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An evaluation of methods to determine K_0' in clays

Literature study and pilot experiments conducted at Tiller in connection with NGTS

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Preface

This project thesis in geotechnical engineering is written as a part of the MSc in Civil and Environmental Engineering at the Norwegian University of Science and Technology during the autumn of 2016.

The original idea for this project thesis was proposed by co-supervisor Dr. Jean-Sebastien L'Heureux at NGI. This report represents a careful selection of ideas suggested by the authors, supervisor Professor Steinar Nordal and co-supervisor Dr. Jean-Sebastien L'Heureux.

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Additionally, we would like to thank Zeynep Ozkul at NGI and Samson Degago at the Norwegian Public Roads Administration for help and support in processing of the field data from Tiller.

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Summary and Conclusions

Knowledge of the in situ stress state of soils is considered essential in many different geotechnical engineering challenges. The vertical stress may easily be evaluated by considering the overburden load. The horizontal stress is considered much more complex, being a result of several different factors. As a consequence, it is the horizontal stress state in situ which represents a problem for the determination of the coefficient of earth pressure at rest, K_0'.

To gain a better understanding of the factors affecting the horizontal stress state, as well as the development within field measurements of the horizontal stress state, a study of published literature is presented and evaluated. In some of the literature the method of hydraulic fracturing, the earth pressure cell and the Camkometer self-boring pressuremeter are used as reference methods when investigating the validity of other methods. These methods represent direct measurements of the horizontal stress state, while other in situ methods are more dependent on empirical correlations.

There exist many empirical and theoretical correlation methods to determine K_0'. Jaky's formula for normally consolidated soil is widely acknowledged. However, there is more debate on which correlations are true for overconsolidated soils. Additionally, a few laboratory methods for determining K_0' have been briefly presented in this report.

The results of pilot field tests using a dilatometer and earth pressure cells at the Norwegian Geo-Test Site for soft clay at Tiller is presented. The testing was conducted to learn more about the equipment and to evaluate the quality of the results. These initial experiments indicate that both types of equipment may be utilized for the determination of K_0' in soft clays.

However, the results presented in this report suggest that the dilatometer overestimates K_0'. Further investigations are required to verify the obtained results and to give a more complete evaluation of the equipment.

Contents

List of Symbols and Abbreviations

Acronyms

Symbol	Description	Units
CPT	Cone Penetration Test	
CRS	Constant Rate of Strain Oedometer test	
DMT	Dilatometer	
NGI	Norwegian Geotechnical Institute	
$NGTS$	Norwegian Geo-Test Sites	
$NPRA$	Norwegian Public Roads Administration	
$NTNU$	Norwegian University of Science and Technology	
OCR	Overconsolidation Ratio	
$SCPT$	Seismic Cone Penetration Test	
$SDMT$	Seismic Dilatometer	
$UNIS$	The University Centre in Svalbard	

Greek Symbols

Symbol	Description	Units

α	Slope parameter	-
β_k	Soil type parameter, dilatometer	-
χ	Modified Cam-Clay slope ratio parameter	-
η	Anisotropic elastic stiffness parameter	-
γ	Unit weight	kN/m^3
ϕ'	Effective friction angle	$\circ$
σ_h	Total horizontal stress	kPa
σ'_h	Effective horizontal stress	kPa
σ_t	Tensile strength of the soil	kPa
σ_v	Total vertical stress	kPa
σ'_v	Effective vertical stress	kPa
σ'_{h0}	Effective horizontal overburden stress	kPa
σ'_{ph}	Horizontal preconsolidation pressure	kPa
σ'_{pv}	Vertical preconsolidation pressure	kPa
$\sigma^{'}_{v0}$	Effective vertical overburden stress	kPa

Roman Symbols

Symbol	**Description**	**Units**
a	Attraction	kPa
c	Cohesion	kPa

E_D	Dilatometer modulus	kPa
G	Elastic shear modulus	kPa
I_D	Material index, dilatometer	-
I_L	Liquidity index	-
I_P	Plasticity index	%
K_0	Coefficient of earth pressure at rest for total stresses	-
K'_0	Coefficient of earth pressure at rest for effective stresses	-
K_D	Lateral stress index, dilatometer	-
K_P	Coefficient of passive earth pressure	-
K'_{0nc}	Coefficient of earth pressure at rest for normally consolidated clay	-
K'_{0oc}	Coefficient of earth pressure at rest for overconsolidated clay	-
p_0	Initial pressure value, dilatometer	kPa
p_1	Inflated pressure value, dilatometer	kPa
p_2	Deflated pressure value, dilatometer	kPa
p_f	Fracturing pressure inside borehole	kPa
p_u	Cavity expansion pressure	kPa
s_u	Undrained shear strength	kPa
u	Pore pressure	-

u_0	Initial pore pressure	kPa
w	Water content	%
w_L	Liquid limit	%

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Chapter 1

Introduction

During the last decades, geotechnical challenges have gained an increasing amount of focus and many questions have been resolved. However, there are still topics puzzling researchers around the world. The determination of the horizontal stress state and the coefficient of earth pressure at rest is one of the subjects that have proven very complicated.

This chapter looks into the background, the problem formulation and the chosen approach to the problem. The objectives as well as limitations of this project will also be introduced, before the chapter ends with a general overview of the structure of the rest of the report.

1.1 Norwegian Geo-Test Sites

A research project called "Norwegian Geo-Test Sites" was launched in June 2016 to help evaluating and developing methods for soil investigation in Norway. The project, which is a cooperation between NGI, NTNU, SINTEF, UNIS and the Norwegian Public Roads Administration, centers around five different test sites located across mainland Norway and Svalbard. As part of the project several different research topics were proposed.

Amongst them is the much debated and researched issue regarding the horizontal in situ stress state, which is necessary to determine the coefficient of earth pressure at rest, K_0'.

1.2 Problem Formulation and Approach

Despite much research and numerous investigations, the uncertainties connected to the horizontal stress state in situ is still substantial. This may be due to two primary factors. First of all, the understanding of the factors affecting the horizontal stress in soil materials is still not satisfactory. Second, making in situ measurements of the horizontal stress state tend to impose stress changes to the soil, and hence the measured stress may deviate from the in situ stress state. This is also true for laboratory experiments.

The first approach to the problem of determining the in situ horizontal stress will be to perform a study of the literature available on the topic. The main focus will be on in situ test methods, while laboratory and correlation methods will be discussed more briefly. The second approach will be to perform pilot experiments in order to make a preliminary evaluation of the dilatometer and the earth pressure cell.

1.3 Main Objective and Limitations

The overall goal of all K_0' investigations is to be able to determine the true K_0' in situ. Since this goal is too extensive to be met in this report, the objectives of this project are more limited.

In this project, the primary objective is to present and evaluate methods to determine the coefficient of earth pressure at rest in situ. It should also briefly treat some correlation and laboratory methods used to determine K_0'. The geological processes and the stress history creating the stress state in situ should only be treated very briefly.

A secondary objective of this project is to perform pilot experiments with the earth pressure cell and the dilatometer at the Tiller site. The extent of these experiments is limited by lack of time in the autumn semester.

1.4 Structure of the Report

The remaining part of this report is structured into four chapters. Chapter 2 Theory and Background gives a quite thorough presentation of the development within the literature on the horizontal stress state and K_0'. Chapter 3 Pilot Experiments deals with the pilot experiments conducted at the Tiller clay site. First, a detailed presentation of both the equipment and utilized procedures is given, before the results are presented and briefly discussed in light of previous findings. Chapter 4 Discussion includes a discussion of the findings in the previous chapters. Finally, Chapter 5 Summary provides a summary of the report, presents key findings and experiences, as well as recommendations for further work.

Chapter 2

Theory and Background

This chapter focuses on the theory available on K_0'. First, K_0' is defined and a general introduction is given to why it is of importance to find good estimates for K_0'. Second, a selection of in situ methods for determining K_0' are treated. Finally, some laboratory and correlation methods to determine K_0' are presented.

2.1 Acquiring Knowledge

The gathering of relevant information was mainly carried out through the use of Google Scholar and the university library service Oria. Examples of the most important key words used were: ”in situ”, ”earth pressure cell”, ”coefficient of earth pressure at rest”, ”hydraulic fracturing” and ”K0”. Searching combinations of these key words resulted in many relevant articles appearing among the first results. Also, co-supervisor Dr. Jean-Sebastien L’Heureux provided some articles and a user manual for the earth pressure cell as well as other field equipment documentation. In addition, several interesting articles were identified as references in other articles. These articles were then found, either through the Internet or by the help of Dr. Jean-Sebastien L'Heureux.

2.2 Definition of K_0'

The relationship between the in situ horizontal and vertical stress is usually expressed by a factor called the coefficient of earth pressure at rest (Sivakumar, et al. 2002). The relationship in Equation 2.1 was first proposed by Donath in his 1891-paper for total stresses (Hamouche, et al. 1995, Brooker & Ireland 1965).

$$K_0 = \frac{\sigma_h}{\sigma_v} \tag{2.1}$$

To obtain a relationship between the horizontal and vertical effective stresses, the effect of the pore pressure u is subtracted. The resulting expression for K_0', as seen in Equation 2.2, is the one that will be used in this report as the coefficient of earth pressure at rest.

$$K_0^{'} = \frac{\sigma_h'}{\sigma_v'} \tag{2.2}$$

2.3 Motivation

In many types of geotechnical engineering, there is a strong need to gather as much information as possible about the stress situation in the soil at the desired site. In order to do accurate testing and research in the lab, it is crucial to recreate the stress situation in situ. As an example, consider the consolidation phase in a triaxial test. For the triaxial test to give as valuable results as possible, it is of great importance that the stress state is recreated as close to the in situ situation as possible (Watabe, et al. 2003).

Knowing the stress situation in situ is not only important for testing in the laboratory. Doing calculations on retaining structures using a finite element analysis will rely on the user specifying enough correct information to the program in order to recreate the stress situation (Watabe et al. 2003, Sivakumar et al. 2002). Additionally, for predicting deformations in connection with retaining walls, piles, tunnels, slopes, dams and excavations, the initial stress situation is of great importance (Sivakumar et al. 2002). Also hand calculations of the forces acting on retaining structures will depend on the initial stresses in the ground.

Typically, the stress state experienced by soil materials is divided into a vertical and a horizontal component. Establishing the vertical stress is a rather straight forward task (Lefebvre, et al. 1991), as it is assumed to be a function of the overburden load alone (Massarsch 1975, Lefebvre et al. 1991, Sivakumar et al. 2002). The overburden load may be determined by multiplying the density of the overburden material by the depth and subtracting the pore pressure. On the contrary, several different factors are suggested to affect the horizontal stress situation, making a general determination difficult.

2.4 The Horizontal Stress State

As a consequence of the great importance in many geotechnical problems, much research has centered around mapping the factors complicating the determination of the in situ horizontal stress. In general, two main factors seem to play an important role.

First of all, it is clear that the horizontal stress is affected by the loading history of the deposit investigated (Hamouche et al. 1995, Brooker & Ireland 1965, Lefebvre et al. 1991, Sivakumar et al. 2002). Clay that has only experienced primary loading is often referred to as normally consolidated, while overconsolidated clays have a preconsolidation pressure higher than the present overburden pressure due to unloading or aging effects (Sivakumar et al. 2002). Sivakumar, et al. (2009) stated that the value of K_0' is constant during first loading, as an increase in vertical loading also affects the horizontal stress. However, in succeeding unloading of the material, the linearity between the vertical and horizontal stress is no longer valid. The vertical stress will reduce more than the horizontal. This is illustrated in Figure 2.1. For soil deposits with high overconsolidation values, i.e. where the preconsolidation pressure is much higher than the current loading, the horizontal stress is even likely to be greater than the vertical stress. Hence, the K_0'-value tend to increase with the overconsolidation ratio (OCR) (Sivakumar et al. 2009). This has also been confirmed experimentally (see for instance Hamouche et al. (1995)).

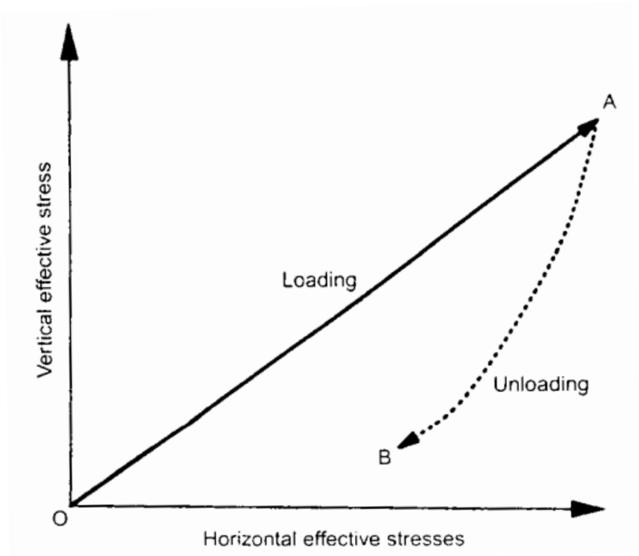


Figure 2.1: Vertical and horizontal pressures are affected by loading and unloading. Figure from Sivakumar et al. (2002).

One straightforward example of such overconsolidated materials are clays deposited towards the end of the last ice age in Norway (Gylland, et al. 2013). As the ice melted and the land began to rise, the clays deposited at the sea bed were brought up to dry land. Subsequent erosion resulted in unloading of the overburden pressure and therefore overconsolidated clay materials were formed. In addition to this, more complicated effects like drained creep and cementation may also to some extent influence the stress situation (Fioravante, et al. 1998, Ku & Mayne 2013). Drained creep is the result of small strains being imposed by constant loading of a soil deposit over time. The understanding of these effects is however limited (Fioravante et al. 1998) and is beyond the scope of this project.

Second, as the horizontal stress situation is vulnerable to even small changes in the loading or strain situation (Fioravante et al. 1998), gaining reasonably accurate measurements of the horizontal stress state without disturbing the initial stress state has proven difficult (Fioravante et al. 1998). Several of the proposed field methods utilize a cone or plate system being penetrated into the soil materials. This is likely to alter the stress state and may also give a local rise in pore pressure. As the amount and kind of disturbance vary between the different methods used and with the type of soil deposit tested, several authors have argued that the methods in general show insufficient reliability (Fioravante et al. 1998).

2.5 Proposed Methods

The importance of information regarding the horizontal stress situation has given rise to an extensive amount of research and development within the field. Numerous experiments have been conducted, resulting in a wide range of proposed methods and correlations. The methods for calculating K_0' may be divided into three categories. The first two are in situ methods and laboratory methods. The third method is based on correlations between K_0' and data acquired in situ or in the laboratory (Hamouche et al. 1995, Fioravante et al. 1998).

For the in situ methods it is possible to make a further division into intrusive and non-intrusive methods (Fioravante et al. 1998). When the intrusive methods are performed, the soil is significantly disturbed due to the penetration of test equipment. This may lead to wrong measurements and bad repeatability of the tests. Most in situ methods are intrusive, and these will be the main focus of this report. The non-intrusive methods try to minimize the amount of disturbance imposed on the soil. These methods include self-boring pressuremeters and equipment for measuring shear wave velocity (Hamouche et al. 1995). In addition, an important factor is whether the method is depending on one or more empirical factors or correlations to give the horizontal stress. In the subsequent part some of these in situ methods, laboratory methods and correlations will be presented more closely.

2.6 Hydraulic Fracturing

The method of hydraulic fracturing has proven very versatile and has been utilized within several different disciplines. In the oil industry, the method has been used to increase reservoir production for decades (Andersen, et al. 1994). The method has also been applied in rock mechanics (Bjerrum & Anderson 1972), as well as for measuring permeability in soils and to evaluate leakage from earth fill dams (Andersen et al. 1994). Furthermore, during the last decades, hydraulic fracturing as a method of investigating the in situ horizontal stress state in soils (Andersen et al. 1994, Bjerrum & Anderson 1972) has gained reputation as one of the more reliable methods for determining K_0', especially in soft clays (Lunne & L'Heureux 2016).

The technical background for the hydraulic fracturing method is rather simple. By injecting water into a borehole, the pressure inside the borehole will increase until a crack is generated (Bjerrum & Anderson 1972, Andersen et al. 1994). When the cracking occurs the pressure p_f inside the borehole is given by the equation below (Andersen et al. 1994):

$$p_f = \sigma_{h0}^{'} + u_0 + \sigma_t \tag{2.3}$$

where σ_{h0}' is the initial effective horizontal stress, u_0 is the initial pore pressure and σ_t is the tensile strength of the soil (Andersen et al. 1994). For the crack to open, the water pressure has to overcome the tensile strength of the soil and be larger than the horizontal total stress. If conditions below ground water is assumed, the total horizontal stress consists of both the horizontal effective stress and the pore pressure. Both pressures will counteract the pressure resulting from water injected into the borehole. It should however be noted that, the test material must have a limited permeability in order for fractures to open. This limits the method to use in fine-grained materials. In coarser material, the injected water will tend to drain away from the tip without causing any defined crack (Lefebvre et al. 1991, Bjerrum & Anderson 1972).

Initial experiments with the method as a way of measuring the in situ stress state was conducted by Bjerrum & Anderson (1972). Typically, a piezometer is used to inject and pressurize fluid inside a borehole. The

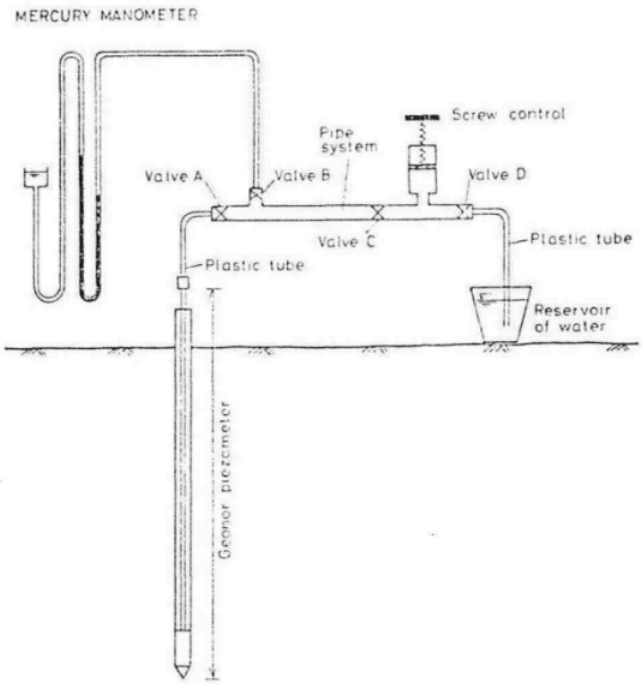


Figure 2.2: Equipment used for hydraulic fracturing. Figure from Bjerrum & Anderson (1972).

equipment used by Bjerrum & Anderson (1972) is shown in Figure 2.2. The pressure required to fracture the soil can be influenced by soil disturbances generated by the installation of the piezometer. Consequently, Bjerrum & Anderson (1972) proposed the closing pressure of the already generated crack as a more reliable indication of the horizontal stress situation. After generating a crack, the observed water pressure will decrease as water flows through the soil material. A typical pressure against time curve is shown in Figure 2.3. The water flow will continue at a relatively high rate as the pressure is reduced, until the closure pressure of the crack is reached. At this point a substantial reduction of water flow is observed (Bjerrum & Anderson 1972).

