mentioned previously, the piezometers and the tensiometers used in the study showed a good response to the variation of water level during the test. They seem to be reliable for recording both the correct water level at each step and the transient process.

However, this first part of the study clearly showed that the calibrations performed by the manufacturers of the electrical resistivity sensors and the thermal conductivity sensors are not accurate enough for these instruments to be used in the domain of pressures studied in this test. The underlying cause of the behaviour of the sensors is probably the fact previously mentioned that these instruments were primarily developed for measuring higher values and do not have a sufficient accuracy in the domain of pressures studied here. However, the inaccuracy may also come from the calibration procedure, which is not accurate enough in the domain of pressures encountered in this investigation and should therefore be made with higher precision for low pressures. Based on this assumption, a calibration of the sensors at low pressures was performed.

4.4 CALIBRATION OF THERMAL CONDUCTIVITY SENSORS
A second laboratory equipment was built in order to perform an accurate calibration of the thermal conductivity sensors. This equipment consists of a pressure cell based on the axis-translation technique (Hilf, 1956), also called a Null Type Pressure Cell, Figure 4.12.

Figure 4.12. Laboratory equipment - Null type pressure cell.
The cell is 400 mm high and has a diameter of 150 mm. At the bottom of the cell is a water reservoir in contact with the cell through a porous plate with an air entry value of -300 kPa. The water pressure is measured using a pressure transducer located at the bottom of the reservoir. The water reservoir is carefully saturated and two communication lines are used to flush, if necessary, any air entering the system and to keep the reservoir fully water-saturated. Air is supplied to the cell through the upper part and the pressure is controlled with a pressure regulator. The air pressure is measured on the supply line with a pressure transducer.

The calibration was performed by installing the sensors in a soil slurry at the bottom of the cells. The pressure in the cell was thereafter increased stepwise and the water was allowed to leave the soil through the porous plate and out of the cell through a burette connected to the water reservoir. At each step, the pressure was kept constant until no water was leaving the cell and the sensors had reached equilibrium.

Figure 4.13 shows the testing system including three pressure cells as used in the present study. To accelerate the calibration process, the cells were later modified so that two sensors could be installed in each of them.
Since it took some time for the soil to drain at each pressure step, another procedure was also tested in which the sensors were placed directly on the porous plate. This decreased significantly the volume of water which had to flow out and therefore accelerated the calibration process.

As an example, the results of the calibration of one sensor are presented in Figure 4.14 (see Appendix A for complete results). It can be seen that the stabilization of the signal from the sensor after an increase in air pressure in the cell took some time, which is the time necessary for the water content in the soil to adjust to the new pressure.

![Figure 4.14. Example of results from the calibration of a thermal conductivity sensor.](image)

This calibration gave very different calibration parameters compared to those provided by the manufacturer, Figure 4.15 (see Appendix B for the complete results). This is probably due to the fact that the reading and recording system used in this study has different characteristics than the one used by the manufacturer. This clearly shows that these instruments have to be calibrated with exactly the same system that will be used later for the measurements. The calibration also showed that the use of a straight line obtained from a linear regression is not accurate enough when the values are close to zero. The calibration parameters should instead be chosen according to the range of pressure in which the measurements are to be performed.
4.5 CALIBRATION OF THE ELECTRICAL RESISTIVITY SENSORS

A calibration of three electrical resistivity sensors was also performed. The calibration was made using the pressure cells described before and following the same procedure as for the thermal conductivity sensors (Figures 4.12 and 4.13). Readings were made using both the data acquisition system and a manual read-out instrument. Results from the calibration are plotted in Figure 4.16 where they can be compared with the curve supplied by the manufacturer. The difference between the two curves can mainly be attributed to different zero-values. The complete results of the calibration are presented in Appendix C.

The calibration of the electrical resistivity sensors permitted to obtain a relatively good relation between the resistivity and the matric suction in the range studied. However, the calibration also showed a certain instability of the sensors near saturation, which may result in inaccuracy of the measurement of low matric suctions.
Figure 4.16. Comparison of the calibration curves for electrical sensors.

The calibration also showed that the signals sent by the sensors for any applied pressure did not correspond to the values given by the general calibration curve proposed by the manufacturer, Figure 4.16. This leads to a similar conclusion to the one made for the thermal sensors, i.e. that the matric suction obtained by an electrical resistivity sensor should always be calculated from a unique and specific relation determined through accurate calibration; the manufacturer’s general calibration curve should only be used when approximative values are satisfactory.
4.6 COMPLEMENTARY INTERPRETATION

Since the calibration of the thermal conductivity and the electrical resistivity sensors has shown significant differences compared with the manufacturer's calibration, it became necessary to make a new analysis of the test results using the new calibration factors.

For the electrical resistivity sensors, the calibration curve shows only a lateral movement compared to the one supplied by the manufacturer. This means that, for the pressure range prevailing during the test, only the zero-value is different. Consequently, the results after correction taking into account the new calibration parameters would be very similar to those presented in Figure 4.10. Therefore, a detailed re-evaluation of the results is not necessary. Using the zero-values found during the calibration would provide better results. However, the fact remains that two sensors (GB-3 and GB-4) showed an inverse reaction to the lowering of the water level, making the use of electrical resistivity sensors unsuitable in the range of pressures studied.

For the thermal conductivity sensors, a re-interpretation using the new calibration parameters shows a better agreement with the variation imposed during the test. The results presented in Figure 4.17 can be compared with Figure 4.11. At zero-pressure (water level above sensors), the values obtained from the calibration correspond to the values recorded during the test. Moreover, a better response of the sensor AGW-2 located in the upper part of the soil could be evaluated after the calibration. The value obtained is much closer to the actual water level in the soil. However, the fluctuations of the mean value are greater than before, since the scale factor found during the calibration is larger than the one proposed by the manufacturer.

4.7 CONCLUSIONS

The investigation showed that tensiometers and BAT-piezometers are well suited for measurement of variations in the pore pressure in the vadose zone. The maximum value of negative pressures which can be measured with the BAT-piezometer was not investigated. It is usually assumed to be as large as for the tensiometer, i.e. about -90 kPa, when using the available high air-entry-value filter. However, many factors can affect the long term functioning of the piezometer, such as leakage through the membrane and air entry during the measuring procedure.