The crack closure will to a great extent reduce the amount of water flowing through the soil materials, and the drop in water flow will be easily detectable (Bjerrum & Anderson 1972). Based on continuous measurements of both water flow and the water pressure inside the borehole,

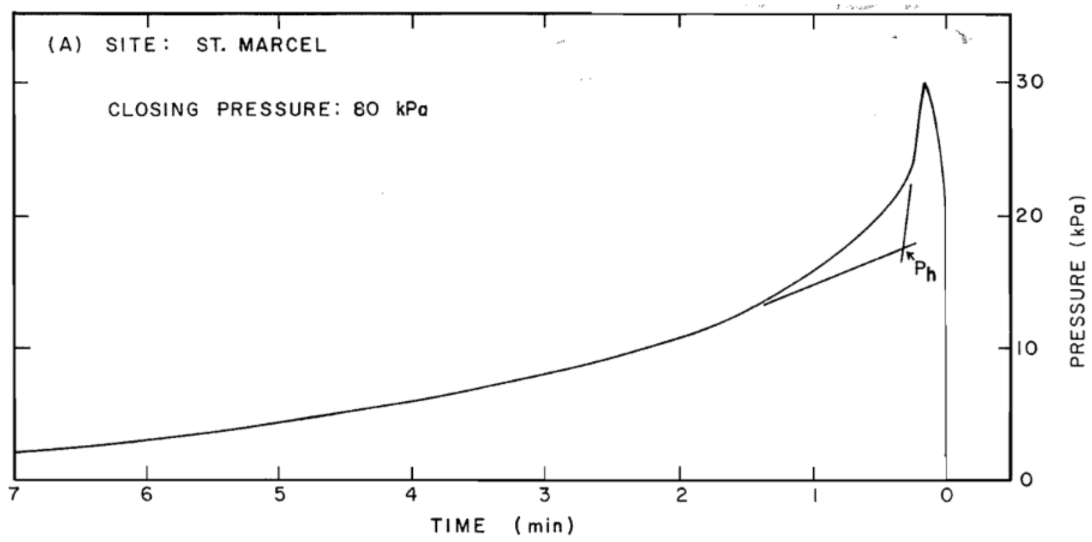


Figure 2.3: Typical pressure against time curve. Please note the direction of the horizontal axis. Figure from Lefebvre et al. (1991).

the pressure may be determined at the point when a reduction in water flow is observed. This pressure is believed to be a rather good estimate of the horizontal total stress at the depth of the piezometer (Bjerrum & Anderson 1972, Lefebvre et al. 1991).

The original mechanism believed to explain hydraulic fracturing is that an increase in pore pressure inside the borehole will reduce both the minor and major stress equally (Panah & Yanagisawa 1989, Lefebvre et al. 1991, Bjerrum & Anderson 1972). As the minor principal stress will be the first to become tensile, the cracks will emerge perpendicular to the minor principal stress direction. In a uniform soil deposit in flat terrain with K_0' smaller than one, the minor principal stress direction will be horizontal and therefore vertical cracks will open due to the hydraulic fracturing. On the contrary, in soil deposits with K_0' larger than one, horizontal cracks will open, as the minor principal stress direction is vertical (Panah & Yanagisawa 1989). Such fractures can not be used to evaluate the horizontal stress state (Bjerrum & Anderson 1972). As a consequence, Bjerrum & Anderson (1972) stated that the method was limited to use in normally consolidated clays with K_0' smaller than one (Bjerrum & Anderson 1972, Massarsch 1975).

The relationship between the minor principle stress direction and the directions of the emerging fractures has been named as a major uncertainty of the method (Massarsch 1975), and has subsequently been further investigated in several later articles. For example, Lefebvre et al. (1991) the authors applied the method to five sites in Eastern Canada consisting of Champlain sea clay with OCR ranging from 1.6 to 4.8.

Lefebvre et al. (1991) determined K_0' values as high as 4.0 at the five sites tested. This supported their proposal that higher OCR would give K_0' values above one. While examining the block samples collected after conducting the hydraulic fracturing tests, the authors observed vertical fractures in samples from all testing sites (Lefebvre et al. 1991).

The discovery of vertical fissures in combination with values of K_0' larger than one, contradicts the fundamental hypothesis of the hydraulic fracturing method, where the fracture direction is directly linked to the minor principal stress direction (Bjerrum & Anderson 1972). However, as the findings in Lefebvre et al. (1991) suggest, the minor principal stress direction may not be the determining factor of the directions of the fissures. Haimson (1978) proposed that in rock mechanics, the shape of the borehole will to a great extent determine the fracturing direction. For long cylindrical boreholes, the first fractures will always be vertical, independent of the major and minor stress directions.

Moreover, Massarsch (1978) found that vertical fractures will emerge in both normally and overconsolidated clays. As a consequence, by using cavity expansion theory, he proposed that hydraulic fracturing in clays could be viewed as an expansion of an infinitely long cylindrical cavity. Thus, the increase in pressure caused by injected water will try to expand the vertical borehole and hence create vertical fractures, independent of the minor and major stress directions. The pressure needed for this expansion is given in Equation 2.4 below:

$$p_u = s_u \cdot (1 + ln\left(\frac{G}{s_u}\right)) \tag{2.4}$$

where s_u is the undrained shear strength of the material and G is the elastic shear modulus (Andersen et al. 1994). However, it should be noted that this equation assumes a constant shear modulus and isotropic undrained shear strength. As the shear modulus varies with the pressure applied and the undrained shear strength is often anisotropic, the formula may only be used as an approximation (Andersen et al. 1994). In addition, this new proposal is also able to explain one of the findings made in the article by Lefebvre et al. (1991), namely an apparent dependency between the fracturing direction and the length of the piezometer tips. If a short piezometer tip, with length of either 10 mm or 100 mm was used, Lefebvre et al. (1991) observed a combination of both vertical and horizontal cracks. The longer tip lengths of 300 mm and 500 mm gave mostly vertical cracks. As the short tip lengths will tend to concentrate the water pressure over a shorter vertical distance, a horizontal crack is more likely to occur with a short tip length compared to a longer tip.

Not only did Lefebvre et al. (1991) find supportive evidence for the cavity expansion theory of Massarch, they also reported both vertical fractures and well defined closing pressures for highly overconsolidated clays. However, for tests at a site with OCR of about 4.8, no clear bend in the dissipation curve (see Figure 2.3) was observed. Hence, no clear closing pressure could be interpreted, making the determination of a reliable K_0' value impossible. At such levels of overconsolidation, the stiffness of the clay as well as the amount of fissures is believed to affect how the water dissipates from the fracture.

When comparing their results to the theoretical relation between K_0' and OCR given in Equation 2.20 in section 2.15, Lefebvre et al. (1991) found that the hydraulic fracturing tests generally gave much higher values of K_0' than those found by using the theoretical relationship. These findings were supported by Hamouche et al. (1995), which compare hydraulic fracturing, self-boring pressuremeter and dilatometer tests conducted at three different Champlain clay sites in Eastern Canada. Several possible explanations for this were suggested. In general, the Champlain clay tested is known for a naturally close-bonded structure. By assuming that this structure was created while the deposit was exposed to the peak load, a later unloading of the clay will not give any particular reduction in the horizontal stress, as this is prohibited by the structure of the material (Lefebvre et al. 1991). The structure may lead to higher K_0' values than those predicted by theory. As the reason for overconsolidation deviate between the different sites tested, the hypothesis about the structure of the clay was to some extent questioned by Hamouche et al. (1995). Due to scatter in the K_0' values found from Equation 2.20, Hamouche et al. (1995) concluded that field measurements is the only way to obtain accurate values of K_0'.

One key aspect when using this method, is the amount of soil disturbance caused by inserting the piezometer. The effect of disturbance around such a cylindrical probe inserted into soil materials has been discussed in several papers. Lacasse & Lunne (1989) stated that a flat dilatometer blade (see section 2.9) is likely to cause less disturbance than that created by a cylindrical probe. Hughes & Robertson (1985) stated that a cylindrical piezometer tip will create a cylinder of compacted soil around the tip. This complicates measurements of an undisturbed horizontal total stress due to disturbance of the soil, as well as arching effects which may result in too low radial stresses acting on the piezometer.

Lefebvre et al. (1991) argue that the cracks opened by hydraulic fracturing may extend into undisturbed areas further away from the borehole and hence enable stress measurements in undisturbed soil. After inducing hydraulic fractures by means of piezometers, the soil material in which the piezometer tip was penetrated, was recovered using a Sherbrooke block sampler (Lefebvre et al. 1991). Later laboratory investigations showed that using a tracer as injection fluid highly improved the visibility of the fractures, and that the fractures generated by hydraulic fracturing had generally expanded into undisturbed soil.

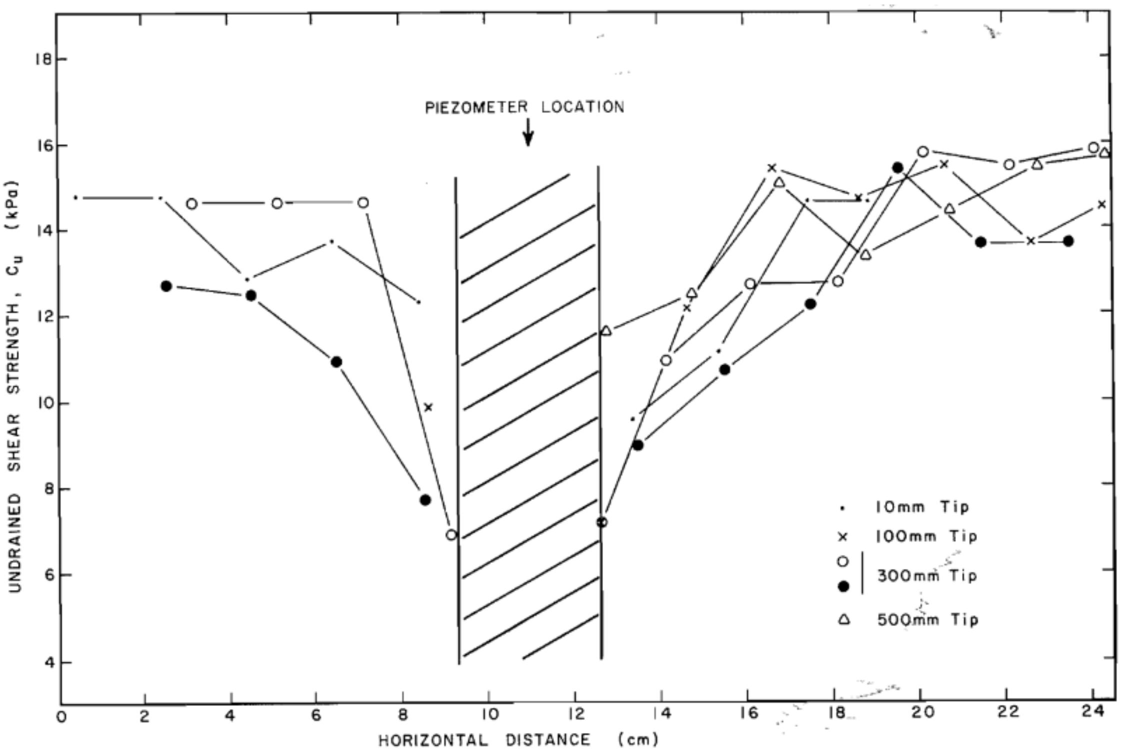


Figure 2.4: Results from fall cone tests close to piezometer location, Canadian clay. Figure from Lefebvre et al. (1991).

Additionally, Lefebvre et al. (1991) also examined the local disturbance caused by the penetration of the piezometer tip in the soil. A simple, although quite rough estimate of this may be achieved by using the fall cone test on the retrieved block samples. The test was carried out by comparing the undrained shear strength determined from the intrusion depth of the cone at varying distances from the original location of the piezometer. An example of the obtained results is presented in Figure 2.4. Generally, the authors found that in a distance of about 30 to 40 mm away from the piezometer rod, the soil strength was mostly unaltered by the penetration of the piezometer tip. Also, the authors emphasize that the cause of the remoulded zone close to the piezometer may partly be the extraction of the piezometer. This late remoulding will not affect the determination of the hydraulic fracture closing pressure (Lefebvre et al. 1991).

In addition to local disturbances, the penetration of a piezometer tip will typically also give an increase in pore pressure. For cohesive materials like clay, which behave undrained in short term loading, this increase will gradually dissipate as the material is drained over time. Failure to let the excess pore pressure dissipate before performing the test is likely to result in an opening pressure being higher than the undisturbed horizontal total stress (Bjerrum & Anderson 1972). In Bjerrum & Anderson (1972), it was suggested to leave the piezometer in the ground for several days before starting the tests. Furthermore, the time required for consolidation of the disturbed zone is found to vary with type of soil. In Lefebvre et al. (1991) this period of time varied between 5 and 20 weeks.

Finally, some authors have investigated the use of the BAT probe as equipment for performing hydraulic fracturing. The BAT probe was invented by Bengt Arne Torstensson and is primarily considered a tool for in-situ pore pressure and permeability measurements (Torstensson 1984). The probe may be used much like a piezometer for hydraulic fracturing. The main difference is that the BAT probe features a water tank inside the probe which suddenly releases the water into the soil. This creates instant fracturing, and by the help of a pressure transducer the opening and closing pressure of the cracks may be registered (Lunne & L'Heureux 2016).

2.7 Earth Pressure Cell

The earth pressure cell is a thin spade-shaped cell able to measure the total stress perpendicular to the bore hole direction and additionally the pore pressure. The older versions did not have an integrated pore pressure measuring device, and therefore the pore pressure had to be measured by a piezometer in the vicinity of the earth pressure cell (Massarsch 1975). When the horizontal effective stress is calculated from the measurements with the earth pressure cell, K_0' can be determined if the overburden pressure is known from the density of the overburden soil.

The earth pressure cell is a sealed hydraulic system filled with oil. The pressure in the oil depends on the total stress acting on the cell. In the old models, there is a valve in the cell which opens when the applied pressure from a hose running to terrain level equals the pressure in the oil. A manometer connected to the pressurized hose measures the pressure in the oil (Lunne & L'Heureux 2016). In newer models, there is a vibrating wire sensor which reads the oil pressure (Lunne & L'Heureux 2016) and the pore pressure (see Appendix B). Figure 2.5 shows the test setup for an old earth pressure cell without vibrating wire sensors.

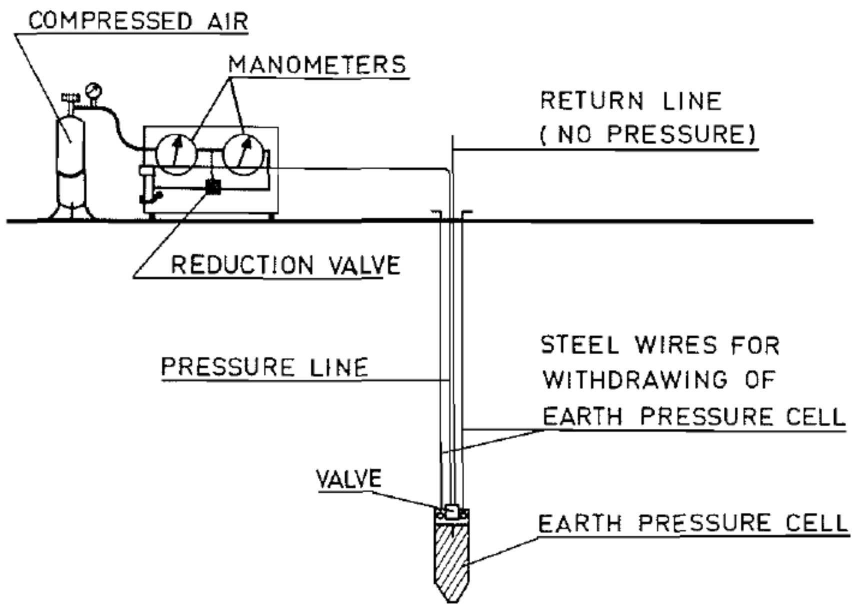


Figure 2.5: Example of old earth pressure cell equipment without vibrating wire sensors. Figure from Ryley & Carder (1995).

There exist several different earth pressure cell models with various geometries. For instance, Massarsch (1975) used a Glötzl cell with dimensions 200 mm x 100 mm x 4 mm. In Vaslestad (1989) a smaller cell with dimensions 140 mm x 70 mm x 4 mm is tested. There are also models with thicknesses of 5 mm (Tedd & Charles 1981) and 6 mm (Ryley & Carder 1995). It is desirable to make the cell as thin as possible, since the insertion of the cell into the soil causes disturbance. There is a tendency that the cells over-read due to the compaction of the adjacent soil during installation (Ryley & Carder 1995), and this effect will be elaborated in a later paragraph. Vaslestad (1989) reports that miniature cells developed in England have given promising results with less over-read than the ordinary size cells.

The earth pressure cell is pushed into the ground by for instance a drill rig. It is possible to predrill a hole to avoid pushing the cell a long way, possibly through firm layers (Ryley & Carder 1995). Some of the models feature a protective casing, allowing the cell to be pushed even through soft silt or loose sand (Massarsch 1975) and stiff London clay (Tedd & Charles 1981). If a hole is pre-drilled or a protective casing is used, it is important that the cell alone is pushed into undisturbed soil. There are different procedures suggested. In Tedd & Charles (1981) and Ryley & Carder (1995) the cell is pushed 0.5 m into the bottom of the bore hole. Massarsch (1975) suggest that the cell should be pushed 0.3 m without the protective casing.

Similarly as for the method of hydraulic fracturing, some time is required in order to let increased pore pressure due to soil compaction around the cell dissipate (Hamouche et al. 1995). Massarsch (1975) found that after installation about a week was needed to dissipate the increased pore pressure caused by installation in soft clay. In stiff to very stiff London clay the measurements had almost stabilized after a month, but there were still a few minor changes during the next month in the deepest cells (Ryley & Carder 1995). Tedd & Charles (1981) report stable values in London clay up to 12 m depth after about a month. Since modern versions of the earth pressure cell often incorporate both total stress and pore pressure measurements in one unit, the changes caused by dissipation of excess pore pressure may be monitored continuously. An example of pressure versus time curves is given in Figure E.1 in Appendix E.

Tedd & Charles (1981) compared results obtained by earth pressure cells to Camkometer self-boring pressuremeter and Camkometer self-boring load cell for a London clay underlying Claygate beds, which is alternating clay and sand. In general the earth pressure cells measured the highest total horizontal stress and the Camkometer self-boring load cell the lowest. The Camkometer self-boring pressuremeter measured on average something in between the other two. Tedd & Charles (1981) concluded that if the average of the measurements from both the Camkometers is assumed to be the true value of the horizontal stress, then the earth pressure cells tend to over-read. This is as one should expect from a push-in method causing compression of the soil around the cell. However, the reproducibility of the results from the earth pressure cell was greater than for the two Camkometers (Tedd & Charles 1981).

In Tedd & Charles (1982) a laboratory study was conducted on the same Essex clay as in Tedd & Charles (1981). The results of both the in situ and laboratory studies are presented in Figure 2.6. When calculating the horizontal stress in the London clay, both an input value for an isotropic elastic material and a typical value for London clay was used. The laboratory results were within the scatter of the measurements made by the Camkometer self-boring pressuremeter for both the input values. The laboratory results support the conclusions and measurements in Tedd & Charles (1981). If a correction for sampling disturbance had been made, this would have given a bit lower values of the horizontal stresses measured in the laboratory. This would bring the laboratory results a bit further away from the values of the horizontal stress measured by the earth pressure cells.

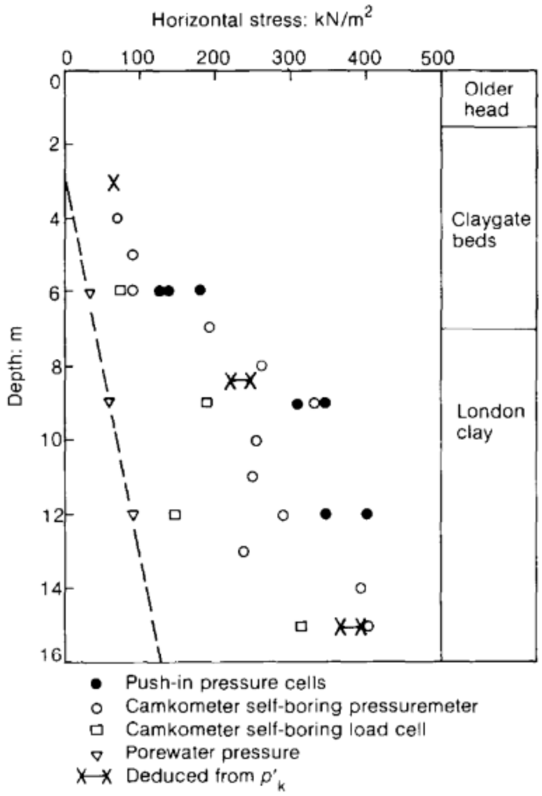


Figure 2.6: Comparison of the results from earth pressure cells, Camkometer self-boring pressuremeter, Camkometer self-boring load cell and laboratory investigations. Figure from Tedd & Charles (1982).

There have been some studies looking into the amount of over-reading in the earth pressure cell measurements. Tedd & Charles (1983) suggest that the over-read can be takes as $0.5 \cdot s_u$. Carder & Symons (1989) report that the over-read may be even larger for very stiff clays with $s_u > 150$ kPa. This conclusion was found by comparing earth pressure cell readings to the Camkometer self-boring pressuremeter and the dilatometer.

Ryley & Carder (1995) performed a study of the over-read for the earth pressure cell when the stress measured was the well-defined overburden pressure. Six cells were installed horizontally at different depths from within the Heathrow Express trial tunnel. The cells were placed so far from the tunnel that the vertical pressure measured would be the actual well-defined overburden pressure. The unit weight of the soil above the cells was determined by laboratory tests.

Ryley & Carder (1995) concluded that for firm to stiff clays with s_u in the range of 40 to 150 kPa, the best fit of the over-read was $0.8 \cdot s_u$. For design purposes $0.5 \cdot s_u$ would be a more conservative value to use, while for research purposes $0.8 \cdot s_u$ would probably be better. For very stiff clay with $s_u > 150$ kPa the best fit would give an over-read significantly higher than $0.8 \cdot s_u$. The best fit line for very stiff clay is reported to be $4 \cdot (s_u - 120\ kPa)$. The best fit lines for the over-read versus undrained shear strength are shown in Figure 2.7.

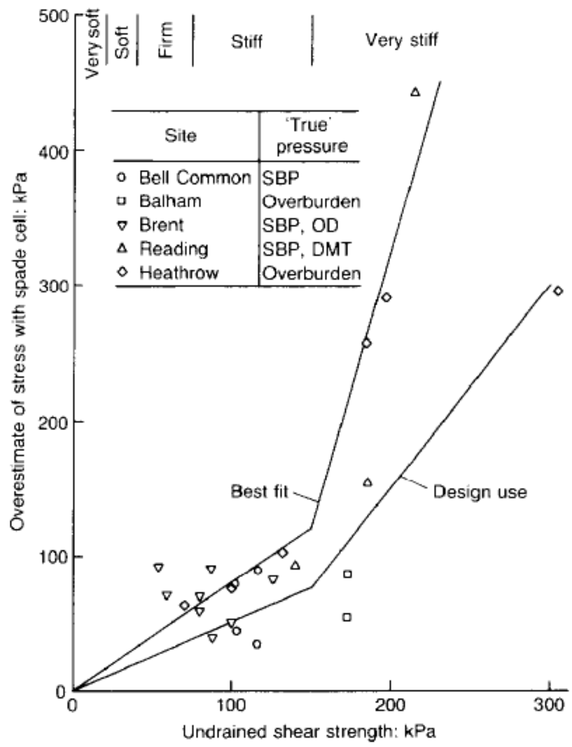


Figure 2.7: Over-read of earth pressure cell versus undrained shear strength with best-fit lines. Figure from Ryley & Carder (1995).

2.8 Shear Wave Measurements

For the methods presented so far, the amount of disturbance caused by penetrating the equipment into the material has been questioned (see for example Tedd & Charles (1981) and Fioravante et al. (1998)). Fioravante et al. (1998) found a general lack of consistency when comparing results obtained with the different intrusive field methods for measuring the horizontal stress state. Consequently, some methods not requiring penetration into the soil materials, so called non-intrusive methods have been proposed (Fioravante et al. 1998, Ku & Mayne 2013). One that has gained quite some attention is the use of shear wave velocity measurements. The method is based on seismic waves and rely on the connection between the wave velocity and the stress state of the soil (Fioravante et al. 1998).

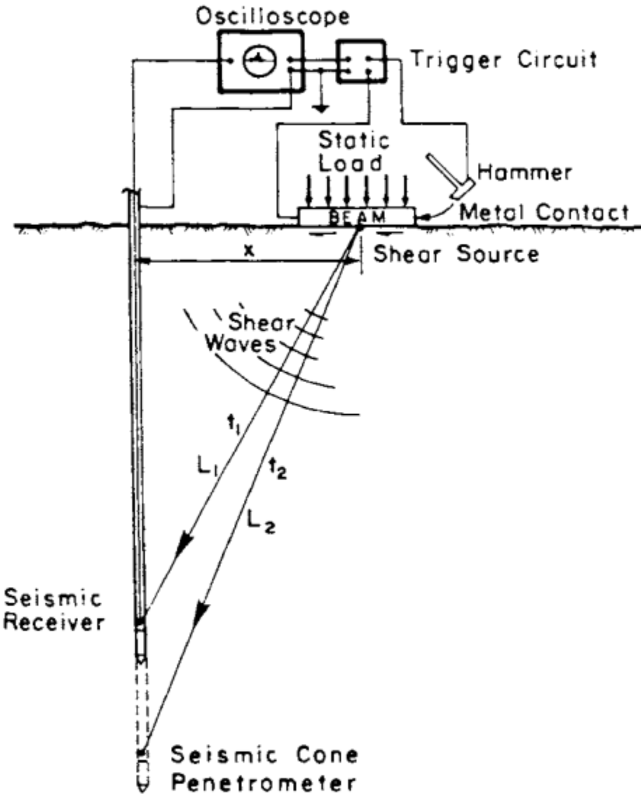


Figure 2.8: Example of equipment used for shear wave measurements. Slightly modified figure from Campanella & Stewart (1992).

Shear wave velocity measurements may typically be carried out by the use of a seismic cone penetrometer (SCPT) or seismic dilatometer (Ku & Mayne 2013). This equipment is quite similar to the original CPT and dilatometer, but is additionally fitted with one or two shear wave receivers (Sully & Campanella 1995). When the probe is at the desired depth, a hammer and anvil at the surface is used to generate the shear waves. The time necessary for the waves to reach the seismic receiver in the ground is registered by a data acquisition system which is activated when the hammer strikes the anvil. Subsequent measurements at different depths makes the generation of a shear wave profile possible (Sully & Campanella 1995). A typical view of the test equipment is shown in Figure 2.8.

Previous research has indicated that the shear wave velocity is dependant on the effective stress state in the soil (Fioravante et al. 1998). Three primary types of shear wave velocity tests exist: 1) downhole (vertical) 2) crosshole (horizontal) and 3) rotary-type (horizontal) (Ku & Mayne 2013).

Fioravante et al. (1998) proposed that the value of K_0' may be determined by comparing the shear wave velocity in the vertical and horizontal direction. By utilizing the rotary crosshole type of measurements, Ku & Mayne (2015) later proposed a method for determining the OCR of a soil deposit (Lunne & L'Heureux 2016). With the OCR known, Equation 2.20 may be utilized as a method for determining the value of K_0'.

Ku & Mayne (2013) found that the combination of downhole and crosshole shear wave velocity measurements may be used as a preliminary evaluation of K_0'. This was supported by Fioravante et al. (1998), which pointed out that the final value of K_0' is very sensitive to even small errors in the determination of the vertical and horizontal shear wave velocities. Also, the authors pointed out that the method was limited to homogeneous deposits, where reflections of the waves is a negligible problem.

2.9 Seismic and Ordinary Dilatometer

The flat dilatometer was first introduced by Marchetti in 1980 (Marchetti 1980), and since then, the equipment has become a quite favored way to investigate the in situ stress state (Ku & Mayne 2013). The dilatometer is shown in Figure 2.9a and features a circular membrane, which by the use of for instance compressed nitrogen may be inflated after penetrating the equipment to the desired depth. A more detailed view of the working principle is shown in Figure 2.9b. An audible signal is used to provide the operator with information about the inflation level of the disc (Marchetti 1980). One pressure reading is taken before the membrane is inflated, called p_0; another called p_1 is taken after the membrane is extended 1.1 mm from the base (Marchetti 1980). An additional pressure reading called p_2 may be taken when the membrane has moved back to the same position as p_0 is taken at (Marchetti, et al. 2006). For sands p_2 will be close to the equilibrium pore pressure, while for clays it will be somewhat higher (Campanella & Robertson 1991). The different states are shown in Figure 2.9a. The dilatometer is typically penetrated another 20 cm before a new reading is taken. Based on the measured pressures, three key parameters may be determined: the material index I_D, the dilatometer modulus E_D and the lateral stress index K_D. Figures for these three key parameters registered at Tiller is given in appendix D.

$$I_D = \frac{p_1 - p_0}{p_0 - u_0} \tag{2.5}$$

where u_0 is the initial pore pressure.

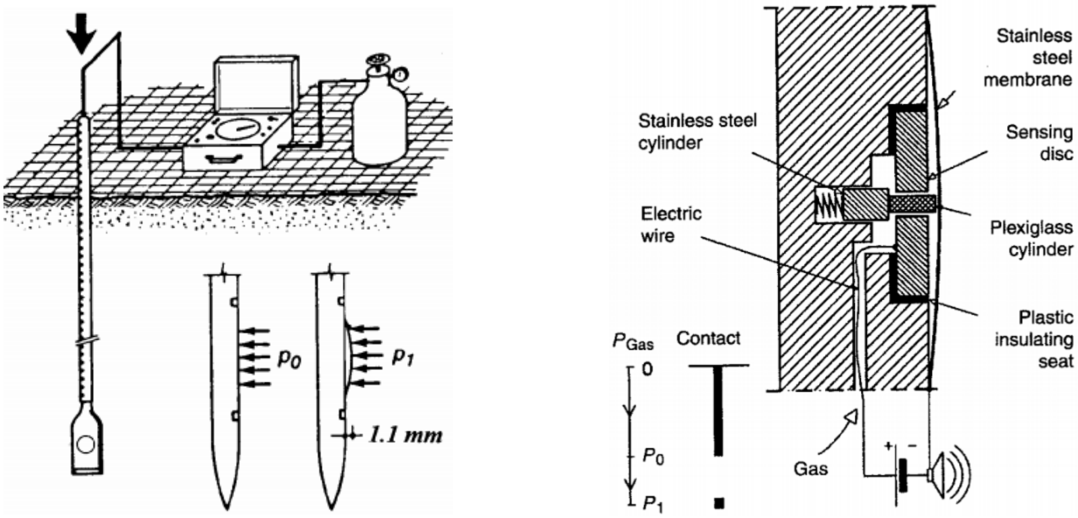
$$E_D = 34.7(p_1 - p_0) \tag{2.6}$$

$$K_D = \frac{p_0 - u_0}{\sigma'_{v0}} \tag{2.7}$$

where σ'_{v0} is the effective overburden pressure.

Based on values from Equation 2.7, the original Equation 2.8 relating the lateral stress index to the coefficient of earth pressure at rest is given below (Marchetti 1980). This equation was originally meant for sand, but may also be used to give an approximate value of K'_0 in clay (Marchetti 2015).

$$K'_0 = \left(\frac{K_D}{\beta_k}\right)^{0.47} - 0.6 \tag{2.8}$$



(a) (b)

Figure 2.9: (a) The Marchetti dilatometer with equipment. (b) A more detailed view of the working principle of the measuring membrane. Figures from Marchetti et al. (2006).

where β_k may vary between 3 and 0.9. The value of 1.5 was originally proposed by Marchetti (1980) and may be used for intact insensitive clays (Hamouche et al. 1995). Later Hamouche et al. (1995) used the value 2 for intact sensitive clays.

Furthermore, based on several field and lab investigations at different NGI sites, Lacasse & Lunne (1989) stated that the original relationship proposed by Marchetti (given in Equation 2.8) tends to overestimate the K_0' value. The authors proposed Equation 2.9 below for Norwegian clays, with m = 0.44 for highly plastic clays and m = 0.64 for low plastic clays (Lacasse & Lunne 1989).

$$K_0' = 0.34(K_D)^m \tag{2.9}$$

Since the first introduction of the dilatometer, much research has centered around finding general correlations to relate dilatometer test results with other index properties (Mayne & Martin 1998). Despite this, the major drawback of the dilatometer continues to be the usage of uncertain empirical correlations to determine material properties and stress states (Lacasse & Lunne 1989). As several investigations have found, the correlations have limited value across different sites (Ku & Mayne 2013) and may be dependant on the geological history of the deposit investigated (Roque, et al. 1988). Despite this apparent lack of consistency, Marchetti has kept his original correlation given in Equation 2.8 (Lunne & L'Heureux 2016). Also, in Mayne & Martin (1998), the self-boring pressuremeter, total stress cell or hydraulic fracturing is recommended instead of the dilatometer to determine K_0' as these methods represents a more direct approach without relaying on empirical correlations. Some authors (see for example Mayne & Martin (1998)) have also indicated that the dilatometer is best suited for usage in softer clays, as a a slight over-read is apparent when used in stiffer clays (Mayne & Martin 1998, Lunne & L'Heureux 2016).

On the other hand, several investigations have found a quite good fit when comparing values of K_0' obtained from the dilatometer with values obtained with other test equipment. Hamouche et al. (1995) found a good fit between results obtained with the dilatometer and the self-boring pressuremeter (Hamouche et al. 1995, Kulhawy & Mayne 1990).

Mayne & Martin (1998) stated that the flat dilatometer must be considered a rather exploratory tool, which should be used with site-specific calibrations. Preferably the results should be compared to results gained from for example the self-boring pressuremeter. Lutenegger (1990) proposed that the dilatometer may be used as a total stress cell after being installed at a desired depth. The pressure reading p_0 may be taken as the pressure required to overcome the horizontal stresses in the soil. One challenge of this is that as the dilatometer is substantially thicker than the earth pressure cell, more time for dissipation of excess pore pressure caused by the installation may be required (Mayne & Martin 1998).

The amount of disturbance and excess pore pressure generated have also been investigated in several papers. Roque et al. (1988) stated that insertion of a dilatometer blade causes soil disturbance in a distance of up to 7 mm from the blade it self. The authors also pointed out the generation of excess pore pressure and hence reduction in effective stresses in saturated soils (Roque et al. 1988). The effect of this stress change is unknown, as it may lead to elastic deformation, failure or something in between.

The combination of the flat dilatometer and the ability to measure shear wave velocities was first proposed by Hepton in 1988 (Marchetti, et al. 2008). The equipment resembles the flat dilatometer in shape and appearance, but the probe shaft is slightly elongated, making room for two seismic receivers with an individual distance of 500 mm. The working principle of the equipment resembles the seismic cone (SCPT) (Marchetti et al. 2008), as described in section 2.8, and makes it possible to combine dilatometer and shear wave measurements as the probe is advanced into the ground.

2.10 Stepped Blade

A concept sharing some similarities with the dilatometer, is the Iowa Stepped Blade, first introduced by Richard Handy in 1980s (Lunne & L'Heureux 2016). The tool consists of a long blade with varying thickness. Each thickness level contains a membrane, which measures the pressure, just as for the earth pressure cell (Vaslestad 1989). Based on the measurements, extrapolation is used to find the stress at an imagined zero blade thickness, see Figure 2.10. This stress is assumed to represent the in situ total horizontal earth pressure (Hamouche et al. 1995), (Handy, et al. 1990). Some articles have found that the extrapolation prohibits a clearly defined zero value, making the horizontal stress determination difficult (Lunne & L'Heureux 2016). Although it is argued by Handy et al. (1990) that the blade is suitable for use both in clay, silt and sand, others report that the blade has proven to be quite easily damaged during testing (see for example Lunne & L'Heureux (2016)).

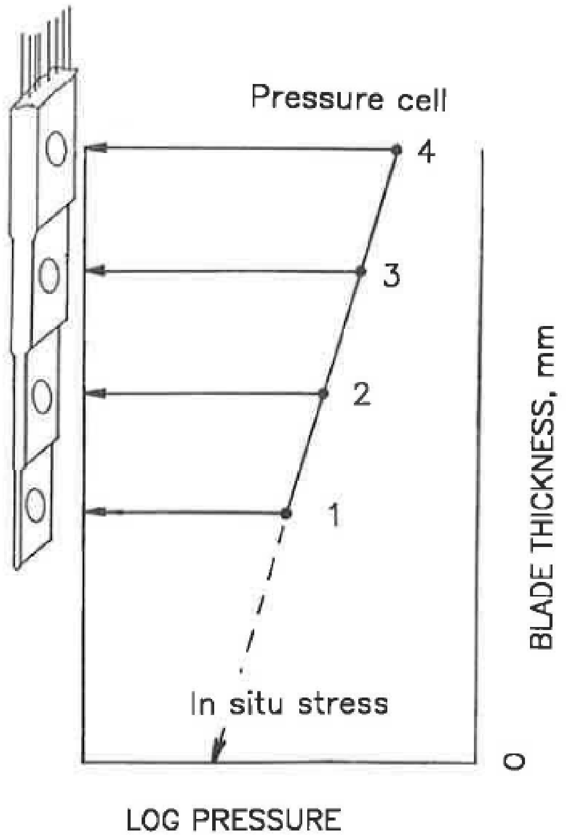


Figure 2.10: The stepped blade equipment and the extrapolation used to determine in situ horizontal stress. Figure from Handy et al. (1990).

2.11 Self-boring Pressuremeter

A different approach to the challenges of in situ stress measurements is taken by the pressuremeter, and more recently the self-boring pressuremeter. One version of the pressuremeter was developed by Menard in France in the 1950s (Vaslestad 1989, Hughes & Robertson 1985). The working principle of the pressuremeter resembles the dilatometer. A probe containing three inflatable rubber membranes located with even distance around the probe cylinder are penetrated into the ground by the use of a jack at the surface. At the desired depth, the membranes are inflated with the use of nitrogen. The expansion of the membranes is measured by three expandable strain arms (Fahey & Randolph 1984). The horizontal stress state may be taken as the pressure required for the initial expansion of the membranes (Hamouche et al. 1995, Ku & Mayne 2013).

The installation of the pressuremeter is often complicated by the soil conditions, as the equipment and membranes are quite fragile. To make use of the equipment in soils with varying stiffness or some hard layers, predrilling of a borehole is often required (Vaslestad 1989). The utilization of predrilling tends to disturb the soils to an unknown extent, much like the penetration of the dilatometer or piezometer. This makes the method less suitable in stiffer soil materials.

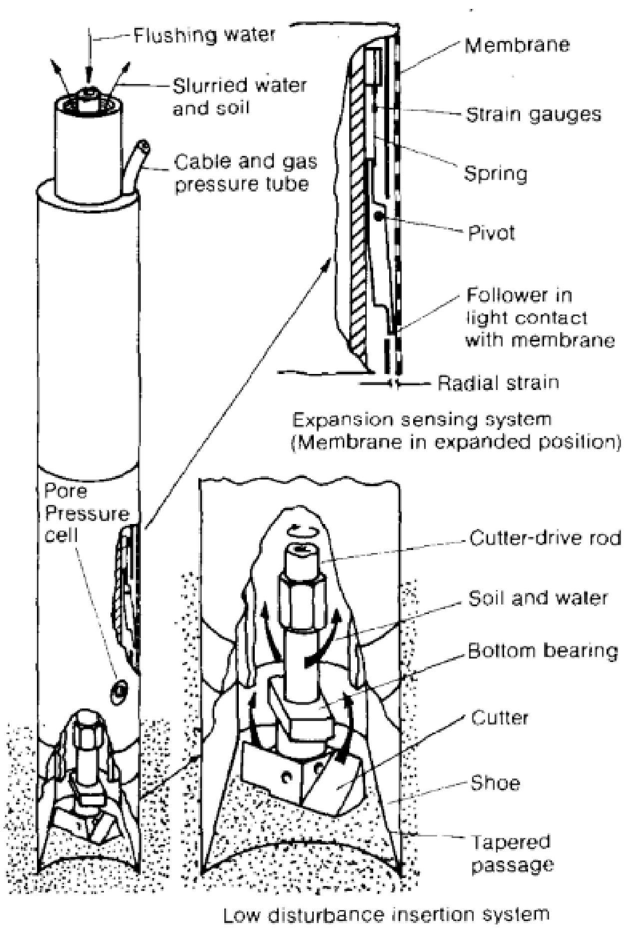


Figure 2.11: Detailed view of the Cambridge self-boring pressuremeter. Figure from Fahey & Randolph (1984).