The test performed also showed that an accurate calibration of the thermal sensors is necessary if these instruments are to be used for measurements in the
zone located immediately above the groundwater table. Since the pore pressure in this zone is generally only slightly lower than the atmospheric pressure, and since this zone is often fully saturated, the accuracy of these instruments is closely related to the accuracy of the calibration. Therefore, calibration parameters should be chosen depending on the pressure range in which the instruments are to be used.

Finally, the results also showed that, despite the efforts made to calibrate them for the range of pressure studied, the electrical conductivity sensors did not provide any acceptable measurements during the test. The inaccuracy of these instruments when measuring small negative pressures makes them inappropriate for this type of measurement.
Chapter 5.
Laboratory Tests

5.1 INTRODUCTION
A number of tests covering different situations have been made in order to study changes in groundwater conditions resulting from various activities. In the first part presented in this chapter, the behaviour of unsaturated soils was studied in laboratory tests performed under fully controlled conditions. In these tests a lowering of the groundwater table in a soil column was generated, in order to obtain a transition from saturated to unsaturated conditions. This also gave the possibility of studying the behaviour of partially saturated soils when submitted to infiltration from above during further stages of the tests.

The main purpose of these tests was to accumulate data for testing and verifying the ability of the computer model to describe the conditions in partially saturated soils and to simulate the behaviour of such soils in different situations. Computer simulations of the tests were therefore also made and comparisons between the test results and the simulation are presented at the end of this chapter.

A secondary purpose of these laboratory tests was to continue the study of the behaviour of different measuring instruments presented in the preceding chapter. The conclusions regarding the response of the instruments used are discussed in connection with the presentation and interpretation of the test results. However, all the measurements made for this purpose are not included in this chapter.

5.2 LABORATORY EQUIPMENT
The equipment used for the laboratory tests consists of a column with a total height of 3.0 m and a diameter of 150 mm, Figure 5.1. The column is made of Plexiglas and, in order to facilitate the preparation of the soil and the installation of the instruments, it is made of two end-parts 0.75 m long and four middle-parts 0.5 m long which can be assembled using rubber O-rings to ensure the tightness of the joints between the different parts. At the bottom of the col-
Figure 5.1 - Capillary column used in the laboratory tests.
umn, a support made of stainless steel includes a small chamber which can be filled with water in contact with the soil through a filter plate. A water reservoir, whose level can be freely regulated, is connected to the bottom of the column. When the soil has been placed in the column, the water level in the soil can be controlled by moving the reservoir to selected levels.

The pore-water pressure can be measured at five different levels using filter tubes inserted through holes located at regular intervals along the column. The measuring system shown in Figure 5.2 consists of a filter tube, a rubber joint (to seal around the hole in the column), a rubber membrane (to close the system), and a pressure transducer or a manometer.

A data acquisition system of type CR10 described earlier in Chapter 4 (see Figure 4.3) was used to make and record the measurements from all the instruments.

5.3 TEST #1 - FINE SAND

5.3.1 Installation
In the first series of tests, a soil consisting mainly of fine sand was used in order to obtain a relatively rapid response in the measuring instruments. The grain size distribution for the soil is shown in Figure 5.3. The soil was placed in the column and lightly compacted. The soil and the water levels were raised.
simultaneously and in small steps in order to obtain a fairly regular compaction and to avoid inhomogeneities in the soil sample. The final height of the soil column was about 2.6 m.

The hydraulic properties of the soil were measured separately in smaller columns (Figure 5.4) which were used to establish the characteristic water retention curve for both the drying and the wetting phases, Figure 5.5. The pF-curve was completed by measuring the water content of the soil after stabilization at different pressures in the cell described in Chapter 4 (section 4.4). The hydraulic conductivity of the saturated sand was determined using the grain size distribution and in falling head tests performed in the columns mentioned above. The values of the hydraulic conductivity obtained were between $4 \times 10^{-6}$ and $5 \times 10^{-5}$ m/s.
During the preparation of the soil in the column, a number of instruments were installed in the soil at selected levels. A total of two Soil Moisture tensiometers, two BAT-piezometers, three Agwatronic sensors and three gypsum blocks were installed at different levels, as shown in Figure 5.6.

Figure 5.5. Results from determination of water retention curve for sand.

Figure 5.6. Schematic of instrumentation in test column - test #1.
5.3.2 Water level variation test

The first part of the test consisted in lowering the water level in the soil. This was done in two steps of about 1.0 and 1.2 m respectively by changing the level of the water reservoir. The water volume leaving the sample was measured manually at regular intervals while the pressure recorded by the instruments was registered automatically with the data acquisition system.

The results of the measurements made with the filter tubes installed in the column are presented as pressure head in Figure 5.7. They show that the pressure transducers connected to the filter tubes responded logically and relatively rapidly to the first variation in the water level, except for the filter tube CYL5 located at the highest level. It seems that the filter element lost contact with the surrounding soil and came in contact with the atmospheric pressure, since the pressure recorded showed values close to zero. This was probably caused by small deformations in the soil column which would affect this filter tube as it was located rather near the soil surface.

![Figure 5.7](image.png)

Figure 5.7. Results of measurements made during variation of the water level - measurements in filter tubes located along the column.

During the second step of the lowering of the water level, the response of the two filter tubes located in the lowest part of the column showed a good agreement with the variation imposed while the two transducers connected to the tubes at the levels directly above showed a very slow response, stabilizing at pressure levels which were too high in relation to the position of the piezometric level.
These measurements indicate that the response of the pressure transducers connected to the filter tubes along the soil sample was directly related to the variation of the water level, as long as the filter element was located in the tension-saturated zone. Figure 5.8 shows the changes in water content in the column during the two steps in which the water level was lowered. In the first step, all filter tubes were located in the fully saturated or in the tension-saturated zones, which resulted in values corresponding to the piezometric level. However, in the second step, the filter tubes CYL3 and CYL4 were above the tension-saturated zone and therefore recorded pressure levels which was less than the piezometric level of the saturated zone. These transducers did not follow the variations in pressure when the porewater channels were no longer continuous. After this stage, the soil surrounding these filter tubes was not in direct contact with the water table and the measurements were therefore not affected by further changes in the water level.