Consequently, the self-boring pressuremeter was developed, see Figure 2.11, primarily in two different versions. One is the PAFSOR pressuremeter developed in France and the other is known as the Cambridge self-boring pressuremeter, or Camkometer for short, developed in Great Britain (Hughes & Robertson 1985). The working principle of the measuring equipment is very similar to that utilized by the standard pressuremeter. Additionally, at the far end of the probe, a cutting shoe encasing a rotating cutter bit is mounted. While a jack at the surface is used to penetrate the probe into the ground, the cutter bit removes material as the probe is lowered. The removed material is brought to the surface by the use of a water flushing system (Fahey & Randolph 1984). Additionally, the Camkometer features an ability to measure the pore pressure.

Since the first introduction, the self-boring pressuremeter has been utilized in several different investigations and materials. Hamouche et al. (1995) argue that the method represents a less intrusive way of measuring the horizontal stress state, as the self-boring ability means less soil disturbance; the soil is removed rather than being compacted by the installation of the probe. However, some authors have argued that the process of removing the soil during installation modifies the stress field around the probe, and hence causes disturbances of unknown consequence (Ghionna, et al. 1982).

Hamouche et al. (1995) used the Camkometer in combination with hydraulic fracturing and a dilatometer at different clay sites in Eastern Canada. The device was lowered at a rate of about 2 cm/min and the probe was left at the desired depth for about 15 hours before the measurements were taken (Hamouche et al. 1995). A rather large variation in the pressure required for the individual movement of the three strain arms was observed. An average value of the three registered pressures was used when calculating the horizontal stress.

Despite the amount of variation in the data material, the authors of Hamouche et al. (1995) found that for an OCR below 4, the three different methods gave quite similar results. For testing at a site with OCR above 5, the values gained from hydraulic fracturing and the self-boring pressuremeter deviated with about 20 %. This was explained by changes in fracture direction in such overconsolidated clays (Hamouche et al. 1995).

The large variation in the data acquired from self-boring pressuremeters has also been observed by others (see for example Tedd & Charles (1981)), and have by some authors been linked to natural variations of the soil material (Hamouche et al. 1995). Anderson & Pyrah (1991) investigated this by using the self-boring pressuremeter in artificially consolidated clay under isotropic conditions in the laboratory. As a difference in pressure measurements was also observed under such conditions, the authors concluded that the in situ differences could not be the cause alone. Instead they proposed the variations to be caused by individual differences in membrane stiffnesses (Anderson & Pyrah 1991). Secondly, the large variations in results have also been connected to the installation of the pressuremeter. Additionally, Hughes & Robertson (1985) states that the quality of the results is highly dependant on the operator performing the test. Despite some variation in the obtained results, the self-boring pressuremeter is often mentioned together with earth pressure cells and hydraulic fracturing as one of the more reliable ways of determining the horizontal stress state (Mayne & Martin 1998, Ku & Mayne 2015, Ku & Mayne 2013).

2.12 Field Vane

Combining a field vane test with a triaxial CAUc test, the value of K_0' for a clay may be found (Aas, et al. 1986). This method is only valid if the material can be assumed to be undrained during the field vane test. In the triaxial cell the sample is consolidated to the assumed in situ stress situation. From the shearing phase, the effective stress path is plotted in a $\frac{\sigma_v'-\sigma_h'}{2\sigma_{v0}'}$ versus $\frac{\sigma_v'+\sigma_h'}{2\sigma_{v0}'}$ plot, as shown in Figure 2.12. Through a graphical construction, as illustrated in the figure, the value of K_0' may be determined with the undisturbed and remoulded shear strength from the field vane test as input parameters. Aas et al. (1986) compared field vane tests with hydraulic fracturing, dilatometer and self-boring pressuremeter tests for clay under the weathered zone at Onsøy and Haga, and found a very good fit between the different methods for determining K_0'.

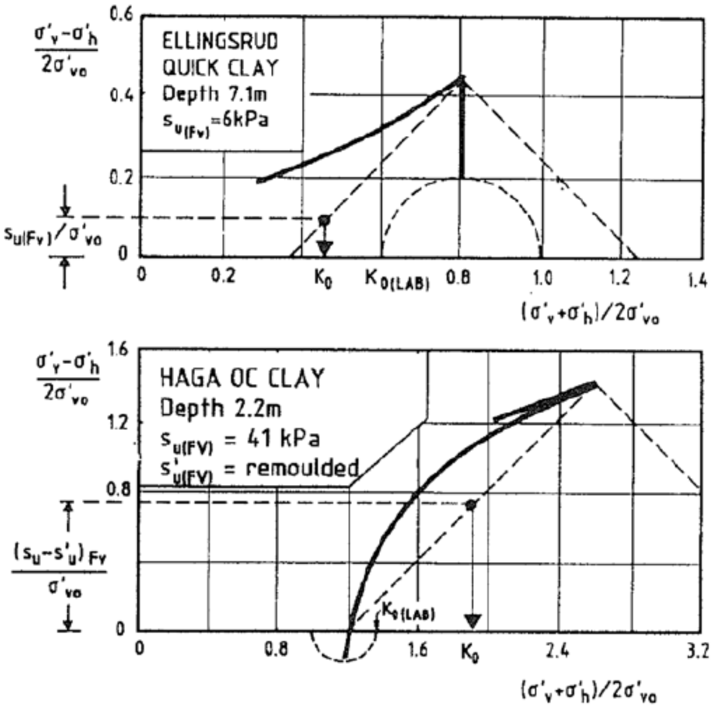


Figure 2.12: Graphical construction to determine K_0' from field vane test. Figure from Aas et al. (1986).

2.13 Lateral Stress Cone

Several attempts were made in the 1980s to correlate lateral stress measured on a friction sleeve of a CPT probe to K_0' (Lunne, et al. 1990). None of the attempts were successful enough to be in use today (Lunne & L'Heureux 2016). Sully & Campanella (1991) investigated the possibility of correlating K_0' to the measured change in pore pressure from the tip u_1 to behind the tip u_2, normalized by the initial effective overburden stress σ_{v0}'. They concluded that for a specific site there seems to be a linear relationship between K_0' and $\frac{u_1-u_2}{\sigma_{v0}'}$. However, the scatter in the measurements between different sites was too large to conclude with a universally valid linear relationship (Sully & Campanella 1991).

In 2014 researchers at University of British Columbia presented a lateral stress seismic piezocone. A schematic overview of the equipment is shown in Figure 2.13. The equipment features a "button" sensor for measurement of lateral stress. Although the researchers believe that the method may give reliable values of the lateral stress, a lot of testing is required before it can be concluded whether or not the measured lateral stress can be correlated to K_0' in a satisfactory way (Lunne & L'Heureux 2016).

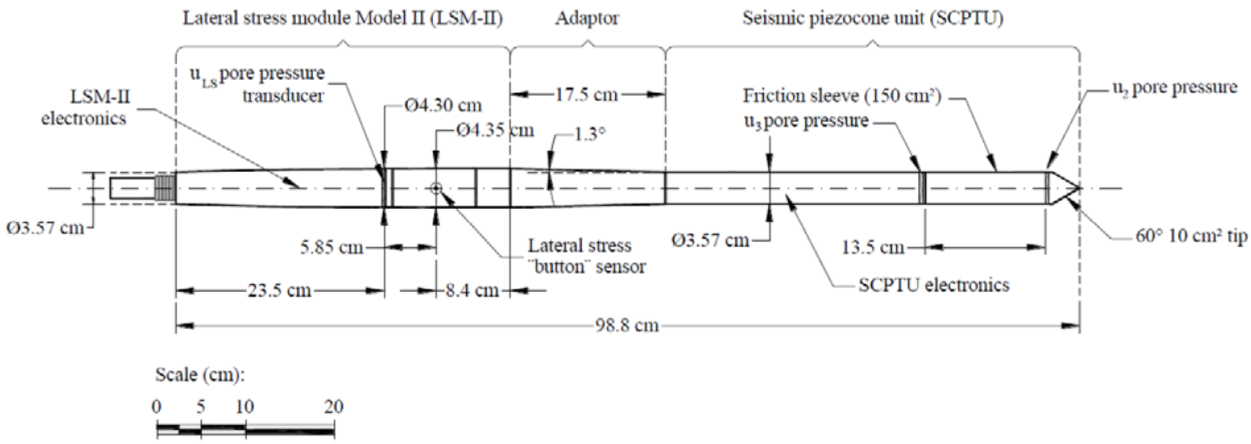


Figure 2.13: Schematic overview of the lateral stress seismic piezocone. Figure from Lunne & L'Heureux (2016).

2.14 Laboratory Methods

There are many different approaches used to determine K_0' through laboratory tests (Ku & Mayne 2015). Among these are the split-ring oedometer and the triaxial test with a K_0' procedure. Two additional methods will also be presented.

To be able to use laboratory methods to predict the in situ K_0' value, the stress state of the samples should be as undisturbed as possible (Ku & Mayne 2015). This often calls for both time consuming and expensive sampling equipment (Fioravante et al. 1998, Ku & Mayne 2015), like the block sampler, which is known to reduce the influence of sample distribution (Watabe et al. 2003, Karlsrud & Hernandez-Martinez 2013). In addition, the samples are vulnerable to disturbance during transport between the site and the laboratory (Ku & Mayne 2015).

The split-ring oedometer test is a modified oedometer test developed at NTH (now NTNU) during the 1980's (Senneset 1989). The sample is cut and placed in the apparatus as for an ordinary oedometer test, but the ring ensuring zero lateral strain in an ordinary oedometer test is replaces by three ring segments. The ring segments feature steel membranes which will ensure a closed ring around the sample. The steel membranes are equipped with high precision strain gauges. When the oedometer test is carried out, the strain in the steel membranes can be correlated to the horizontal total stress in the sample. The vertical total stress is known from a load cell on the load piston. If the test is run as a CRS experiment with constant rate of strain, the pore pressure at the bottom of the sample is measured in order to calculate the effective stresses from the measured total stresses. From the effective horizontal and vertical stresses the value of K_0' can be calculated.

The triaxial test apparatus can be run with a K_0' consolidation procedure (Watabe et al. 2003). The principle is that during the consolidation phase, the cross-sectional area is kept constant like in an oedometer by increasing the cell pressure. The cell pressure is adjusted so that the amount of pore water expelled is equal to the volume change caused only by the vertical strain. Since the consolidation phase is drained, the cell pressure is equal to the horizontal effective stress in the sample and the piston force equals the vertical effective stress. From the measured values of piston force and cell pressure the value of K_0' can be calculated.

By utilizing this procedure, the authors of Watabe et al. (2003) investigated the K_0' values of marine clays from several different locations around the world. The results were compared to values based on a K_0-OCR-relationship and from dilatometer tests. Among several concluding remarks, they acknowledged the usefulness of the famous Jaky equation (given in Equation 2.11 below), and found quite comparable values when comparing laboratory tests on overconsolidated samples and results from the dilatometer tests performed in the field (Watabe et al. 2003).

Tavenas, et al. (1975) suggest that K_0' can be calculated as the ratio of the preconsolidation stresses in the horizontal and the vertical direction as in Equation 2.10. The preconsolidation stresses can be found from oedometer tests performed on samples trimmed so that the longitudinal axis correspond to either the vertical or the horizontal direction in situ.

$$K_0' = \frac{\sigma'_{ph}}{\sigma'_{pv}} \tag{2.10}$$

Hamouche et al. (1995) state that the method proposed by Tavenas et al. (1975) is wrong. In situ measurements show values of K_0' increasing with increasing OCR, whereas $\frac{\sigma'_{ph}}{\sigma'_{pv}}$ seem to be approximately the same for all OCR. This suggests that there is no significant correlation between K_0' and $\frac{\sigma'_{ph}}{\sigma'_{pv}}$, contradicting the original proposal by Tavenas et. al.

2.15 Correlation Methods

As in situ and laboratory testing is undoubtedly both difficult and costly, the horizontal stress state in soil deposits is frequently approximated using theoretical or empirical equations alone (Ku & Mayne 2015). Several formulas of varying complexity have been proposed for calculating K_0'. Some of the formulas are used for normally consolidated soils, while others account for the effect of the stress history.

2.15.1 Normally Consolidated Soils

The simplest formula was originally proposed by Jaky in 1944 (Jaky 1944), and simplified in 1948 (Jaky 1948). Equation 2.11 states that K_{0nc}' for a normally consolidated soil depends only on the effective friction angle ϕ'.

$$K_{0nc}' = 1 - sin\phi' \tag{2.11}$$

The validity of Equation 2.11 has been investigated both in laboratory and in situ. For laboratory conditions the work by Mayne & Kulhawy (1982) and Diaz-Rodriguez, et al. (1992) validated the equation. Based on in situ measurements at Berthierville, Hamouche et al. (1995) suggest that the equation is also a good approximation for K_0' when the clay has an OCR close to one. The equation has gained great popularity and is widely used for calculating K_{0nc}', for instance in the finite element program Plaxis.

However, there are studies suggesting that modified versions of Equation 2.11 gave better fit to their results. Brooker & Ireland (1965) suggest that Equation 2.12 gave a better fit to their results for cohesive soils, but the scatter is quite significant and the amount of samples limited.

$$K'_{0nc} = 0.95 - sin\phi' \tag{2.12}$$

$$K'_{0nc} = \frac{1 - sin(\phi' - 11.5)}{1 + sin(\phi' - 11.5)} \tag{2.13}$$

$$K'_{0nc} = \frac{\sqrt{2} - sin\phi'}{\sqrt{2} + sin\phi'} \tag{2.14}$$

Furthermore, Bolton (1991) proposed Equation 2.13 and Simpson (1992) suggested Equation 2.14. The difference between the estimates of K'_{0nc} are relatively small with all the above equations (Sivakumar et al. 2002). Mayne & Kulhawy (1982) report that with only small changes made to the original Jaky's equation it would fit well with their 121 samples of clays and sands. The modified equation from Mayne & Kulhawy (1982) is Equation 2.15.

$$K'_{0nc} = 1 - 1.003sin\phi' \tag{2.15}$$

Mayne & Kulhawy (1982) report that many attempts have been made to correlate K'_{0nc} with index properties such as plasticity index, liquid limit, void ratio, clay fraction and others.

Larsson (1977) report that for Scandinavian inorganic clays many tests have shown a relationship between the plasticity index I_P and K'_{0nc}, alternatively between the liquid limit w_L and K'_{0nc}. The correlations are given in equations 2.16 and 2.17. An apparent relationship between I_P and K'_{0nc} was also suggested by Brooker & Ireland (1965).

$$K'_{0nc} = 0.31 + 0.71(w_L - 0.2) \tag{2.16}$$

$$K'_{0nc} = 0.315 + 0.77 I_P \tag{2.17}$$

However, the test results from more than 170 samples of clays and sands in Mayne & Kulhawy (1982) support none of the correlation between index properties and K'_{0nc}. Also Mayne & Kulhawy (1982) indicate that for 130 clay samples the scatter was too large to find any useful correlations.

2.15.2 Overconsolidated Soils

Brooker & Ireland (1965) state that the value of K'_0 depends heavily on the stress history of the soil. This is supported by many studies, including Sivakumar et al. (2002) and Mayne & Kulhawy (1982). In their famous article, Brooker & Ireland (1965) investigated the relationship between the earth pressure at rest and stress history. By doing high-pressure one-dimensional compression tests on five cohesive soils, the authors found that the stress history of a soil deposit is the primary factor influencing the coefficient of earth pressure at rest. As OCR increases, the value of K'_{0oc} should theoretically approach the coefficient of passive earth pressure, K_P (Brooker & Ireland 1965).

Schmidt (1966) proposed Equation 2.18 taking into account the effect of overconsolidation for soils experiencing first unloading.

$$K'_{0oc} = K'_{0nc} OCR^{\alpha} \tag{2.18}$$

where α is the slope of the curve when log K'_{0oc} is plotted against log OCR. In order to utilize this relationship, laboratory investigations to determine the preconsolidation pressure is necessary (Fioravante et al. 1998).

Mayne & Kulhawy (1982) uses the relationship between K'_{0oc} and OCR for more than 170 samples of clays and sands in order to state that

$$\alpha = sin\phi' \tag{2.19}$$

Combining Equation 2.19 with Equations 2.11 and 2.18 gives Equation 2.20

$$K'_{0oc} = (1 - sin\phi')OCR^{sin\phi'} \tag{2.20}$$

The results reported by Hamouche et al. (1995) suggest that Equation 2.19 does not fit well with their findings on sensitive clays in Eastern Canada. Based on in situ tests using the dilatometer, hydraulic fracturing and the Cambridge self-boring pressuremeter, higher values than those predicted by Equation 2.20 were found. The values of K'_{0oc} found from the three in situ methods were quite similar, and normally they corresponded to values of α in the range of 0.75 to 1.15. In Hamouche et al. (1995), also results from a number of other authors are presented. The results suggest that the value of α can vary between $sin\phi'$ and more than one. They also suggest that the value of α tend to increase with increased sensitivity.

Moreover, the authors of Sivakumar et al. (2009) made an evaluation of the existing theoretical approaches to estimating the coefficient of earth pressure for overconsolidated clays. In agreement with Sivakumar et al. (2002) they stated that the different estimates of K'_{0oc} which are primarily based on the angle of internal friction and OCR show rather large variations (Kulhawy & Mayne 1990).

Sivakumar et al. (2002) takes on a thorough theoretical approach based on the modified Cam-Clay model in order to find the relation between K'_{0oc} and OCR given in Equation 2.21, taking into account anisotropic in situ stresses.

$$OCR = \frac{\sigma'_{pv}}{\sigma'_{v0}} = \left[\frac{1 - \chi K'_{0oc}}{1 - \chi K'_{0nc}}\right]^{\frac{1}{\chi}} \tag{2.21}$$

χ is the ratio between the slopes of the unloading-reloading curves used in the modified Cam-Clay model for 1D and isotropic loading conditions (Sivakumar et al. 2002). K'_{0nc} can be found for instance from Equation 2.11 (Sivakumar et al. 2002). Experiments reported in Sivakumar et al. (2009) confirm that the relation in Equation 2.21 manages to take into account the effect of anisotropic in situ stresses.

Sivakumar et al. (2009) try to relate K'_{0oc} not only to the stress history of the clay, but also to the structure of the material. The background for the proposed Equation 2.22 is the modified Cam-Clay model, as for Equation 2.21.

$$K_{0oc} = \frac{1}{\eta}[1 - (1 - \eta K_{0nc})OCR^{(1-\chi)}] \tag{2.22}$$

η describes the anisotropical elastic stiffness behaviour of the clay within the yield locus. If $\eta = -2$ the material is assumed to be isotropically elastic, and a higher value of η means that the anisotropy is increasing. χ is as for Equation 2.21. Theoretically the ratio χ should be equal to 1 for linearly elastic materials, while it is found through laboratory experiments to typically be around 0.8 (Sivakumar et al. 2009). K'_{0nc} can be found for instance from Equation 2.11 (Sivakumar et al. 2009).

The challenge with using equations 2.21 and 2.22 for predicting K'_{0oc}

is the need for several laboratory tests (Sivakumar et al. 2002, Sivakumar et al. 2009).

Hamouche et al. (1995) conclude that it is hard to determine K'_{0oc} from correlations, and therefore stresses the need for in situ measurements if an accurate value of K'_{0oc} is necessary. This is especially because the value of α in Equation 2.14 tends to vary substantially, and showing increasing values with increasing sensitivity (Hamouche et al. 1995). This is also supported by Ku & Mayne (2015), stating that the most accurate values of K'_0 are found from direct in situ measurements. Even though the correlation methods are increasing in complexity and ability to account for different soil behaviour effects, it seems like the conclusion in Hamouche et al. (1995) is still valid.

2.16 Summary

The purpose of this chapter was to give a summary of the relevant theory available on K'_0 and presenting a collection of different field, laboratory and correlation methods. A selection of relevant articles and their findings have also been introduced. Some of these findings will be further discussed in Chapter 4.

Chapter 3

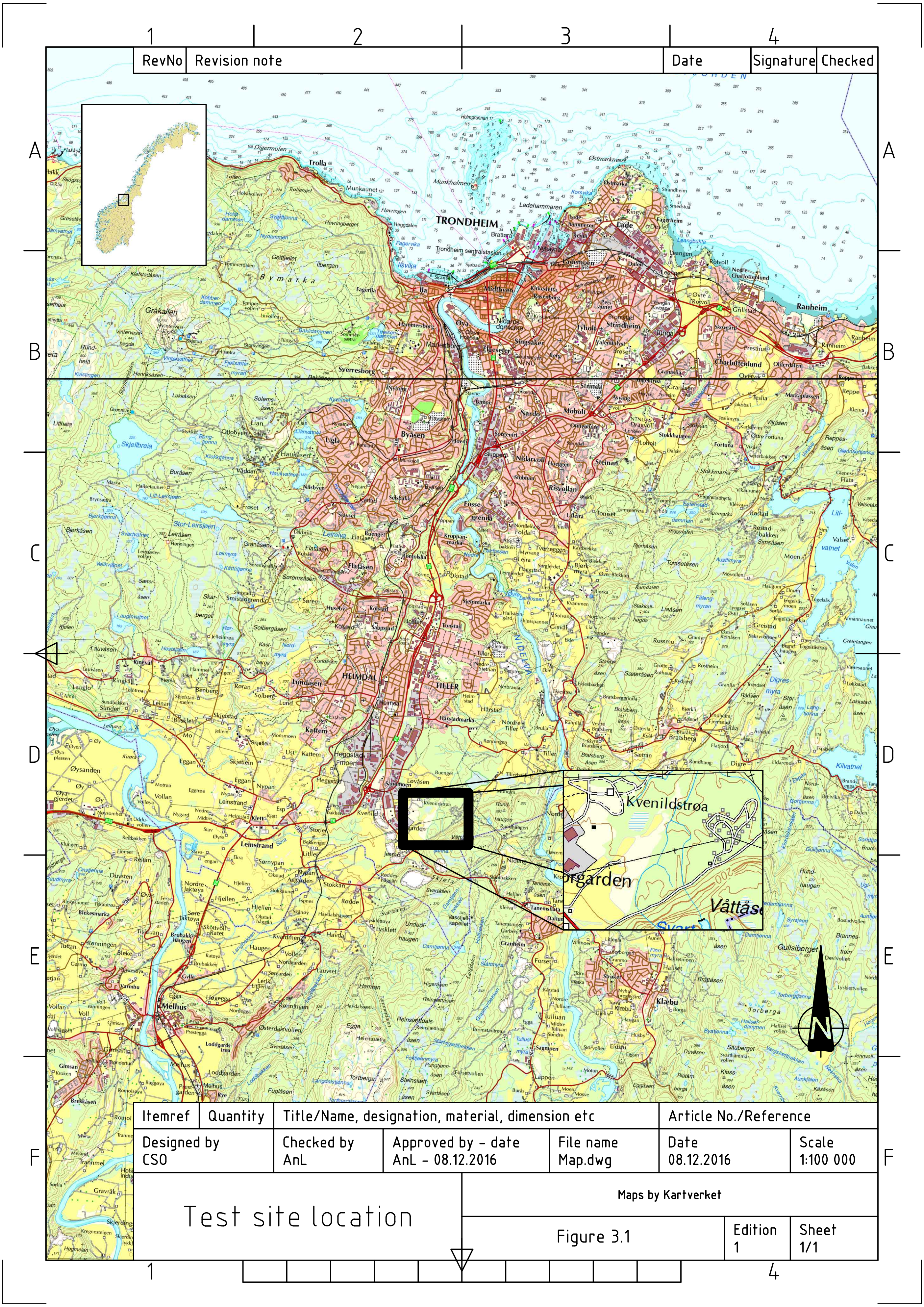
Pilot Experiments

Pilot experiments were conducted at Tiller using a Marchetti flat dilatomter and two Glötzl earth pressure cells featuring pore pressure measurements. The testing was conducted in November 2016, by technical operators from NGI. The objectives of the pilot experiments were to get familiar with the equipment and check if reasonable results could be obtained.

This chapter starts with a brief presentation of the test site before more details are given about the procedures used when testing the equipment. An evaluation of the efficiency and user friendliness of the equipment will be given before the chapter ends with a presentation of the acquired results and subsequent data analysis.

3.1 Site Description

The experiments were performed at the NGTS test site at Tiller, close to Trondheim. The test site location is shown in Figure 3.1.


RevNo
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Article No./Reference
Designed by CSO
Checked by AnL
Approved by - date AnL - 08.12.2016
File name Map.dwg
Date 08.12.2016
Scale 1:100 000
Maps by Kartverket
Test site location
Figure 3.1
Edition 1
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The NGTS site is located in close proximity to a test site which has been used by NTNU (formerly NTH) since at least the beginning of the 1980's (Gylland et al. 2013). At the NTNU test site, extensive work has been made to characterize the properties of the soil materials, mainly consisting of clay. A summary of the material properties are given in Gylland et al. (2013), along with a list of the main research projects performed at the Tiller site until 2012.

The investigations at Tiller indicate that below a few meters of dry crust and peat, quick clay is present from a depth below around 5 m. In order to utilize some of the theoretical correlations presented in section 2.15, some key material properties for the Tiller clay are required. Based on recent site investigations conducted by the Norwegian Public Roads Administration in September 2016 very close to the installation location of the dilatometer and earth pressure cell, Table 3.1 below is compiled. In the table, values for a depth of about 5 m are presented, as this is the installation depth of the earth pressure cell. Further details regarding the laboratory test results are given in appendix F. It should be noted that some of the parameters are based on a limited amount of tests and hence should be considered preliminary. Similarly, it should be noted that below a depth of 5 m, the properties of the clay changes quite substantially, as the sensitivity increases and the remoulded shear strength goes towards zero. The data in Table 3.1 fits reasonably well with the material parameters from the NTNU site, as presented in Gylland et al. (2013).

Table 3.1: A summary of key material properties for the clay at the Tiller test site.

Strength parameters	
Attraction, a	7 kPa
Friction angle, ϕ	32°
Cohesion, c	4.4 kPa
Undrained shear strength, s_u	30 kPa
Index properties	
Average unit weight, γ	18.5 kN/m^3
Water content, w	38 %
Plasticity index, I_p	17 %
Liquidity index, I_L	1.1
Overconsolidation ratio, OCR	1.6

3.2 Equipment

For the dilatometer test, a Marchetti seismic dilatometer was used. This equipment features both the standard dilatometer and two shear wave receivers located 50 cm apart on the same probe (see section 2.9). Together with the probe, gas hoses, a tank of nitrogen and a mobile monitoring and data acquisition system was used. For the shear wave velocity measurements conducted, a hammer and an anvil supplied by Marchetti DMT pressed to the ground surface by the drill rig was used.

Glötzl earth pressure cells with model number EPE/P AI 7/14 K5 were used. These cells feature both earth pressure and pore pressure measurements incorporated in the same cell, using two separate vibrating wires for the readings. A total of four cables are connected to the cell and makes up the two circuits that are used for measuring the earth pressure and pore pressure. By the use of a handheld measuring device, measurements in milliampere were made. These readings were later converted to pressure values by the use of calibration sheets provided together with the cell.

3.3 Method

The dilatometer test was performed at the Tiller test site in November by operators from NGI using a Geotech drill rig and ϕ44 mm drill rods. The dilatometer was penetrated into the ground without predrilling and the penetration was halted and both a p_0- and a p_1-reading was made every 20th cm (see section 2.9 for more details). Additionally, downhole shear wave velocity measurements was also conducted for every 50th cm (see section 2.8 for more details). The total penetration depth was 25 meters and the acquired raw data are presented in appendix D.

The Glötzl earth pressure cell was installed on the 15th of November by operators from NGI using a Geotech drill rig and ϕ44 mm drill rods at almost the same location as the dilatometer test was conducted. Before installation, zero readings at the surface were taken both for the earth pressure and pore pressure system, after preparing the earth pressure cell as described in Appendix A. For the initial attempt, a borehole predrilled to a depth of 4 m the day before was used. While penetrating the earth pressure cell, the measured values at the measuring device were continuously monitored. Upon closing in on the final depth of 5 m the values were quite close to the upper limit of 20 mA for the equipment, before a sudden drop to almost zero was witnessed. Despite continued penetration, the measurements did not show any changes, and the cell was then retracted to the surface. Upon visual inspection it became clear that the cell was bent to an approximate angle of 30°. This was thought to be a result of the lack of predrilling to the desired installation depth and the quite thin and fragile design of the cell.

After predrilling close to the desired installation depth was achieved by using an auger, a new cell was installed. The predrilling depth was 4.70 m, while the final depth of the middle of the cell was 5.00 m. This is approximately in accordance with the installation description in Massarsch (1975). Readings of both the earth pressure and the pore pressure circuits were taken frequently in the minutes after installation. Readings were also taken later the same day and a couple of times during the subsequent week.

3.4 Efficiency

There is a significant difference in the efficiency of the dilatometer and the earth pressure cell. The dilatometer equipment takes continuously readings at 20 cm spacing over the entire depth of the borehole. With one person operating the dilatometer equipment and one person operating the drill rig, it is possible to take readings from several boreholes in one day. The dilatometer is less fragile than the earth pressure cell, so extra time for predrilling will only be required for especially hard layers.

On the contrary, the earth pressure cell requires predrilling almost to the final depth of installation. Both this preparation and the installation of the cell requires only one person. With a manual measuring device as used in this project, some time is spent on taking a lot of readings just after the installation. This is particularly interesting when the test is run as a pilot experiment, since the first development of pressures with time can be investigated. More readings should be taken during the next 24 hours and the succeeding week or even month. Consequently, one operator must visit the site several times following installation. For commercial use, the steady-state pressure values are the most interesting, and therefore fewer readings are required compared to a research project. The procedure of taking readings is quite simple and may be conducted by almost anyone. After completing the entire process, one value of K_0' at one depth is achieved.

The efficiency of the dilatometer in terms of values obtained versus time spent is vast compared to the earth pressure cell. On the other hand, it is possible for one person to install and take readings of the earth pressure cell, while the dilatometer equipment requires two operators.

In terms of equipment costs, the dilatometer is assumed to be much more expensive than the earth pressure cell. One earth pressure cell of the same model as used in this pilot experiment costs about NOK 13 000. In addition, cables and a reading unit must be purchased. The cost of a Marchetti dilatometer, cables and reading unit is not listed on the Marchetti web page, but it may be assumed to be much higher than the cost of an earth pressure cell. On the other hand, some earth pressure cells are not made to retrieve from the ground after readings are finished, and thereby the cost of using earth pressure cells will be higher if readings are made at many locations.

3.5 Difficulties Related to Field Work and Data Interpretation

The initial attempt of installing an earth pressure cell was unsuccessful, and proved the rather fragile nature of the earth pressure cell. A picture illustrating the damages to the cell is given in Figure 3.2. The problem is believed to be that the earth pressure cell was gradually bending as it was pushed through 3 meters of clay, even though the clay is relatively soft. Based on these experiences, the need for predrilling to almost the desired depth became evident.

Figure 3.2: Damaged earth pressure cell after first trial installation.

Computer software developed by Marchetti DMT was used for processing of the dilatometer measurements taken at the Tiller site. In the initial processing where the automated procedure of the software was used, the soil material was incorrectly interpreted. The software interpreted the soil all the way down to 25 m depth as mostly mud with some instances of peat, with an average unit weight of about 13-14 kN/m^3. Laboratory samples taken close to the borehole used for the dilatometer test show an average unit weight of about 18.5 kN/m^3 below the uppermost meters of soil (see Appendix F). This mismatch indicates that the software is not calibrated for usage in very sensitive clays. In later processing of the material, the effective overburden pressure was corrected as described in the next section.

3.6 Analysis and Results

The acquired field data material from the dilatometer test was initially processed by Zeynep Ozkul at NGI, using the Marchetti DMT software. The software automatically calculates the three key parameters I_D, E_D and K_D (see section 2.9). Several other material properties were also calculated by using empirical correlations. Later, further adjustments of the unit weight and calculation of the overburden pressure was conducted. In the adjusted interpretation, a dry crust with $\gamma = 16\ kN/m^3$ was assumed down to a depth of 1 m. Below, an unit weight of $\gamma = 18.5\ kN/m^3$ was assumed. A complete view of the data material is given in Appendix D.

From Figure D.1 it is clear that the values of both I_D and E_D are generally very low. The possibility of this happening is also pointed out in Marchetti et al. (2006). Here, the use of p_2-readings are recommended as an aid for this case. The very low values of I_D and E_D may explain why the software was unable to give a good estimate of the soil properties.

The registered values from the earth pressure cell were processed in Excel, where the measured values were converted to pressure measurements as described in Appendix B by the use of the calibration sheets given in Appendix C. The same assumptions as for the dilatometer results regarding the overburden pressure were used for the earth pressure cell. The earth pressure readings were not corrected for over-read, since the corrections suggested by Ryley & Carder (1995) are not valid for clays with $s_u < 40\ kPa$. A complete view of the data material for the earth pressure cell is given in Appendix E. In the appendix, Figure E.1 shows the development of the measured pressures with time. It may be seen that the pressures have reached an equilibrium situation. The reason why the readings of pore pressure and earth pressure are not made simultaneously is that the same measuring device had to be used for both pressures, one pressure at a time.

3.7 Comparison of Earth Pressure Cell, Dilatometer and Theoretical Solutions

Table 3.2 gives a comprehensive presentation of the different theoretical correlations used, as well as the input parameters in each correlation. References to the presentation of each equation in Chapter 2 is also made. Furthermore, Figure 3.3 gives a comparison of K_0' values calculated from the earth pressure cell results, the dilatometer as well as a selection of theoretical correlations. The figure is further discussed in Chapter 4.

It should be noted that as the three equations 2.13, 2.14 and 2.15 give essentially the same value of K_0' as the original Jaky's Equation 2.11 (see Table 3.2). This is to be expected based on Sivakumar et al. (2002). These values are therefore not included in the figure. Also, the original equation for calculating K_0' from K_D proposed by Marchetti (given in Equation 2.8) is represented by a solid line. A value of $\beta_k = 2$ is chosen, as this gives a line more in accordance with the other results than $\beta_k = 1.5$. The value $\beta_k = 2$ was proposed by Hamouche et al. (1995) for sensitive clays, as is the case at Tiller. Additionally, a dashed line illustrates the calculated values based on the Equation 2.9 proposed by Lacasse & Lunne (1989) for Norwegian clays. Here, m = 0.44 for plastic clays is chosen. With $I_p = 17$ %, the Tiller clay is medium to high plastic.

Equation 2.22 is included in Figure 3.3 to show that it gives a value not unlike the other correlation methods. The input values are taken from Sivakumar et al. (2009), and these are typical values based on four different clay types. Therefore, the input values are not necessarily true for Tiller clay and the result should be viewed only as an example of which value to expect from Equation 2.22.

It should also be noted that due to an error in the measurement procedure, the values of p_0 and p_1 was not obtained correctly for the depth 5.0 m. Average values of the pressures for depth 4.8 and 5.2 m were consequently used for the value at 5.0 m depth. As measurement were taken as frequent as every 20 cm, the error introduced by the interpolation is assumed to be small.

Table 3.2: A summary of equations, input parameters and results from the correlation methods.

Equation	Formula	Input parameters		Value
Jaky original 2.11	$K'_{0nc} = 1 - sin\phi'$	ϕ=32		0.47
Jaky cohesion 2.12	$K'_{0nc} = 0.95 - sin\phi'$	ϕ=32		0.42
Bolton 2.13	$K'_{0nc} = \frac{1-sin(\phi'-11.5)}{1+sin(\phi'-11.5)}$	ϕ=32		0.48
Simpson 2.14	$K'_{0nc} = \frac{\sqrt{2}-sin\phi'}{\sqrt{2}+sin\phi'}$	ϕ=32		0.46
Mayne 2.15	$K'_{0nc} = 1 - 1.003sin\phi'$	ϕ=32		0.47
Larsson for w_l 2.16	$K'_{0nc} = 0.31 + 0.71(w_L - 0.2)$	w_l = 38 %		0.44
Larsson for I_p 2.17	$K'_{0nc} = 0.315 + 0.77I_P$	I_p=17 %		0.45
Modified Jaky 2.20	$K'_{0oc} = (1 - sin\phi')OCR^{sin\phi'}$	ϕ=32	OCR=1.6	0.60
Sivakumar 2.22	$K'_{0oc} = \frac{1}{\eta}[1 - (1 - \eta K'_{0nc})OCR^{(1-\chi)}]$	χ = 0.8	η = -2	0.59

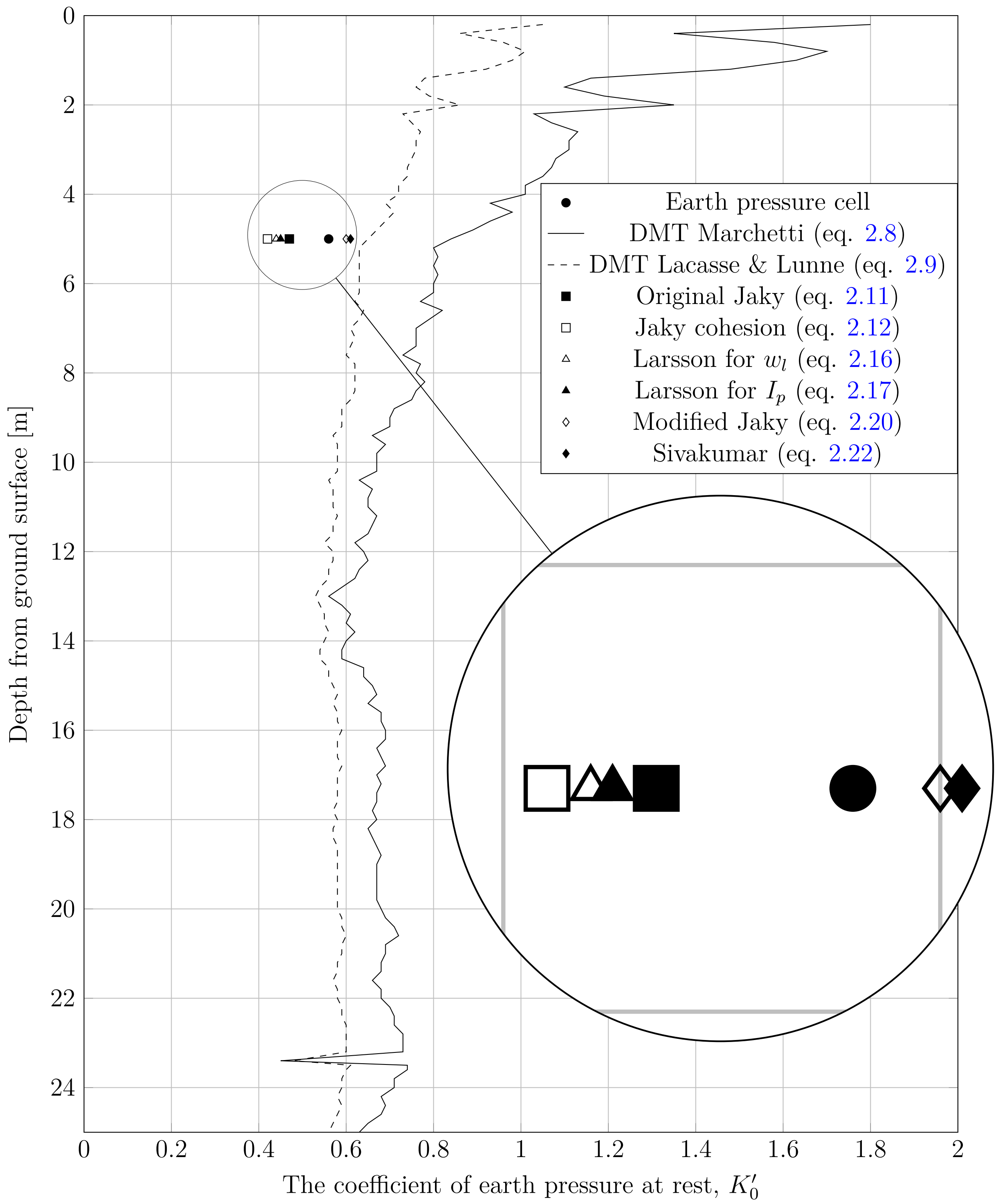

Figure 3.3: Comparison of different approaches to K_0'.