The measurements made with the tensiometers and the piezometers installed inside the soil sample confirmed the behaviour observed above. Figure 5.9 shows how the pressure in the soil was lowered according to the different water levels.
levels imposed during the test. The first change of water level was recorded directly and correctly by these instruments, while the response to the second level change was a slow stabilization process to a final level which was higher than the actual piezometric level.

The measurements made with the thermal conductivity and the electrical resistivity sensors were, as expected, not sufficiently accurate to register correctly the changes in the water level. However, some response was registered by the sensors during the test but it was not in proportion to the actual variations.

5.3.3 Infiltration test
After maintaining the water level at the bottom of the soil sample for some time, the second phase of the test was started which consisted of infiltration of water through the soil column. The infiltration was initiated by pouring water on top of the soil sample. Two different procedures were followed: first, a given amount of water was poured on top of the sample and left to sink freely through the soil until a steady-state was obtained again. In the second phase, water was poured on top of the soil until the free water level was about 20 cm above the soil. This level was kept constant for a few hours by pouring additional water before it was left to sink freely again.
During the infiltration process, the pressure inside the filter tubes located along the soil column was continuously recorded in order to enable a study of the changes occurring at different levels as water was seeping down the column. The results of these measurements are presented in Figure 5.10. They show clearly that pouring water on top of the soil affected the hydraulic conditions in the column almost instantaneously, but also that a second response was registered after some time, Figure 5.11.

The first reaction recorded in all transducers was probably caused by an increase in the air pressure in the pores due to the infiltration of water. The filter tube CYL5, which did not function properly before, reacted very rapidly and recorded a pressure corresponding to the level of free water, which may be seen as a confirmation of the assumption made previously of direct contact with the soil surface. The pressure in the filter tubes CYL4 and CYL3 increased rapidly to the level at which they had stopped following the piezometric level during the lowering of the water level (see section 5.3.2); this increase is probably a combination of the increase in air pressure mentioned above and a small increase in water content (in this part of the column, a small increase in water content in the surrounding soil may result in a noticeable increase of the negative pressure). The same phenomenon, although less pronounced, was also observed in CYL2.

**Figure 5.10.** Results of measurements made in filter tubes during infiltration of water in the column - test # 1 (sand).
The second reaction recorded was due to the passage of the front of infiltration water which means that the pressure recorded was controlled by the free water level instead of the groundwater level in the column. Since the contact with the free water was made at different intervals for different filter tubes, the reaction was registered at different times. In the first infiltration test, the volume of infiltration water was not enough for the water front to reach the two lowest filter tubes before the free water sank into the soil, which is why they did not show the same response as the others, Figure 5.11a. However, in the second test, the free water level was kept constant long enough to influence all the transducers, Figure 5.11b. In both cases, the pressure level restabilized after some time at the level registered before infiltration was initiated.

As in the first phase of the test in which the water level was lowered (section 5.3.2), the measurements made during infiltration with the tensiometers and the piezometers installed in the soil were very similar to the measurements made in the filter tubes, Figure 5.12. Even in this case, the results show a rapid and almost immediate reaction followed by a new increase after a few hours, probably when the front of infiltrating water reached the measuring devices.

The thermal conductivity sensors showed small and uncertain reactions during the infiltration phase, while the electrical resistivity sensors did not show any reaction at all. Obviously, the pressure levels observed in this test were too low for these instruments.

After restabilization of the pressures in the column, the test was stopped and the water content at different levels determined, Figure 5.13. It shows a good agreement with the water retention relationship determined earlier in the small capillary columns (see Figure 5.5).
Figure 5.11. Details of measurements in filter tubes during infiltration of water - test #1 (sand).
Figure 5.12. Results of measurements made during infiltration of water - measurements made in piezometers and tensiometers.

Figure 5.13. Results from determination of water retention curve after the test #1.
5.4 TEST #2 - SILT

5.4.1 Installation
In the second series of tests, a soil consisting mainly of silt was used. The soil, whose grain size distribution is shown in Figure 5.14, was placed in the column following the same procedure as for the test #1. The total height of the soil column was about 2.7 m. The water retention relationship of the soil was determined using the small column previously described (Figure 5.4) after modification to increase the height to about 3 m, Figure 5.15. Some additional measures were made in the pressure cell to complete the relationship established in the capillary column. The hydraulic conductivity of the soil was also determined in this column and was found to be about $7 \times 10^{-7}$ m/s.

Different instruments were installed in the soil during the preparation of the soil column following basically the same procedure and choosing approximately the same locations as in the first column (see section 5.3.1). A total of two Soil Moisture tensiometers, two BAT-piezometers, three thermal conductivity sensors and three gypsum blocks were installed at different levels, as shown in Figure 5.16.

![Grain size distribution for silt used in test #2.](image-url)
Figure 5.15. Results from determination of water retention curve for silt.

Figure 5.16. Schematic of instrumentation in capillary column - test #2.
5.4.2 Water level variation test
The first part of the test consisted in lowering the water level in the soil column. The lowering was performed in three steps of about 0.7, 0.65 and 0.9 m respectively, by regulating the level of the water reservoir. The water volume leaving the column was measured manually at regular intervals while the pressure recorded by the instruments was registered automatically with the data acquisition system.

The measurements made in the filter tubes along the column are presented in Figure 5.17. The first observation which can be made is that all the transducers connected to these tubes, with the exception of the one located in the upper part of the column, showed a direct correspondence with the water level in the column during the whole process. This is different from the results obtained from the sand column where only the transducers located in the lower part of the column responded directly during the last step. However, this is in line with the explanation given earlier for the behaviour observed in the sand column. Since the water retention characteristic curve of the silt shows an air entry value which is higher than 20 kPa, this means that the soil was nearly fully saturated in the whole column even when the water level has been lowered by about 2.0 m, and that the pressure level in the column was directly influenced by changes in the water level.

![Figure 5.17](image_url)

Figure 5.17. Results of measurements made in filter tubes during lowering of the water level in the column - test #2 (silt).
The response from the tensiometers and the piezometers is in very good agreement with the actual changes in water level in the column, Figure 5.18. Once again the measurements made with these four instruments placed inside the soil column at two different levels confirm the behaviour observed in the filter tubes.

![Figure 5.18](image)

**Figure 5.18.** Results of measurements made during lowering of the water level - measurements with piezometers and tensiometers.

The thermal conductivity sensors showed a better response to the variations in the water level than in the first test, even though the variations in water content were rather limited. On the other hand, the measurements made with the electrical resistivity sensors did not accurately register the changes in water level.

### 5.4.3 Infiltration test

The second part of the test consisting of infiltration of water from the top of the soil column was performed following a similar procedure as in the first test made with sand (section 5.3.3). The infiltration test was made in two phases. The first phase consisted in pouring a given volume of water on top of the soil, which was left to seep freely through the soil. The second phase consisted in pouring enough water to reach a free water level situated about 30 cm above the soil and continuing to pour water in order to keep the water level constant during a few hours. The water level was left free to sink when the water volume coming out at the bottom of the column had become equal to the volume poured in, which corresponds to a constant flow through the soil.
During the infiltration process, the porewater pressure inside the soil was registered relatively often in order to study the pressure variations at different locations at different times. The results of these measurements are presented in Figures 5.19 and 5.20.

Figure 5.19. Results of measurements made in filter tubes during infiltration of water in the column - test #2 (silt).

Figure 5.20. Results of measurements made during variation of water level - measurements made in piezometers and tensiometers.
The results presented above clearly show that the infiltration water affected the hydraulic conditions rather instantaneously in all measuring devices. One important difference compared with the results from the first column is that there seems to be only one rather instantaneous reaction of the transducers to the addition of water, Figure 5.21. This can probably be explained by the water

![Graph](image_url)

**Figure 5.21.** Details of measurements made in filter tubes during infiltration of water - test #2 (silt).
retention characteristics of the silt used for the test, since the loss of water in the silt in the upper part of the column is small and almost negligible compared to the substantial decrease in water content in the sand used in the first column. This explains why there was no noticeable delay in the increase in pressure in the whole column and no detectable front of infiltrating water.

As in the first test, the water content at different levels was determined after restabilization of the pressures in the column, Figure 5.22, showing a good agreement with the relationship established earlier in the capillary columns (Figure 5.15).

Figure 5.22. Results from determination of water retention curve after the test #2.
5.5 SIMULATION OF COLUMN TESTS WITH COMPUTER PROGRAM AND COMPARISON WITH OBSERVED BEHAVIOUR

The tests performed in the column were simulated using the computer program SEEPW described in Chapter 3. The main purpose of this study was to control the ability of the soil model in the program to simulate the different processes observed in the laboratory tests.

The simulation was performed using 8-node elements to form a column representing the soil column in the two tests, Figure 5.23.

![Figure 5.23. Mesh used for simulation of laboratory test.](image-url)
The procedure followed during the laboratory tests was reproduced in three steps presented in Figure 5.24.

![Figure 5.24. Procedure used for simulation of the laboratory tests.](image)

(a) Steady state  
(b) Water level variation  
(c) Infiltration

The characteristic water retention curves used for the simulation were those measured in the capillary column as presented earlier in this chapter. The hydraulic conductivity functions were estimated using the method developed by Green and Corey (1971) (see section 2.4.3) and were based on the water retention curves and the initial hydraulic conductivity measured.

In order to make a comparison between the results from the computer simulation and the actual recorded behaviour, the results of the simulation are presented in Figures 5.25 to 5.28 together with the measurements made during the tests. As seen from these figures, the behaviour of the silty material used in the test #2 can be simulated with very good agreement both during the lowering of the water level and the infiltration test. However, the behaviour of the sandy material used in the test #1 was much more difficult to simulate with the computer model, showing some dissimilarities with the measurements, especially during the infiltration test.

The problems encountered with the test simulation in the sandy material are probably due to the steep hydraulic conductivity function. In the first phase of the infiltration test, the computed pressure head did not quite reach the values measured with the instruments in the upper part of the soil column. Moreover, in both phases, the instantaneous response of the transducers followed by a
delayed second response depending on their position could not be found in the simulation. Instead, the model shows an increase of the pressure head starting at different intervals depending on the position of the transducers, which is theoretically correct but does not fully correspond to the behaviour observed during the tests.

However, despite the small dissimilarities between the measured and the computed behaviour, the study of the volume of water flowing in and out of the soil during the test showed a rather good agreement between the observations made during the test and the calculated volume, Figure 5.29. This shows that the modelling of the test in the sand column was rather close to the real behaviour and certainly reliable.
Figure 5.26. Comparison of the measurements (symbols) and the results from the numerical simulation (full lines) - test #1 (infiltration).

Figure 5.27. Comparison of the measurements (symbols) and the results from the numerical simulation (full lines) - test #2 (water level variation).
Figure 5.28. Comparison of the measurements (symbols) and the results from the numerical simulation (full lines) - test #2 (infiltration).

Figure 5.29. Comparison of the volume of water measured during the test in sand and the volumes computed during the simulation.
5.6 CONCLUSIONS

Different conclusions can be drawn from this laboratory study. The first one concerns the behaviour of the soil and the ability of the pressure transducers to measure negative pore pressures in the partially saturated zone. The two most reliable instruments used in this study, the Soil Moisture tensiometer and the BAT-piezometer, registered a similar behaviour to the one observed in the filter tubes placed along the column. The results from the measurements made during the test performed in sand showed that the pore pressure in the soil at the level of the measuring instruments corresponded to the position of the water level as long as the instruments were located in the tension-saturated zone. However, the pressure decreased only marginally when the water level was lowered again, showing that the porewater in the soil surrounding the instruments was no longer in full contact with the water in the fully saturated zone. This indicated that the negative pore pressure in the soil above the capillary zone was not controlled by the position of the water table, but took different values depending on the flowing process involved in this zone controlling the water content. The test performed in silt showed a good correlation between the measurements and the water level in the column. This was expected since the water retention curve of the silt shows a much higher air entry pressure than the sand, which means that the soil in the whole column was near full saturation and that all the porewater was therefore in contact with the water in the saturated zone.