3.8 Summary

After a brief presentation of the Tiller test site, this chapter focused primarily on giving a quite thorough presentation of the equipment and procedures utilized during the pilot experiments. Subsequently, after presenting the data processing, a comparison of the measured and calculated values of K_0' was presented.

Chapter 4

Discussion

In this chapter, some of the key findings of the literature study will be discussed, although much of the evaluation of methods has already been made in Chapter 2. Additionally, some important experiences from the pilot experiments will be assessed.

4.1 Theory and Background

When looking through some of the literature on the determination of the horizontal stress state, the three methods of earth pressure cell, self-boring pressuremeter and hydraulic fracturing are mentioned as reference methods, yielding relatively repeatable values in different soil materials (Ku & Mayne 2013). Typically, if investigations are conducted in the laboratory or new theoretical approaches are proposed, the values are compared to one or more of these reference methods (see for instance Massarsch (1975), Tedd & Charles (1981) or Hamouche et al. (1995)). Indeed, these methods all represent a more direct approach to the determination of K_0', compared to for instance the dilatometer which rely on empirical correlation methods (Lacasse & Lunne 1989).

Of the three methods mentioned, the method of hydraulic fracturing is often presented as a both straightforward and versatile approach to field measurements of the horizontal stress in soils. The measured water pressures may be taken as the horizontal total stresses without any further processing. Much of the equipment required is already part of any geotechnical field investigation and the method may in many cases be utilized without any need for predrilling. However, the method requires some time, as excess pore pressure generated during installation must be allowed to dissipate before the measurements are made. Additionally, even though several articles have found good repeatability of the method, some have pointed out that the results tend to deviate from other methods, when used in soil materials with high OCR (see for instance Hamouche et al. (1995) and Lefebvre et al. (1991)).

Just as for the hydraulic fracturing, the earth pressure cell is also regarded as rather straightforward and simple. The measurements taken in the field may be corrected for possible over-read, which increase in value for increasing stiffness of clay (Ryley & Carder 1995). Even though the cell may be installed without the use of a predrilled borehole at shallow depths in soft material, the experiments at Tiller indicate that the quite thin cell is rather fragile and may require predrilling. Analogous to the hydraulic fracturing method, some time is required after installation of the cells. As concluded by Tedd & Charles (1981), the earth pressure cell gave better reproducibility than the Camkometer self-boring pressuremeter and the Camkometer self-boring load cell in stiff London clay.

In much of the literature, the self-boring pressuremeter is regarded as a less intrusive method, causing smaller disturbance effects compared to the hydraulic fracturing and earth pressure cell (Hughes & Robertson 1985, Ku & Mayne 2013). Also, the measurements may be taken immediately after arriving at the desired depth, so no time for dissipation of excess pore pressure is required. Similar to the earth pressure cell, the amount of equipment needed is quite limited when using the self-boring pressuremeter. However, the self-boring pressuremeter stands out as the method requiring the most training and experience in order to gain reliable results (Hughes & Robertson 1985, Lunne & L'Heureux 2016). Also, the equipment in it self is rather expensive, and the usage beyond measuring of stresses in soils is limited.

When considering the dilatometer, on of the primary advantages is the large amount of different theoretical correlations proposed. By just performing one sounding, several important geotechnical parameters may be approximated. Moreover, the equipment is quite robust and easy to operate. On the other hand, this rater heavy dependency on theoretical correlations may also be viewed as the primary disadvantage of the method. In fact, the pilot experiments (see Chapter 3), indicate that the original value of $\beta_k = 1.5$ (in Equation 2.7) gives a too high value of K_0', when compared to the value obtained from the earth pressure cell. The limited amount of data from the pilot experiments suggests that the value of $\beta_k = 2$ proposed by Hamouche et al. (1995) indeed gives a better fit for sensitive clays.

Often presented as a completely non-intrusive method, shear wave velocity measurements may also be used for determining K_0' (Ku & Mayne 2013). However, there is a need for more research to evaluate the dependency on theoretical correlations, as well as to investigate the versatility and repeatability of the method in different types of soil.

The stepped blade seems to be too fragile to be of any real interest in field investigations in various soils. And since the assumptions on where the zero thickness should be taken are not good enough, the stepped blade should probably be regarded as less valuable than the reference methods mentioned earlier in this chapter (Lunne & L'Heureux 2016).

The field vane approach seems to rely too much on high quality laboratory results and assumption, even though it has proven to give good results in clay at Haga and Onsøy (Aas et al. 1986).

The latest lateral stress seismic piezocone may be a promising method if one is to listen to the developers of the equipment. Nevertheless, a lot more testing is required before the method can be trusted. The older models of the lateral stress cone seem to be of limited value (Lunne & L'Heureux 2016).

Another distinct feature in much of the literature is the acknowledgement of the simplest theoretical relationships for estimating K'_{0nc}. Several of the articles find that among all the proposed theoretical correlations, the Jaky formula given in Equation 2.11 gives a very reliable estimate of K'_{0nc}.

Throughout the last decades many new theoretical correlations for estimating K'_{0oc} have been proposed. Some by a theoretical approach, others by trying to link results obtained in the field with already existing theoretical assumptions. A general problem with many of these approaches, is their lack of applicability across different sites with different material properties. The primary reason for this seems to be that to a varying degree several different factors affect the process of overconsolidation. The correlation methods to calculate K'_{0oc} lack the ability to account for all these factors in a satisfactory way.

Much research has focused on finding reliable methods of determining the in situ stress state by doing laboratory investigations (see for example Watabe et al. (2003) and Mesri & Hayat (1993)). This is of great interest since some kind of laboratory investigations are part of almost any geotechnical investigation. Nonetheless, the amount and effect of disturbance during sampling and handling of the test specimens continues to be an important and unresolved issue (Karlsrud & Hernandez-Martinez 2013). Furthermore, the ease of sampling is closely linked to the type of material investigated. Consequently, some of the most prevailing articles still recommend in situ measurements as the best way of obtaining valuable estimates of the undisturbed, real stress state (see for instance Hamouche et al. (1995) and Ku & Mayne (2013)).

4.2 Pilot Experiments

The discussion about the pilot experiments is greatly affected by the fact that the amount of data obtained is limited. Therefore, it is hard to draw bold conclusions, but some considerations will be made. Especially the estimated friction angle is uncertain, as given in Appendix F, and the correlations are highly dependant on this value.

Massarsch (1975) found that up to a week was required for dissipation of excess pore pressure after installation of the earth pressure cell. As it may be seen from the Figure E.1, a substantially smaller amount of time was required for reaching an approximate equilibrium situation at Tiller after installation of the cell. Massarsch (1975) used the cell in normally consolidated clay with undrained shear strength of about 20 kPa, decreasing with depth, which resembles the soil conditions at Tiller. The deviation in dissipation time may be explained by difference in installation depth. This factor was also pointed out by Ryley & Carder (1995).

As it may be seen from Figure 3.3, the value of K_0' obtained from the earth pressure cell is somewhat lower than the ones from the dilatometer at a depth of 5 m. All of the correlation values are lower than the dilatometer results. The selection of the two dilatometer curves is briefly discussed in section 3.7. The value of $\beta_k = 2$ may be a bit too low if the results in Figure 3.3 are true, even though Kulhawy & Mayne (1990) predicted β_k to be equal to 2 for sensitive clays. The dilatometer correlation by Lacasse & Lunne (1989) corresponds more to the results obtained from the earth pressure cell and correlation methods if values in Figure 3.3 are true. Lacasse & Lunne (1989) stated that the dilatometer results are normally a bit on the high side, and this seems to be true for the results in the figure.

Furthermore, by comparing the values of K_0' obtained from the theoretical correlations, presented in Table 3.2, it is clear that all the correlations for normally consolidated clays yield pretty similar values. This is also supported in the literature (see for instance Sivakumar et al. (2009)). These values are generally somewhat lower than those obtained in situ at the Tiller site, and this is to be expected since there is slightly overconsolidated clay at Tiller. As it can be seen from Figure 3.3, K_0' decreases with depth in the dilatometer results. This is to be expected in an overconsolidated clay (Tedd & Charles 1981). An approximately OCR of 1.6 based on the oedometer results presented in appendix F was used in the theoretical correlations. Since the sample for the oedometer test is of poor quality, as pointed out in Appendix F, the uncertainty of the OCR value found may be substantial. Due to this uncertainty, also the estimates of K'_{0oc} based on OCR are prone to uncertainties. Gylland et al. (2013) found that OCR was approximately 2 for the NTNU site, and this makes an OCR value of 1.6 used in the pilot experiments probable.

The earth pressure cell result is in between the values from the theoretical correlations. It is a bit higher than the correlation values for normally consolidated clays, which is to be expected for a clay with an OCR of 1.6. The modified Jaky Equation 2.20 is likely to be the most trustworthy of the ones in Figure 3.3 for predicting K'_{0oc}, and the earth pressure cell result is not too far away from this value. The validity of Equation 2.20 is however questioned for sensitive clays (Hamouche et al. 1995).

When considering the results presented in Figure 3.3 it is clear that the correlations between K'_{0nc}, I_p and w_L proposed by Larsson (1977) show consistent results for the clay at Tiller in the pilot experiments. The equations were originally proposed for Scandinavian inorganic clays, see section 2.15, as is the clay type at Tiller (Gylland et al. 2013). Although the relation between K'_{0nc}, I_p and w_L contradicts the findings of Mayne & Kulhawy (1982), which were unable to prove this relationship in their research, no conclusions regarding this apparent relationship can be made based on the limited amount of data presented in this report.

4.3 Summary

During this chapter, some of the key findings of the literature study have been discussed. This discussion adds to the evaluation of the methods already made in Chapter 2. Some important experiences from the pilot experiments have be evaluated, and the findings have been compared to different correlation methods.

Chapter 5

Summary and Recommendations for Further Work

5.1 Summary and Conclusions

Information about the in situ stress state of soils is considered essential in geotechnical engineering challenges, and the horizontal stress state may be hard to determine. To gain a better understanding of the factors affecting the horizontal stress state, as well as the development within field and laboratory measurements of the horizontal stress state and correlation methods, a study of published literature is presented in Chapter 2 and further evaluated in Chapter 4.

Some of the literature indicate a preference for the use of direct measurements of the horizontal stress state. Often, the direct methods of hydraulic fracturing (section 2.6), earth pressure cells (section 2.7) and the Camkometer self-boring pressuremeter (section 2.11) are used as reference methods when investigating the validity of other methods.

Other in situ methods like the dilatometer discussed in section 2.9 are more dependent on empirical correlations. The literature suggest that many of the empirical correlations have limited value when applying them on different soil materials and sites, and therefore there is a need for material specific correlations.

Extensive development has lead to the proposal of several other in situ methods; some intrusive like the field vane (section 2.12), other completely non-intrusive like the shear wave velocity measurements (section 2.8). What many of these other methods have in common is a relatively small amount of published literature treating the methods, and consequently a lack of information regarding the repeatability of the methods.

Many empirical and theoretical correlation methods to determine K_0' exist. A collection is presented in section 2.15. In the literature there is a general agreement that Jaky's formula $K_{0nc}' = 1 - sin\phi'$ in general gives a good estimate for normally consolidated soils. Other formulas for estimating K_{0nc}' have been proposed, but none are as widely acknowledged as Jaky's formula, and some formulas are said to have limited value between different sites. Formulas for calculating K_{0oc}' generally seem less reliable than those for K_{0nc}'. Jaky's formula is often used as a part of the formulas for calculating K_{0oc}'.

A few laboratory methods for determining K_0' have been briefly presented in section 2.14.

In Chapter 3 the Norwegian Geo-Test Site for soft clay at Tiller and the results of initial experiments using a dilatometer and an earth pressure cell are presented. The primary goal of this testing was to learn more about the equipment and to evaluate the quality of the results. Details regarding the data processing and final results are given in section 3.6, while the results are more closely discussed in Chapter 4. These initial experiments indicate that both types of equipment may be utilized for the determination of K_0' in soft clays. However, the results presented in Figure 3.3 suggest that the dilatometer tend to overestimates K_0'. This figure and Table 3.2 also indicate that the collection of correlation methods for normally consolidated clay give comparable results. Further investigations are required to verify the obtained results and to give a more complete evaluation of the equipment.

The primary objective of this project has been to present and evaluate methods to determine the coefficient of earth pressure at rest in situ, as well as briefly treating some correlation and laboratory methods. Based on the literature study presented in Chapter 2 and the discussion in Chapter 4, it may be concluded that this objective has been met.

The secondary objective of this project was to perform pilot experiments with the earth pressure cell and the dilatometer at the Tiller site. The results from the pilot experiments presented in Chapter 3 and the discussion of the results in Chapter 4 show that also the secondary objective has been met.

5.2 Recommendations for Further Work

The work reported herein will be the background for a master's thesis to be written in the spring semester of 2017. The preliminary main objective of the master's thesis will be to gain high quality in situ data. These data can be utilized in order to assess different in situ, laboratory and correlation methods to determine K_0', when combining them with high quality index parameters which need to be determined.

As part of the Norwegian Geo-Test Sites project, there exist further plans at the Tiller test site to utilize earth pressure cells, hydraulic fracturing, a dilatometer and possibly a self-boring pressuremeter for in situ testing. Based on the literature presented in this report, these methods are known to give quite valuable results. Consequently, a comparison of these methods would therefor be of great interest.

If several dilatometer tests are performed and compared to the most reliable in situ methods, it would be interesting to evaluate if the correlations suggested by Marchetti (1980) (Equation 2.8) and Lacasse & Lunne (1989) (Equation 2.9) could be revised in order to give a better fit to the sensitive clay at Tiller.

For the earth pressure cell already installed at Tiller, the zero value of the cell should be checked when the cell is retrieved. The deviation of the zero reading from the calibration value would give a good indication of the quality of the earth pressure cell result reported herein.

Furthermore, retrieval of samples for routine investigations in the laboratory to determine key index parameters for the Tiller clay is required. Hence, it would also be of great interest to investigate some of the laboratory methods like triaxial testing with a K_0'-procedure or utilizing the split-ring oedometer for the determination of K_0', and compare them to the in situ methods.

Finally, as the amount of disturbance between the different methods for determination of the horizontal stress state continues to be debated, it would also be interesting to look more closely into which effects the penetration of a piezometer or dilatometer impose on soil materials. One possible approach to this is duplicating the experiments conducted by Lefebvre et al. (1991). By running falling cone tests on block samples taken at the same depth and location as already performed hydraulic fracturing tests, the disturbance at different distances from the penetrated equipment may be quantified.

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Appendix A

Installation Instructions

This appendix contains the installation instructions for the earth pressure cell used in the pilot experiment.

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GLÖTZL Baumeßtechnik

PRESS-IN PRESSURE CELL for EARTH PRESSURE, combined with POREWATER PRESSURE

Type EPE, EPE/P

Art. No.: 68.60/68.70

With the earth pressure cell to press in, also in combination with a water or porewater pressure cell for effective stress, it is possible to carry out subsequent measurements at or in constructions or in possibly undisturbed underground. The robust model enables an application of pressing powers of up to 2 tons. The cells are available in two pressure pad dimensions, material stainless steel and with load ranges of up to 50 bars.
When loading the pressure pad, the arising hydraulic pressure is transferred to the diaphragm of the electric transducer, and converted into a stress proportional to the loading.

Some application fields:

- Subsequent installation in or at constructions
- Investigation and control of landfills
- Installation behind supporting walls, e.g. port installations
- Earth pressure and porewater pressure in dams
- Pressing into soft, binding soils for control of consolidation at backfills

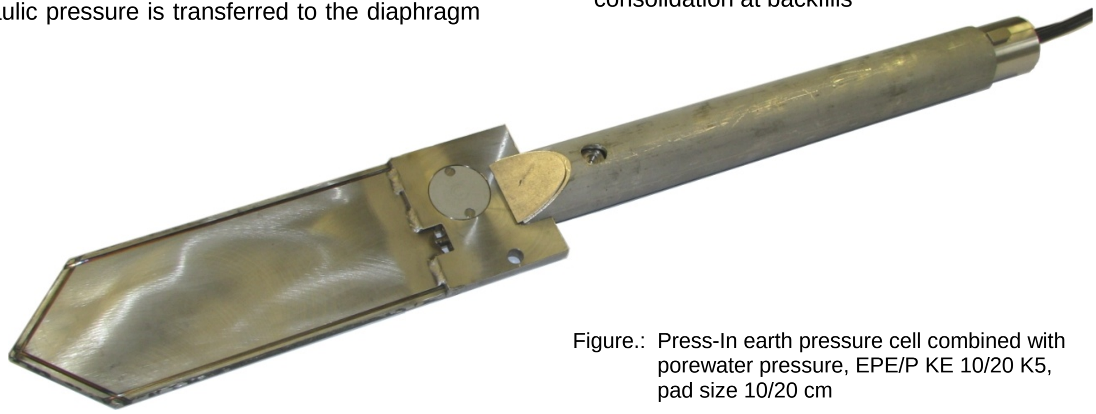

Figure.: Press-In earth pressure cell combined with porewater pressure, EPE/P KE 10/20 K5, pad size 10/20 cm

Models:

EPE Press-In Earth pressure cell
EPE/P Press-In Earth pressure cell combined with porewater pressure

Types:

KE Pressure sensor piezoelectric, 4-conductor system

Technical data:

Supply	constant current 1 mA
Supply optional	4 mA or 10V_{DC}
Output signal	0 – 250 mV
Overload protection (1–50 bars)	50% f.s.
Linearity incl. hysteresis	< 0.5% f.s.
Linearity incl. hysteresis optional	< 0.1% f.s.
Thermal zero drift	0.025 mV/K
Operating temperature range	+5 up to +80 °C
Storage temperature range (dry)	-40 up to +100 °C
Long-term drift temperature dependent (at 0 °C up to 50 °C), typ.	0.25 mV
Resonance	> 30 KHz
Meas. frequency	1 KHz

KO Pressure sensor piezoelectric as above, but with installed amplifier and optional temperature sensor

Technical data:

Supply	15 up to 30 V
Output signal	4 – 20 mA, 2-conductor system
Overload protection	1 – 50 bars, 50% f.s.
Linearity incl. hysteresis	< 0.5% f.s. (optional 0.1% f.s.)
Temperature coefficient	< 0.01%/ °C f.s.
Burden	(U_s-9V) : 20 mA
Operating temperature range	-15 up to +70 °C
Storage temperature range	-15 up to +125 °C
Initialization time after switch-on	6 seconds

Optional with temperature sensor AD 590, output signal 1µA/K

VW Vibrating wire sensor, operating frequency from 2000 cps up to 3300 cps
Thermistor type BR55, T_{25} = 3000 Ohm

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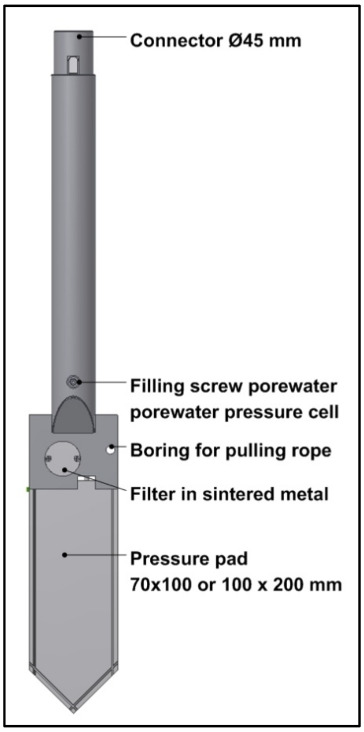
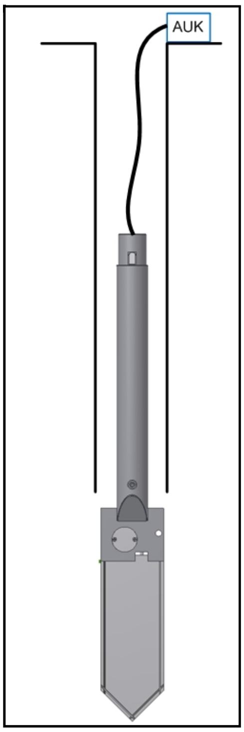




Installation in borings

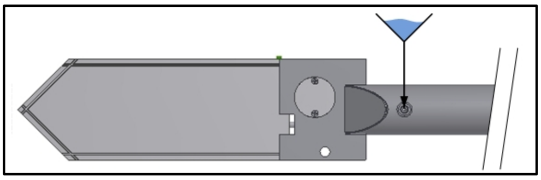
Normally, a boring is done till approx. 0.5 m before the installation point of the cell. From this position, the cell is injected into the surrounding material by means of rods. In soft soils, also an injection is possible without rough-boring.

Injection procedure is done with rods. For this, a thread G 1 ½“ or optionally a connection pivot with diameter 45 mm is fitted at the cell.

After installation, the borehole is backfilled and sealed according to the respective requirements.

Filling of pressure filter of porewater cell

Remove filling screw, screw in water bottle and press the water in. After pressing-in procedure, close again the filling connection with the screw.



Pressure pad size:

70 x 140 mm, 100 x 200 mm, other sizes available on request
720 mm 780 mm total length

Filling:

K Pressure pad with oil filling for the material surrounding the cell, E-modulus ≤ 10.000 bars

Measuring ranges:

1 bars = 100 kPa

Pressure sensor piezoelectric (KE/KO): 0 – 2, 0 – 5, 0 – 10, 0 – 50 bars
Vibrating wire sensor (VW): 1.7, 3.5, 7, 10, 20, 50 bars

Connection

R = rods connection G 1 ½“ Z = thread connection Ø 45 mm

Type key (example for ordering):

EPE/P VW 10/20 K5 Z

- Thread connection G 1 ½” (Z)
- Oil filling, pressure range 0 – 5 bars
- Pressure pad size, **10/20** (100 x 200 mm) or **7/14** (70 x 140 mm)
- Types: Vibrating wire sensor (VW)
- Model (EPE/P)

Registration:

- Battery-operated readout units
- Intermediate amplifier for remote control
- Manually operated change-over manifolds
- Automatic measuring and recording devices with data carrier resp. memory

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Appendix B

Earth Pressure Cell User Manual

This appendix contains the user manual for the earth pressure cell used in the pilot experiment.

Instruction manual: Glötzl pressure cells for horizontal earth pressure

The pressure cells contains two sensors; pore water pressure and earth pressure as shown in the image below. The instrument is inserted vertically using bore rods so that the large sides of the pressure pad are aligned with the gravity vector. Measuring the porewater pressure at the same time allows for compensation of the water pressure and calculating only the horizontal earth pressure.

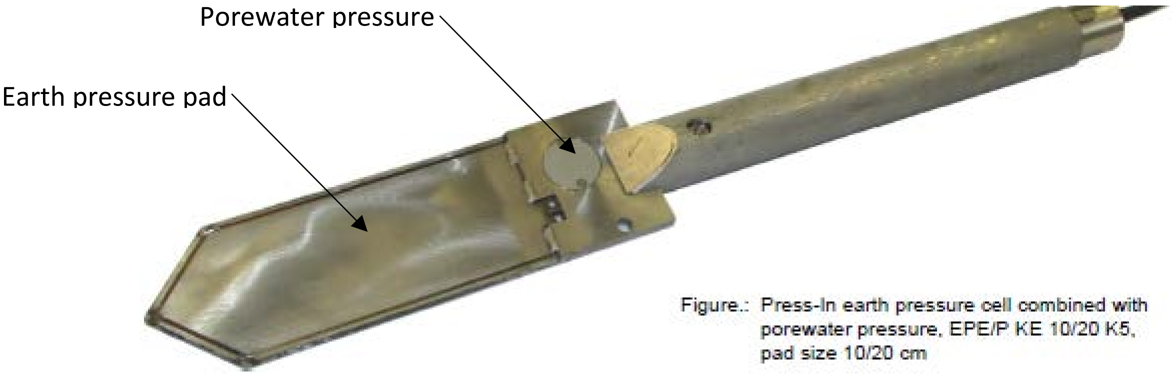
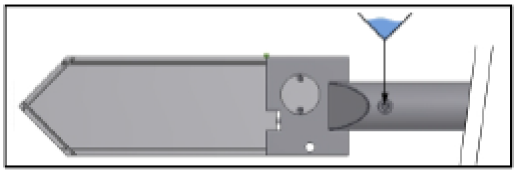


Figure.: Press-In earth pressure cell combined with porewater pressure, EPE/P KE 10/20 K5, pad size 10/20 cm

Consult the calibration sheets to find a pressure cell with the correct measurement range (e.g. 0-2 bar, 0-5 bar or 0-10 bar). The calibration sheets contains parameters necessary for converting raw data to engineering values, so print a copy to bring with you. The serial number of each sensor can be found on a sticker on the pressure pad, or on a sticker near the end of the cables.

Preparations before usage

Make sure that the space behind the porewater pressure filter is filled with water. Remove the filling screw (see image below) and fill as described in the data sheet and water filling instructions. Tap water may contain oily or calcium components that can block the filter. Use either water provided by Glötzl or Milli-Q water from the NGI chemistry lab. If the sensor is to be used in below zero temperatures, pure glycol may be added to the water to avoid freezing. The pressure cells are delivered from Glötzl with a pivot shaft connection without threads. The NGI workshop has added a 5/4" threaded bore rod connection. The data cables have to pass through the bore rods all the way to the surface.



Measurements

Data read out is done with a handheld measurement device (HMG), see operation manual. The earth pressure and pore water pressure is read from different data cables, locate the sticker with serial number near the end of the cable to identify the sensor. Use the grey wire clamps to connect the HMG to the sensor you want to read. Connect the red wire with the red wire from the sensor, and the blue wire with the blue wire from the sensor. The other wires (green, white) can stay disconnected.

After connecting the HMG, one push on the button will start the data read out from channel 1. Channel 2 is for temperature read out, and is not in use on the pressure cells. Normal operation is that the HMG will power down after a few minutes. If you want to avoid this, a long push on the button will enter 'always on' mode.

The raw value range of both earth and pore water pressure sensors is 4-20mA, with a corresponding pressure value in bar, see the calibration sheet for the particular sensor.

Insertion

The sensors should not be exposed to pressures exceeding 150% of full range. For insertion into hard soils or at great depths, this could be an issue. Make sure to read the pressure during insertion and use a sensor with adequate range.

Raw data conversion example

Consult the calibration sheet. The **pore water cell** with serial no. 1624703 has a calibration factor of 3.2 mA/bar. Air pressure at the time of insertion is 1024.3 hpa = 1.0243 bar. The sensor measurement in air is 4.02 mA = (4.02-4.0)mA/3.2mA/bar = 0.0625 bar. Sensor measurement after insertion is 15.6mA = (15.6-4)mA/3.2mA/bar = 3.625 bar. 1.0 bar is 10.2 meter water level. If desired, this can then be compensated for air pressure changes if the air pressure is measured simultaneously with another sensor.

The earth pressure cells have a pre-excitation from the welding of the sensor plates. This is quantified by the pV value in the calibration sheet. For the **earth pressure cell** with serial no. 1624702, we have pV = 8.72mA. The calibration factor is 2.28571mA/bar. The zero-point of this cell is (8.72-3.98)mA/2.28571mA ≈ 2.074bar, which means that the zero point of the measuring value is 2.074bar.

Appendix C

Calibration Sheets

This appendix contains the calibration sheets for the earth pressure cell successfully installed in the pilot experiment.

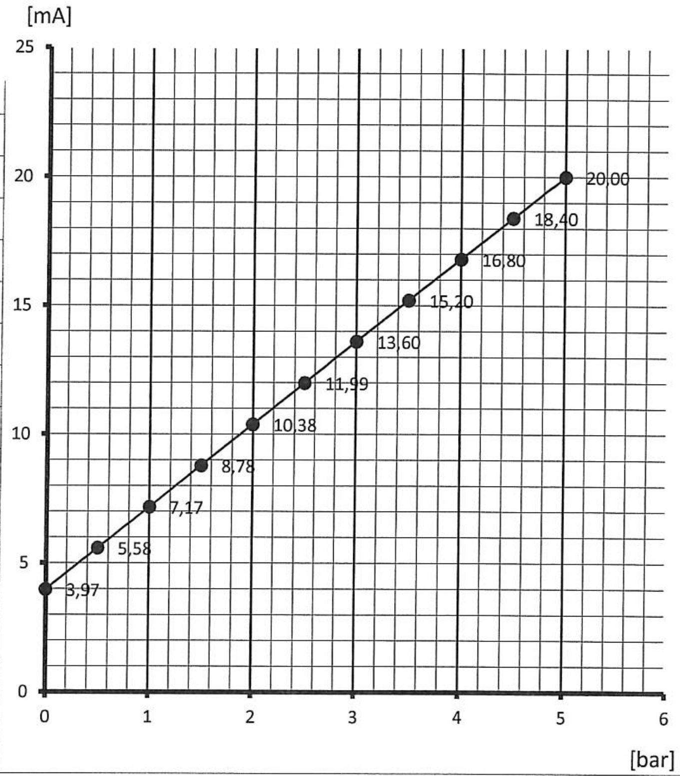
Calibration Data Sheet Advanced Solutions GLÖTZL

EPE/P AI 7/14 K5 Name of measuring point: **PWD**

Order No.: **26537/3**	Cell / Transducer for: **Einpressgeber Porenwasserdruck**			Responsible: Stab
Serial No.: 16 24701	Client / Project: Norwegian Geotechnical Institute			Date: 27.09.2016
Surrounding parameter: 23°C / 1000 mbar	Measuring range: 0-5 bar	Transducer: PA-8-10 / 11	Cable / Length [m]: PE 2(4)x0,5 / 20	Tested:
Linearity: < ± 0,5 %v.E.	Testing equipment: DPI 610 / FMG 2 K-T		Temperature coefficient: <0,5 % / °C v.E.	Temperature range: +5 . . . +60 °C
Supply: 15 - 30 A	Output signal: 4 - 20 mA	Load:	Mean sensitivity: 3,2 mA/bar	Overload security: 20%
Remarks:	**Initialization time of the sensor is 6 seconds.** **Please take over the measuring value only after init. has been finished.**			

Calibration:

Basis in [bar]	Measured value in [mA]	
0,00	3,97	
0,50	5,58	
1,00	7,17	
1,50	8,78	
2,00	10,38	
2,50	11,99	
3,00	13,60	
3,50	15,20	
4,00	16,80	
4,50	18,40	
5,00	20,00	



Calibration factor: 3,2 mA/bar

Calibration factor:

Parameters of measuring equipment:

Offset:	Factor:	Zero:	A-time:	pV-value:

Grouping plan:

Temp.- MV	Resistivity	Pin con-figuration	Connection line Color	Number	Main cable Grouping		Terminator	Measuring device Connection	Channel
		Supply +	red						
		Supply -	blue						
		Screen	transp.						

2010 GS Version 03.05.12

Calibration Data Sheet

Advanced Solutions GLÖTZL

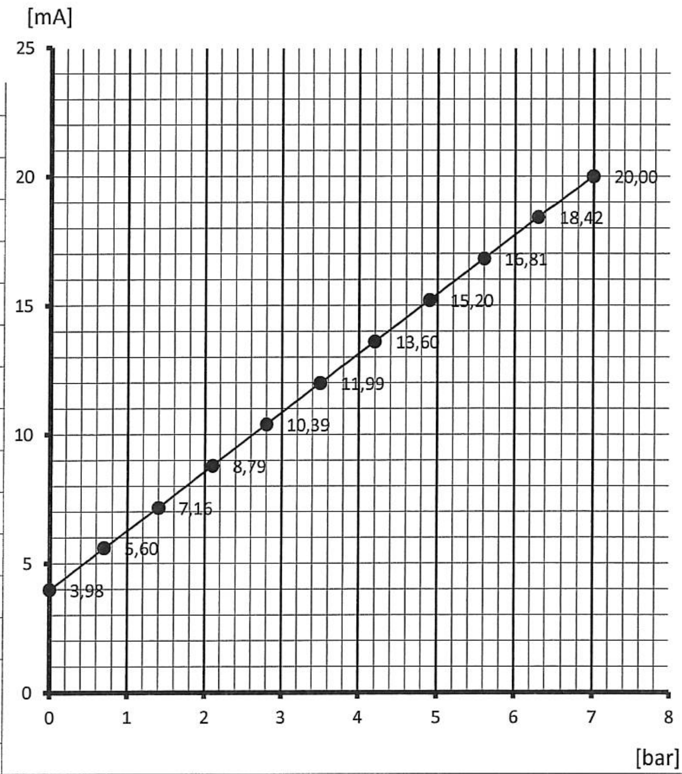
EPE/P AI 7/14 K5 Name of measuring point: **EE**

Order No.: **26537/3**	Cell / Transducer for: **Einpressgeber Erddruck**			Responsible: Stab
Serial No.: 16 24702	Client / Project: Norwegian Geotechnical Institute			Date: 28.09.2016
Surrounding parameter: 23°C / 1000 mbar	Measuring range: 0-7 bar	Transducer: PA-8-10 / 27	Cable / Length [m]: PE 2(4)x0,5 / 20	Tested:
Linearity: < ± 0,5 %v.E.	Testing equipment: DPI 610 / FMG 2 K-T		Temperature coefficient: <0,5 % / °C v.E.	Temperature range: -25 . . . +60 °C
Supply: 15 - 30 A	Output signal: 4 - 20 mA	Load:	Mean sensitivity: 2,28571 mA/bar	Overload security: 20%

Remarks: **Initialization time of the sensor is 6 seconds.**
Please take over the measuring value only after init. has been finished.

Calibration:

Basis in [bar]	Measured value in [mA]	
0,00	3,98	
0,70	5,60	
1,40	7,16	
2,10	8,79	
2,80	10,39	
3,50	11,99	
4,20	13,60	
4,90	15,20	
5,60	16,81	
6,30	18,42	
7,00	20,00	



Calibration factor: 2,28571 mA/bar

Calibration factor:

Parameters of measuring equipment: mA

Offset:	Factor:	Zero:	A-time:	pV-value: 8,72

Grouping plan:

Temp.- MV	Resistivity	Pin con-figuration	Connection line Color	Number	Main cable Grouping		Terminator	Measuring device Connection	Channel
		Supply +	red						
		Supply -	blue						
		Screen	transp.						

Appendix D

Data From Dilatometer

This appendix contains the data material acquired from the dilatometer test at Tiller. Both measurements and calculated correlations is given in the presented spreadsheet. As discussed in section 3.6, some modifications to the original interpretation by the Marchetti DMT software has been made.