Another important observation is that the “high air entry” filter elements used as filter tips in piezometers and tensiometers succeeded in preventing the water to flow out of the tip. This conclusion can be made after studying the reaction of these instruments during the infiltration tests. Any pressure change caused by the addition of water on top of the column or by variation in the water content in the surrounding soil was correctly and rapidly recorded, which indicates that the filter tips were fully saturated.

The simulation of the sand column gave overall results in good agreement with the measurements in the fully saturated and the tension-saturated zones. However, the theoretical values obtained in the soil above the capillary zone did not correspond to the observed behaviour. This is due to the fact that the soil model in the computer program assumes a continuous porewater channel even at very low degrees of saturation, which is not true in practice. The simulation also showed that the behaviour of soils with steep hydraulic functions (water retention and hydraulic conductivity) is difficult to simulate and that great care need to be taken in choosing the time steps for the computation. A reliable simula-
tion requires a very accurate determination of the characteristic hydraulic functions.
The simulation made for the test in silt was relatively easy to perform and gave acceptable results for both test phases (water level variation and infiltration).
The smoother form of the characteristic curves (water retention and hydraulic conductivity), compared to the sand, makes it easier to perform numerical simulations with good accuracy and reliability in such soils, the changes in hydraulic properties being much smaller and less sudden.
Chapter 6.

Full Scale Field Test at Linköping

6.1 INTRODUCTION
A field test consisting in measurements of changes in the groundwater conditions during pumping tests was made at one site located outside Linköping, Sweden. The purpose of the test was to simulate the effect of an excavation under well defined conditions, and to study the changes in the groundwater conditions in the immediate surroundings of the excavation. To obtain a lowering of the water table, a drainage system was installed about 2.5 m below the ground surface. The system consisted of a drainage pipe about 100 m long connected to a well in which the water level could be lowered by pumping. The natural groundwater level in the area was between 0.5 and 1.0 m below the ground surface, which means that a groundwater lowering of about 1.5-2.0 m could be achieved with this installation.

These measurements were later used for comparison with the results obtained from simulation with the computer program. The tests also gave the opportunity of studying the behaviour of different measuring instruments under field conditions.

6.2 GEOTECHNICAL PROPERTIES
The test site is situated outside the city of Linköping, in the south-eastern part of Sweden, called Tokarp, Figure 6.1. The topography of the site is quite regular and flat with a small hill nearby.

Figure 6.1. Location of the test site.
6.2.1 Site investigation
The soil properties have been investigated earlier by the City of Linköping for the installation of a water supply system. The site investigation included Static Penetrometer Soundings and Percussion Soundings. Samples were taken with an auger sampler for soil classification. Measurements of the groundwater level were also made.

Additional sampling was made to determine the water content and the grain size distribution of the soil. The hydraulic conductivity was determined in the field, in some of the piezometers installed for the present study.

6.2.2 Soil properties
The results from the site investigation indicated that the upper soil in the area consists of a layer of about 2.5-3.0 m of silt with thin lenses of clay. Underlying the silt is a silty, sandy moraine (till).

The natural water content varies between 13 and 24 % in the soil samples investigated. The grain size distribution curves presented in Figure 6.2 show a relatively large range for the material investigated. However, it can be seen from the curves that the soil is rather coarse immediately below the ground surface, becoming progressively finer with increasing depth.

![Grain size distribution, Linköping](Figure 6.2. Grain size distribution, Linköping.)
6.2.3 Hydraulic conductivity
The hydraulic conductivity was measured in open standpipes and piezometers installed at the site for the monitoring of the pumping test. The determination of the hydraulic conductivity was made in variable and in constant head tests in open standpipes and in variable head tests in BAT-piezometers. The results presented in Table 6.1 indicate a rather wide range of hydraulic conductivity, showing a clear decrease with increasing depth.

Table 6.1. Results from field measurement of the hydraulic conductivity, Tokarp test field in Linköping.

<table>
<thead>
<tr>
<th>Depth investigated</th>
<th>Hydraulic Conductivity, m/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>between 1 and 4 m</td>
<td>7.3 e-7</td>
</tr>
<tr>
<td></td>
<td>4.7 e-6</td>
</tr>
<tr>
<td></td>
<td>4.0 e-7</td>
</tr>
<tr>
<td></td>
<td>9.0 e-8</td>
</tr>
<tr>
<td>between 4 and 6 m</td>
<td>1.4 e-8</td>
</tr>
<tr>
<td></td>
<td>7.8 e-9</td>
</tr>
<tr>
<td></td>
<td>7.0 e-9</td>
</tr>
<tr>
<td></td>
<td>5.4 e-9</td>
</tr>
<tr>
<td></td>
<td>3.5 e-9</td>
</tr>
<tr>
<td></td>
<td>2.5 e-9</td>
</tr>
<tr>
<td></td>
<td>1.6 e-9</td>
</tr>
</tbody>
</table>

6.3 MEASURING SYSTEMS USED IN THE FIELD TESTS

6.3.1 Measurement of pore pressure and groundwater level
Measurements of the groundwater conditions were made for different purposes, resulting in the choice of different instruments. Natural variations in the groundwater level were followed using open standpipes, while rapid changes in the groundwater level and the pore pressures were recorded by means of piezometers.

The open standpipe used was a Geotech filter tip which is made of a perforated pipe of steel partly filled with sand and epoxy forming the filter, Figure 6.3. Readings of the water level in the open systems were made manually using a conventional readout instrument consisting of a coaxial cable and a signal indicator.
The closed system used to measure pore pressure was a BAT-piezometer equipped with a standard filter tip with a plastic filter element, Figure 6.4. Readings of the pore pressure in the closed system were taken manually by lowering a pressure transducer onto the filter tip or automatically by leaving the transducer permanently on the tip and connecting it to a data acquisition system.

The accuracy of the measurements is about 2 cm when using open standpipes, and 0.5 kPa for the BAT-piezometer.
6.3.2 Measurement of soil suction
Two types of instruments were used for measuring the soil suction in the partially saturated zone. The first type of instrument was a Soil Moisture tensiometer using a pressure transducer when measurements were recorded automatically during rapid changes in pore pressure, or a manometer when the measurements were made manually at given intervals. The second system was a BAT-piezometer equipped with a ceramic filter element with a high air entry value (filter with a pore size 2). The accuracy of both systems is about 0.5 kPa. Both instruments were described earlier in Chapter 4.

6.3.3 Measurement of the water flow
Measurement of the flow of pumped water during the field tests was made with a spin flow meter of type Pollux SPX (7060-25). The flow meter was equipped with two independent recording systems:
- a counter with a conventional display and a precision of 0.1 litre.
- a pulse recorder with a frequency of 1 pulse/10 litre.

The pulse recorder was connected to a data acquisition system for automatic reading and recording of the flow.

6.3.4 Measurement of temperature and atmospheric pressure
The air temperature was measured and recorded using a thermoelement of Type T (ASTM E230-83), with an accuracy of about 1°C. The atmospheric pressure was measured with a relative pressure transducer with an accuracy of about 0.05 kPa.

6.3.5 Automatic data acquisition system
The data acquisition system of type AAC-2 was used for automatic reading and recording of different parameters, Figure 6.5. The acquisition system was equipped with 24 channels and was able to read and directly record the data from different types of transducers, and to store the values after being processed according to the calibration characteristics of the transducers.
6.4 INSTALLATION

6.4.1 Drainage system
The purpose of the installation was to simulate a lowering of the ground-water caused by an excavation. In order to create a lowering over some length, a drainage system consisting of a 100 m long drainage pipe was placed about 2.5 m below the ground surface, Figure 6.6. The drainage pipe was, at one end, connected to a well with a diameter of about 600 mm. In order to limit disturbance of the soil surrounding the drainage system, the installation was made

*) see Figure 6.10 for instrumentation in cross-sections 1 and 2

Figure 6.6. Drainage installation.
with a ploughing machine. Since this type of machine could not install the pipe at a depth of more than about 1.6 m below the ground level, a layer of 1.0 m was first excavated before the pipe was ploughed down. The excavated material was then backfilled and the ground surface levelled off to its original level, Figure 6.7.

Two similar installations were made using two different types of drainage system. In the first installation, a conventional drainage pipe covered with a fiber filter was installed, Figure 6.8a. In the second installation, the drainage system consisted of a band drain of type “Hydraway”, Figure 6.8b.

![Figure 6.7. Ploughing down of drainage system.](image)

![Figure 6.8. Type of drainage system used in the investigation.](image)
6.4.2 Instrumentation

A total of forty-seven, (47) instruments (32 BAT-piezometers, 11 Geotech open standpipes and 4 Soil Moisture tensiometers), were installed in the area in order to monitor the variations in the groundwater conditions, Figure 6.9.

Figure 6.9. Plan over the installation.
For each drainage installation, two cross-sections were instrumented with six piezometers of type BAT with standard filter element, two Geotech open standpipes, one BAT-piezometer with “high air-entry value” ceramic filter and one Soil Moisture tensiometer, Figure 6.10.

Outside the immediate area for the drainage installations, four BAT-piezometers and three Geotech open standpipes were installed as references for natural variations in the groundwater conditions in the area during the pumping tests (see Figure 6.9).

![Figure 6.10. Typical instrumented cross-section.](image)

### 6.5 RESULTS FROM PUMPING TESTS

#### 6.5.1 Pumping tests

Pumping tests were performed by lowering the water level in the well during a period varying between 3 to 5 weeks. The first test was performed in the drainage system A in the spring of 1992 (Test A92), but this test did not provide any useful results because the groundwater level was unusually low in the area during this period. A second series of tests was performed in both systems A and B during the winter and spring of 1993 (Test A93 and B93).
A few weeks before the start of a pumping test, the 14 piezometers and the 2 tensiometers located near the drainage system in which the test was to be performed, as well as the 4 piezometers used as reference, were connected to an automatic monitoring system. The water flow, the atmospheric pressure and the temperature were also registered using the same monitoring system. These parameters are presented in Figure 6.11.

During the pumping test, the measuring interval in the monitoring system was chosen so that the changes in the groundwater conditions were reasonably well monitored. The open standpipes were also read more frequently at the beginning of the test in order to monitor the rapid changes in the groundwater level.

![Graphs showing water flow, atmospheric pressure, and temperature measurements during pumping tests.](image)

*Figure 6.11. Measurements of water flow, atmospheric pressure and temperature during pumping tests: (a) Test A-93; (b) Test B-93.*
6.5.2 Interpretation and presentation of test results

As a first step in the interpretation, a certain time was chosen for each test as initial time for the measurements. At this initial time, all values recorded were set to zero and only changes in groundwater level in relation to the initial zero-values were computed and presented.

The natural variations in the groundwater occurring during the periods of monitoring were established using the measurements made in the four piezometers, and the three open standpipes located outside the immediate area of the drainage systems. **Figure 6.12** and **6.13** present these measurements together with the averaged variations used in the other steps of the interpretation.

![Figure 6.12](image1)

*Figure 6.12. Measurements performed during test A93 - reference.*

![Figure 6.13](image2)

*Figure 6.13. Measurements performed during test B93 - reference.*
Figures 6.14 and 6.15 present some of the measurements made during the two pumping tests A93 and B93, together with the natural variations in groundwater levels recorded during these periods. The complete set of measurements made during the tests can be found in Appendices D and E.
Several problems were encountered with the measuring equipment, which was not surprising considering that the measurements were performed in the field during a rather long period of time. The most serious problem occurred during test A93 when low night temperatures caused temporary failures of the record-
ing system between April 8th and 16th. Manual readings were made until a heating element was installed in the box protecting the data acquisition system. Some other problems with the reading system were encountered during test B93, which were mainly related to failure of the power supply.

A few instruments also showed irregularities during the period of measurement. The open standpipe 42 and the piezometer 28, which were located rather close to Highway E4, seem to have been installed in a layer which was not in direct contact with the aquifers studied, responding in a strange way to both the natural and the generated groundwater variations. Also, some piezometers stopped functioning properly after failure of the power supply. The tensiometer and the piezometer installed above the water table in section B2 did not record any reasonable values during test B93, probably due to an improper installation.

The transient state of the groundwater conditions across the drainage system can be followed from the measurements made during the tests. The lowering of the groundwater level at different time intervals after the start of the pumping test in each cross-section is shown in Figures 6.16 and 6.17.

![Figure 6.16. Groundwater conditions at different time intervals, test A93.](image)

![Figure 6.17. Groundwater conditions at different time intervals, test B93.](image)
The measurements of the water flow during the tests show that the steady-state was not quite reached during the tests, Figure 6.11. The time required for reaching steady-state was expected to be too long for a continuation of the tests until it was obtained. However, the lowering of the groundwater in the area was judged to be so close to the steady-state that a prolongation of the tests would not result in significant changes in the piezometer readings.

6.5.3 Correction of measurements for variations in atmospheric pressure

Pore-water pressure measurements were made using absolute pressure transducers and had therefore to be corrected for variations in the atmospheric pressure (see Tremblay, 1989). All the measurements made with piezometers were subjected to this correction.

For the piezometers located in the saturated zone, this correction gave the expected results, eliminating irregular fluctuations in the recordings. However, the piezometers located in the unsaturated zone did not follow the same pattern as those installed in the saturated soil. Although the measurements were made using absolute pressure transducers, the readings did not seem to systematically be affected by variations in the atmospheric pressure. Figures 6.18 and 6.19

![Graph](image-url)
Figure 6.19. Influence of variations in the atmospheric pressure to the measurements made in partially saturated soil, Test B-93.

shows measurements made in BAT-piezometers installed above the drainage system. The soil surrounding these piezometers was fully saturated at the beginning of the pumping test but became partially unsaturated during the test. Both the uncorrected and corrected results are presented, together with the corrected measurements made in a piezometer which was always surrounded by fully saturated soil.

These measurements clearly indicate that, as long as the instruments were located in saturated soil, the variations in the atmospheric pressure affected the water pressure which was reflected in the recordings made with the absolute pressure transducers. However, when the pores were filled with water and air, the variations in the air phase did not seem to directly affect the pressure in the water phase.

6.6 COMPUTER SIMULATION

6.6.1 Description of the model
The pumping tests performed in Linköping were simulated using the computer program SEEPW described in Chapter 3. The simulation was made with 8-node elements forming the mesh presented in Figure 6.20. The drainage sys-
tem was modelled as a quadratic hole in the mesh with an area similar to the drainage pipe. The elements near the drainage system were given smaller dimensions than those in the surrounding soil. This was made in order to obtain a better discretization of the variations in the groundwater regime near the pipe, and was possible because the soil properties were fairly homogeneous. Infinite elements were used at the extremities in order to minimize the size of the mesh in the horizontal direction.

The soil deposit was described in the model as two layers of 4 m each. The properties of these layers were chosen according to the results of the investigation presented earlier in section 6.2. The top layer was described as a silty/sandy material, and the other layer as a silty, sandy moraine. The characteristic water retention curve of the top layer is given in Figure 6.21a. It is an average curve based on the information available (grain size distribution, natural water content, water retention tests on similar material) and information on the existing curves in the database of the computer program. The hydraulic conductivity function of the layer (Figure 6.21b) was estimated using Green and Corey equation, based on the water retention curve mentioned above and results from the investigation regarding the initial value of the hydraulic conductivity. Since the moraine layer was to be constantly below the water table, the choice of the characteristic water retention curve was not considered to be important since it would not affect the results in any way. On the other hand, the initial hydraulic conductivity might have some influence on the results and was therefore chosen more carefully, based on the investigation performed in the field. The value of the initial hydraulic conductivity was found to be about $5 \times 10^{-9}$ m/s.

The outer boundary conditions were taken directly from the measurements performed during the test in the reference instruments (see section 6.5.2). A small infiltration flow was introduced on the upper boundary of the model for simulation of the periods of rain. The pumping test was simulated by lowering the total head in the drainage pipe to the level of the pipe in the actual section. The
6.6.2 Results from the computer simulation
The simulations were performed without any major difficulties regarding the convergence of the solution. However, many problems were caused by the use of infinite elements combined with the variation of the outer boundary conditions. It appeared that the infinite elements were damping away the natural groundwater variations recorded in the area and applied to the boundary. In order to proceed with the simulation, it was therefore necessary to eliminate these elements, increasing the size of the mesh and using only regular elements.

Some of the results obtained from the simulations are presented in Figures 6.22 to 6.24, together with corresponding measurements for comparison. As seen from these results, the simulations were generally in good agreement with the behaviour observed during the test. However, some difference were observed regarding mostly the lowering of the groundwater at a certain distance from the drainage, the simulation always giving a larger influence area than the measurements.
6.7 CONCLUSIONS
The main conclusion which can be drawn from the tests and the numerical simulations is that it is possible to simulate with good accuracy the variations in groundwater conditions due to an induced lowering of the water table. Since
the hydraulic properties of the soil have to be given in the numerical model, it is necessary to determine them with relevant methods. A sensitivity analysis of the results from the modelling due to variations of different parameters showed, as expected, that the hydraulic conductivity is the most important parameter, having an almost direct influence on the results in the transient state. In a situation such as the one prevailing during these tests, i.e. when the lowering is relatively small and the influence of the unsaturated zone is limited, it seems that the characteristic water retention curve may be established, with acceptable accuracy, on the basis of the grain size distribution.

The measurements also showed that the zone which was influenced by the lowering of the water table was not as extensive as indicated by the results from the model simulation. This may be due to different reasons, as for example, the difficulty to accurately determine and describe the stratigraphy of the site and an eventual anisotropy of the hydraulic conductivity.

Finally, the measurements performed in both the piezometers and the tensiometers installed in the upper part of the saturated zone showed that variations in the atmospheric pressure do not affect the pore-water pressure when the soil becomes partly unsaturated.
7.1 MEASUREMENT OF NEGATIVE PORE PRESSURES

The investigation performed on four different types of measuring equipment showed that instruments directly measuring the pore pressure, such as tensiometers and piezometers, are well suited for recording negative pore pressures in the range recorded in this study. The measuring principle of these instruments enables measurement of suctions down to about -90 kPa, provided that they are equipped with filter elements with high air entry value. Variations in pore pressure can also be registered with good accuracy using these types of instrument. The accuracy of the measurements is mainly controlled by the type of reading instrument (manometer or pressure transducer) used.

The results obtained from thermal conductivity sensors confirmed that these instruments are not suitable for measuring pressures in the range encountered in this study. If such instruments are to be used for different reasons, it is important to remember that their ability to measure small values of matric suctions is very much dependent on the accuracy of the calibration available. The recordings made with these instruments show large fluctuations, which makes them unsuitable for making measurement with a single reading. In order to obtain an acceptable value of the actual matric suction, it is necessary to make several readings at short time intervals and to calculate the mean value.

The tests performed also showed that electrical resistivity sensors are not suitable for measuring small matric suction. The problems encountered with zero-values of the recordings make it impossible to use these sensors outside the range for which the calibration characteristics are provided by the manufacturer (< -20 kPa).

Measurements performed in the field showed that the variations in atmospheric pressure did not affect the instruments located in the partially saturated zone. It was found that the recordings made with absolute pressure transducers should be corrected for changes in atmospheric pressure as long as they are located in
the saturated zone, but that such corrections are unnecessary when the soil surrounding the filter tip becomes partially disaturated.

7.2 VARIATIONS IN PORE PRESSURES IN PARTIALLY SATURATED SOIL

The laboratory tests performed in a soil column showed the variations in pore pressure in the unsaturated zone due to a lowering of the water table and during infiltration and percolation of water. The measurements performed in a silt showed that the pressure variations in the tension-saturated zone, and at levels immediately above the tension-saturated zone, are directly controlled by the water level in the soil column. However, during the test performed in sand, the increase in suction in the unsaturated zone (especially in the upper part of the soil column) due to the lowering of the water table showed a different pattern, in which the suction stopped increasing and went towards stabilization at a lower suction level compared to the value obtained in silt. The infiltration tests performed in the two different materials basically followed a similar pattern, recording an increase in pressure due to the addition of water and eventually a re-stabilization to the level recorded before infiltration.

These tests clearly showed that the behaviour of unsaturated soil is not only controlled by the groundwater level in the soil, but also by the water retention characteristics of the soil. A decrease in pore pressure corresponds to the lowering of the water level as long as the pore water is in full contact with the water in the saturated zone. When this contact is broken due to drainage of water (decrease in water content), the suction reaches a stabilized level, independently of the position of the water table. Consequently, the variations in suction in the unsaturated zone is directly related to the thickness of the capillary zone. This was observed in the test in the sand column, during which the pressure recorded stopped decreasing when the instrument became located above the tension-saturated zone according to the water retention curve established for this type of soil. Further drying or lowering of the water content could, however, lead to a further decrease in pore pressure.

7.3 COMPUTER SIMULATION OF VARIATIONS IN PORE PRESSURE

The simulations of variations in pore pressure recorded during the various tests performed in the laboratory and in the field tests have pointed out some important factors to consider. The results clearly showed that the numerical simulation of groundwater lowering in saturated soil is relatively straightforward, as long as the hydraulic properties of the soil have been determined with good accuracy. The concordance between the simulation and the actual behaviour of
the soil also depends on the accuracy to which the stratigraphy of the soil deposit has been established and modelled.

The simulation of the soil behaviour in partially saturated conditions has also been successful as long as the matric suction is slightly lower than the air entry pressure. When the variations in matric suction are to be simulated above this level, the idealized theoretical model does not correspond to what really happens in the soil, and the behaviour is therefore not correctly computed. This problem is specially important in soils, such as sand, with relatively low air entry pressure and steep hydraulic functions (water retention and hydraulic conductivity relationships), or when the unsaturated zone is relatively thick, as in high natural slopes.

The simulation was generally easy to perform, except when the soil was characterized with steep hydraulic functions. In this case, the time steps used in the simulation had to be chosen very carefully in order to obtain convergence of the solution as well as useful results.
Chapter 8

Future Research

During the course of this project, many different questions have arisen. Some of them have been partly answered, but many still require more research before a desirable level of understanding is reached.

Unfortunately, modelling of the behaviour of partially saturated soils is made using an idealized representation based on theory only. Now that research performed in many institutions has provided a better understanding of the behaviour of this zone, some efforts should be made to find a physical/mathematical model which would follow more closely the actual behaviour. This type of model would be very useful for predicting more exactly the variations in water conditions when subjected to any altering activity (excavation, pumping, infiltration and percolation of rain water, etc.).

In order to acquire a better knowledge of the actual variations, more measurements should be performed around real excavations and road cuts. The information required for comparison with the results from simulation does not demand a large number of instruments and expensive equipment, and it should therefore be possible to obtain such valuable data rather easily.

Finally, it can be seen from the governing equations of the model that two properties of the soil which are not always determined for geotechnical purposes are required for this type of analysis: the hydraulic conductivity and the storage capacity. Many methods have been developed for the direct determination of these parameters, but they are seldom used in practice for different reasons (demand complex equipment and specialized personnel, high cost, time-consuming, etc.). Fortunately, there are several empirical or semi-empirical models presented in the literature, which can be used for this purpose. However, the accuracy of these methods has to be studied more carefully before any of them can be recommended.


Appendices
Appendix A.1 - Results from the calibration of thermal conductivity sensors #1039 and 1063.
Appendix A.2 - Results from the calibration of thermal conductivity sensors #1069 and 1091.
Appendix A.3 - Results from the calibration of thermal conductivity sensors #1095 and 1100.
Appendix B.1 - Comparison of calibration coefficients for thermal conductivity sensors #1039 and 1063.
Appendix B.2 - Comparison of calibration coefficients for thermal conductivity sensors #1069 and 1091.
Appendix B.3 - Comparison of calibration coefficients for thermal conductivity sensors #1095 and 1100.
Appendix C - Results from calibration of resistivity sensors.
Appendix D.1 - Measurements made during test A-93.
Appendix D.2 - Measurements made during test A-93.
Appendix D.3 - Measurements made during test A-93.
Appendix D.4 - Measurements made during test A-93.
Appendix E.1 - Measurements made during test B-93.
Appendix E.2 - Measurements made during test B-93.
Appendix E.3 - Measurements made during test B-93.
Appendix E.4 - Measurements made during test B-93.
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The purpose of the Institute is to achieve better techniques, safety and economy by the correct application of geotechnical knowledge in the building process.

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

Information
Research reports, brochures, courses. Running the Swedish central geotechnical library with more than 85,000 documents. Computerized retrieval system.

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

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