DMT TEST RESULTS

Z (m)	A (kPa)	B (kPa)	C (kPa)	Po (kPa)	P1 (kPa)	P2 (kPa)	Gamma (kN/m^3)	Sigma' (kPa)	U (kPa)	Id	Kd	Ed (MPa)	Ud	Ko beta = 2	Ko beta = 1,5	K0_New m = 0,44	K0_New m = 0,64
0,20	28	143		41	77		16,0	3		0,86	12,9	1,2		1,80	2,15	1,05	1,75
0,40	44	242		53	176		16,0	6		2,31	8,3	4,3		1,35	1,64	0,86	1,32
0,60	81	248		92	182		16,0	9	1,0	1,00	10,5	3,1		1,58	1,90	0,96	1,53
0,80	108	271		119	205		16,0	10	2,9	0,74	11,8	3,0		1,70	2,03	1,01	1,65
1,00	120	367		127	301		16,0	11	4,9	1,43	11,0	6,0		1,63	1,95	0,98	1,58
1,20	119	306		129	240		18,5	13	6,9	0,91	9,5	3,9		1,48	1,78	0,92	1,44
1,40	105	458		106	392		18,5	15	8,8	2,93	6,7	9,9		1,16	1,42	0,78	1,15
1,60	104	332		112	266		18,5	16	10,8	1,53	6,2	5,4		1,10	1,35	0,76	1,09
1,80	128	329		137	263		18,5	18	12,8	1,01	6,9	4,4		1,19	1,45	0,79	1,17
2,00	177	518		179	452		18,5	20	14,7	1,66	8,3	9,5		1,35	1,64	0,86	1,32
2,20	135	447		138	381		18,5	22	16,7	1,99	5,7	8,4		1,03	1,27	0,73	1,03
2,40	145	282		157	216		18,5	23	18,6	0,42	6,0	2,0		1,07	1,31	0,75	1,07
2,60	167	273		181	207		18,5	25	20,6	0,16	6,4	0,9		1,13	1,38	0,77	1,12
2,80	176	277		190	211		18,5	27	22,6	0,13	6,3	0,7		1,11	1,36	0,76	1,10
3,00	188	289	143	202	223	158	18,5	28	24,5	0,12	6,2	0,7	0,75	1,11	1,35	0,76	1,10
3,20	196	303		210	237		18,5	30	26,5	0,15	6,1	0,9		1,08	1,33	0,75	1,08
3,40	204	312		218	246		18,5	32	28,4	0,15	5,9	1,0		1,07	1,31	0,74	1,06
3,60	213	317		227	251		18,5	34	30,4	0,12	5,8	0,8		1,05	1,29	0,74	1,05
3,80	215	324		229	258		18,5	35	32,4	0,15	5,5	1,0		1,01	1,25	0,72	1,02
4,00	224	326	169	238	260	184	18,5	37	34,3	0,11	5,5	0,8	0,74	1,01	1,24	0,72	1,01
4,20	213	304	13	228	238	28	18,5	39	36,3	0,05	4,9	0,4	-0,04	0,93	1,15	0,69	0,94
4,40	240	347	8	254	281	23	18,5	41	38,3	0,13	5,3	0,9	-0,07	0,98	1,21	0,71	0,99
4,60	235	340		249	274		18,5	42	40,2	0,12	4,9	0,9		0,93	1,15	0,69	0,94
4,80	233	325		247	259		18,5	44	42,2	0,06	4,7	0,4		0,89	1,10	0,67	0,91
5,00	82	219		245	260		18,5	46	44,1	0,07	4,4	0,5		0,84	1,05	0,65	0,87
5,20	228	326		242	260		18,5	48	46,1	0,09	4,1	0,6		0,80	1,01	0,63	0,84
5,40	238	343		252	277		18,5	49	48,1	0,12	4,1	0,9		0,81	1,01	0,63	0,84
5,60	246	356		260	290		18,5	51	50,0	0,15	4,1	1,1		0,80	1,00	0,63	0,84
5,80	256	359		270	293		18,5	53	52,0	0,11	4,1	0,8		0,81	1,01	0,63	0,84
6,00	263	364	220	277	298	235	18,5	55	54,0	0,09	4,1	0,7	0,81	0,80	1,00	0,63	0,84
6,20	272	366		286	300		18,5	56	55,9	0,06	4,1	0,5		0,80	1,00	0,63	0,84
6,40	272	376		286	310		18,5	58	57,9	0,11	3,9	0,8		0,77	0,97	0,62	0,82
6,60	296	386		311	320		18,5	60	59,8	0,04	4,2	0,3		0,82	1,02	0,64	0,85
6,80	295	391		309	325		18,5	61	61,8	0,06	4,0	0,5		0,79	0,99	0,63	0,83
7,00	292	378	249	307	312	264	18,5	63	63,8	0,02	3,8	0,2	0,82	0,76	0,96	0,61	0,80
7,20	302	398		316	332		18,5	65	65,7	0,06	3,9	0,5		0,76	0,96	0,62	0,81
7,40	308	393		323	327		18,5	67	67,7	0,02	3,8	0,1		0,76	0,95	0,61	0,80
7,60	306	404		320	338		18,5	68	69,7	0,07	3,7	0,6		0,73	0,92	0,60	0,78
7,80	330	426		344	360		18,5	70	71,6	0,06	3,9	0,5		0,77	0,96	0,62	0,81
8,00	338	434	286	352	368	301	18,5	72	73,6	0,06	3,9	0,5	0,82	0,76	0,96	0,62	0,81
8,20	355	452		369	386		18,5	74	75,5	0,06	4,0	0,6		0,78	0,98	0,62	0,82
8,40	355	450		369	384		18,5	75	77,5	0,05	3,9	0,5		0,76	0,96	0,62	0,81
8,60	355	441		370	375		18,5	77	79,5	0,02	3,8	0,2		0,75	0,94	0,61	0,79
8,80	347	439		361	373		18,5	79	81,4	0,04	3,6	0,4		0,71	0,90	0,59	0,76
9,00	351	446	305	365	380	320	18,5	81	83,4	0,05	3,5	0,5	0,84	0,70	0,89	0,59	0,76
9,20	358	450		372	384		18,5	82	85,3	0,04	3,5	0,4		0,70	0,89	0,59	0,76
9,40	346	440		360	374		18,5	84	87,3	0,05	3,2	0,5		0,66	0,84	0,57	0,72
9,60	369	467		383	401		18,5	86	89,3	0,06	3,4	0,6		0,69	0,87	0,58	0,75
9,80	370	460		385	394		18,5	88	91,2	0,03	3,3	0,3		0,67	0,86	0,58	0,74
10,00	377	466	328	392	400	343	18,5	89	93,2	0,03	3,3	0,3	0,84	0,67	0,86	0,58	0,74
10,20	385	486		399	420		18,5	91	95,2	0,07	3,3	0,7		0,67	0,86	0,58	0,74
10,40	373	469		387	403		18,5	93	97,1	0,05	3,1	0,5		0,63	0,81	0,56	0,71
10,60	392	480		407	414		18,5	95	99,1	0,02	3,3	0,3		0,66	0,84	0,57	0,72
10,80	397	490		411	424		18,5	96	101,0	0,04	3,2	0,4		0,65	0,83	0,57	0,72
11,00	402	509	344	416	443	359	18,5	98	103,0	0,09	3,2	0,9	0,82	0,65	0,83	0,57	0,71
11,20	422	511		437	445		18,5	100	105,0	0,03	3,3	0,3		0,67	0,85	0,58	0,73
11,40	425	512		440	446		18,5	101	106,9	0,02	3,3	0,2		0,66	0,84	0,57	0,73
11,60	429	523		443	457		18,5	103	108,9	0,04	3,2	0,5		0,65	0,84	0,57	0,72
11,80	415	509		429	443		18,5	105	110,9	0,04	3,0	0,5		0,62	0,79	0,55	0,69
12,00	438	540		452	474		18,5	107	112,8	0,07	3,2	0,8		0,64	0,82	0,57	0,71
12,20	451	543	393	465	477	408	18,5	108	114,8	0,03	3,2	0,4	0,84	0,65	0,83	0,57	0,72
12,40	447	538		461	472		18,5	110	116,7	0,03	3,1	0,4		0,63	0,81	0,56	0,71
12,60	445	533		460	467		18,5	112	118,7	0,02	3,0	0,3		0,62	0,80	0,56	0,69
12,80	437	530		451	464		18,5	114	120,7	0,04	2,9	0,4		0,59	0,77	0,54	0,67
13,00	426	527	377	440	461	392	18,5	115	122,6	0,07	2,8	0,7	0,85	0,56	0,73	0,53	0,65
13,20	448	552	123	462	486	138	18,5	117	124,6	0,07	2,9	0,8	0,04	0,59	0,76	0,54	0,67
13,40	467	567		481	501		18,5	119	126,5	0,06	3,0	0,7		0,61	0,78	0,55	0,68
13,60	473	581		487	515		18,5	121	128,5	0,08	3,0	1,0		0,60	0,78	0,55	0,68
13,80	490	587		504	521		18,5	122	130,5	0,04	3,1	0,6		0,62	0,80	0,56	0,69
14,00	484	571	429	499	505	444	18,5	124	132,4	0,02	3,0	0,2	0,85	0,60	0,77	0,55	0,68
14,20	484	574	425	499	508	440	18,5	126	134,4	0,03	2,9	0,3	0,84	0,59	0,76	0,54	0,67
14,40	489	577		504	511		18,5	128	136,4	0,02	2,9	0,3		0,59	0,76	0,54	0,67
14,60	533	624		547	558		18,5	129	138,3	0,03	3,2	0,4		0,64	0,82	0,56	0,71
14,80	537	639		551	573		18,5	131	140,3	0,05	3,1	0,8		0,64	0,81	0,56	0,71
15,00	560	658		574	592		18,5	133	142,2	0,04	3,3	0,6		0,66	0,84	0,57	0,72
15,20	576	668	51	590	602	66	18,5	134	144,2	0,03	3,3	0,4	-0,18	0,67	0,85	0,58	0,73
15,40	572	677	502	586	611	517	18,5	136	146,2	0,06	3,2	0,9	0,84	0,65	0,83	0,57	0,72
15,60	601	699		615	633		18,5	138	148,1	0,04	3,4	0,6		0,68	0,87	0,58	0,74
15,80	609	701		623	635		18,5	140	150,1	0,02	3,4	0,4		0,68	0,87	0,58	0,74
16,00	626	727	538	640	661	553	18,5	141	152,1	0,04	3,4	0,7	0,82	0,69	0,88	0,59	0,75
16,20	630	731		644	665		18,5	143	154,0	0,04	3,4	0,7		0,69	0,87	0,58	0,75
16,40	627	725		641	659		18,5	145	156,0	0,04	3,3	0,6		0,67	0,86	0,58	0,74
16,60	639	739		653	673		18,5	147	157,9	0,04	3,4	0,7		0,68	0,86	0,58	0,74
16,80	656	757		670	691		18,5	148	159,9	0,04	3,4	0,7		0,69	0,88	0,59	0,75
17,00	649	761	574	662	695	589	18,5	150	161,9	0,07	3,3	1,1	0,85	0,67	0,86	0,58	0,73
17,20	661	766		675	700		18,5	152	163,8	0,05	3,4	0,9		0,68	0,86	0,58	0,74
17,40	664	772		678	706		18,5	154	165,8	0,06	3,3	1,0		0,67	0,86	0,58	0,73
17,60	669	777		683	711		18,5	155	167,8	0,06	3,3	1,0		0,67	0,85	0,58	0,73
17,80	673	780		687	714		18,5	157	169,7	0,05	3,3	0,9		0,66	0,85	0,57	0,73
18,00	685	801	601	698	735	616	18,5	159	171,7	0,07	3,3	1,3	0,84	0,67	0,85	0,58	0,73
18,20	680	792	53	693	726	68	18,5	161	173,6	0,06	3,2	1,1	-0,20	0,65	0,84	0,57	0,72
18,40	694	796	55	708	730	70	18,5	162	175,6	0,04	3,3	0,8	-0,20	0,66	0,84	0,57	0,73
18,60	712	820	53	726	754	68	18,5	164	177,6	0,05	3,3	1,0	-0,20	0,67	0,86	0,58	0,74
18,80	726	839		739	773		18,5	166	179,5	0,06	3,4	1,2		0,68	0,86	0,58	0,74
19,00	729	837	640	743	771	655	18,5	168	181,5	0,05	3,3	1,0	0,84	0,67	0,86	0,58	0,74
19,20	734	845		747	779		18,5	169	183,4	0,06	3,3	1,1		0,67	0,86	0,58	0,73
19,40	737	852		750	786		18,5	171	185,4	0,06	3,3	1,2		0,67	0,85	0,58	0,73
19,60	751	853		765	787		18,5	173	187,4	0,04	3,3	0,8		0,67	0,86	0,58	0,74
19,80	759	871		772	805		18,5	174	189,3	0,06	3,3	1,1		0,67	0,86	0,58	0,74
20,00	771	888	668	784	822	683	18,5	176	191,3	0,06	3,4	1,3	0,83	0,68	0,86	0,58	0,74
20,20	794	912		807	846		18,5	178	193,3	0,06	3,4	1,3		0,69	0,88	0,59	0,75
20,40	817	928		831	862		18,5	180	195,2	0,05	3,5	1,1		0,71	0,90	0,59	0,76
20,60	836	950		849	884		18,5	181	197,2	0,05	3,6	1,2		0,72	0,91	0,60	0,77
20,80	816	935		829	869		18,5	183	199,1	0,06	3,4	1,4		0,69	0,88	0,59	0,75
21,00	824	949	72	837	883	87	18,5	185	201,1	0,07	3,4	1,6	-0,18	0,69	0,88	0,59	0,75
21,20	826	944	97	839	878	112	18,5	187	203,1	0,06	3,4	1,3	-0,14	0,68	0,87	0,58	0,75
21,40	831	953	714	844	887	729	18,5	188	205,0	0,07	3,4	1,5	0,82	0,68	0,87	0,58	0,74
21,60	818	942		831	876		18,5	190	207,0	0,07	3,3	1,6		0,66	0,84	0,57	0,73
21,80	840	960		853	894		18,5	192	209,0	0,06	3,4	1,4		0,68	0,86	0,58	0,74
22,00	857	970	740	870	904	755	18,5	194	210,9	0,05	3,4	1,2	0,83	0,68	0,87	0,58	0,75
22,20	880	1003		893	937		18,5	195	212,9	0,06	3,5	1,5		0,70	0,89	0,59	0,76
22,40	899	1026		912	960		18,5	197	214,8	0,07	3,5	1,7		0,71	0,90	0,59	0,76
22,60	916	1049		928	983		18,5	199	216,8	0,08	3,6	1,9		0,71	0,90	0,60	0,77
22,80	947	1108		958	1042		18,5	201	218,8	0,11	3,7	2,9		0,73	0,93	0,60	0,78
23,00	948	1130	789	958	1064	804	18,5	202	220,7	0,14	3,6	3,7	0,79	0,73	0,92	0,60	0,78
23,20	957	1106		969	1040		18,5	204	222,7	0,10	3,7	2,5		0,73	0,92	0,60	0,78
23,40	665	747		680	681		18,5	206	224,6	0,00	2,2	0,0		0,45	0,60	0,48	0,57
23,50	991	1134		1003	1068		18,5	208	225,6	0,08	3,7	2,3		0,74	0,93	0,61	0,79
23,60	994	1095		1008	1029		18,5	211	226,6	0,03	3,7	0,7		0,74	0,93	0,60	0,79
23,80	969	1055		984	989		18,5	213	228,6	0,01	3,5	0,2		0,71	0,90	0,59	0,76
24,00	977	1039	801	993		816	18,5	215	230,5	-1,30	3,6		0,77	0,71	0,90	0,59	0,77
24,20	948	1022		963			18,5	216	232,5	-1,32	3,4			0,68	0,86	0,58	0,74
24,40	971	1059		986	993		18,5	218	234,5	0,01	3,4	0,3		0,69	0,88	0,59	0,75
24,60	965	1047		980	981		18,5	220	236,4	0,00	3,4	0,0		0,68	0,87	0,58	0,74
24,80	935	1006		950			18,5	222	238,4	-1,33	3,2			0,65	0,83	0,57	0,72
25,00	922	982	799	938		814	18,5	223	240,3	-1,34	3,1		0,82	0,63	0,81	0,56	0,70

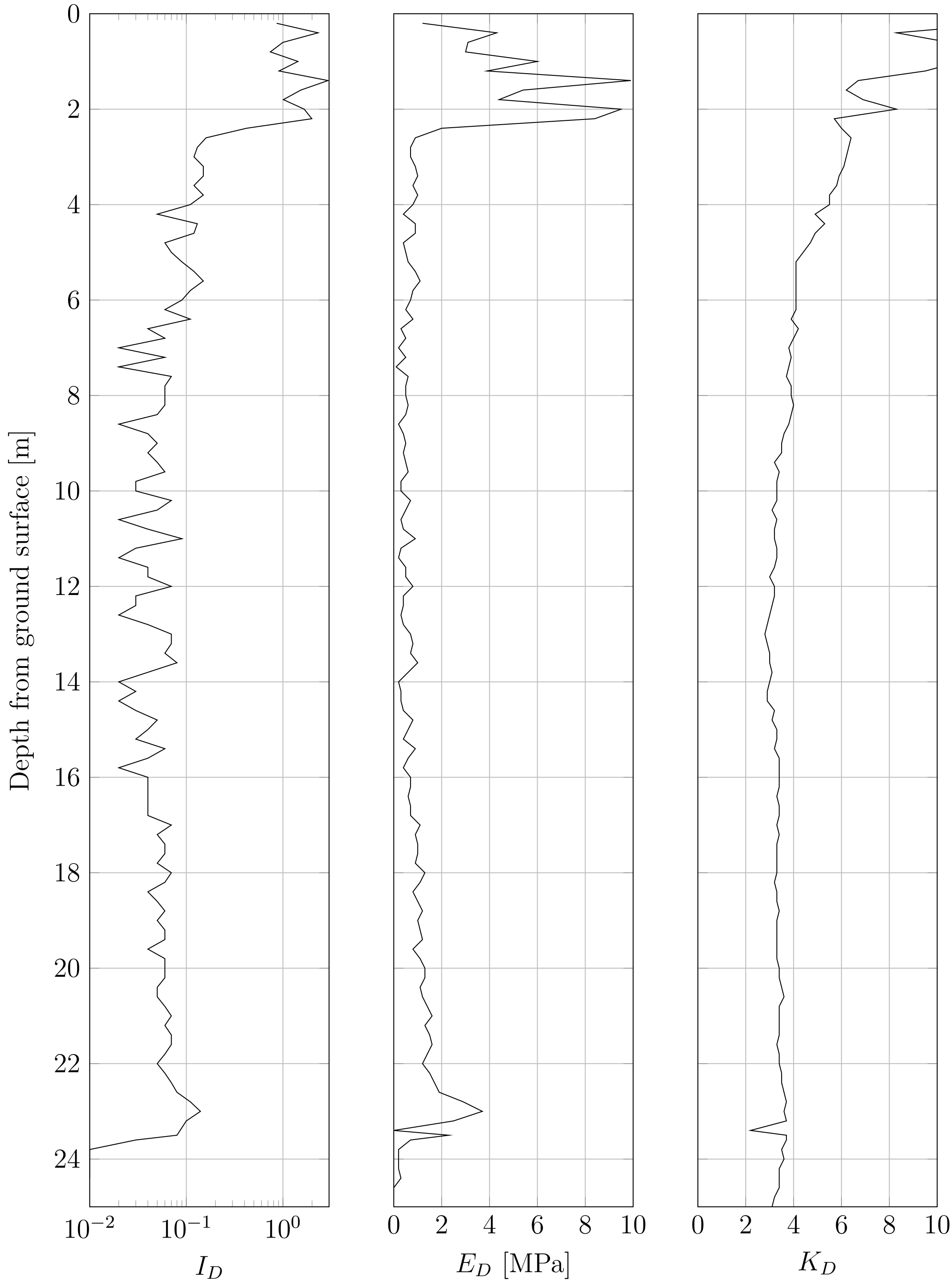


Figure D.1: Presentation of key parameters from dilatometer test at Tiller.

Appendix E

Data From Earth Pressure Cell

This appendix contains data material acquired from the earth pressure cell at Tiller, as well as the calculations made to find the value of K_0'. Also, the figure E.1 showing how the readings reach an equilibrium situation with time is included.

The registered values from the earth pressure cell were processed in Excel as described in Appendix B and section 3.6.

Bore hole	**BH13, Tiller**
Time of installation	15.11.2016 11:00:00
Depth middle of cell	5,0 m
Predrilling depth	4,70 m
Earth pressure sensor serial no.	EE24702
Mean sensitivity [mA/bar]	2,28571
Current at 0 bar [mA]	3,98
pV [mA]	8,72
Pore pressure sensor serial no.	PWD24701
Mean sensitivity [mA/bar]	3,2
Current at 0 bar [mA]	3,97
Calculated final pore pressure [kPa]	45,13
Calculated final total earth pressure [kPa]	70,26
Calculated horizontal effective stress [kPa]	25,13
Effective overburden pressure [kPa]	45
Calculated K_0'	**0,56**

Earth pressure

Zero reading at surface [mA]: 8,656

Time	Time after installation [min]	Reading [mA]	Calculated pressure [bar]	Calculated pressure [kPa]
15.11.2016 11:00:00	0	13,659	2,160816552	216,0816552
15.11.2016 11:07:00	7,0	12,195	1,520315351	152,0315351
15.11.2016 11:07:30	7,5	12,176	1,512002835	151,2002835
15.11.2016 11:08:00	8,0	12,157	1,503690319	150,3690319
15.11.2016 11:08:30	8,5	12,134	1,493627801	149,3627801
15.11.2016 11:09:00	9,0	12,113	1,484440283	148,4440283
15.11.2016 11:09:30	9,5	12,084	1,47175276	147,175276
15.11.2016 11:10:00	10,0	12,07	1,465627748	146,5627748
15.11.2016 11:13:30	13,5	11,932	1,405252635	140,5252635
15.11.2016 11:14:00	14,0	11,921	1,400440126	140,0440126
15.11.2016 11:14:30	14,5	11,901	1,391690109	139,1690109
15.11.2016 11:15:00	15,0	11,888	1,386002599	138,6002599
15.11.2016 11:15:38	15,6	11,87	1,378127584	137,8127584
15.11.2016 11:18:30	18,5	11,79	1,343127518	134,3127518
15.11.2016 11:19:00	19,0	11,775	1,336565006	133,6565006
15.11.2016 11:19:30	19,5	11,766	1,332627499	133,2627499
15.11.2016 11:20:00	20,0	11,752	1,326502487	132,6502487
15.11.2016 11:20:30	20,5	11,745	1,323439981	132,3439981
15.11.2016 11:24:00	24,0	11,663	1,287564914	128,7564914
15.11.2016 11:24:30	24,5	11,659	1,285814911	128,5814911
15.11.2016 11:25:00	25,0	11,646	1,2801274	128,01274
15.11.2016 11:25:30	25,5	11,634	1,27487739	127,487739
15.11.2016 11:26:00	26,0	11,627	1,271814885	127,1814885
15.11.2016 11:26:30	26,5	11,617	1,267439876	126,7439876
15.11.2016 11:27:00	27,0	11,605	1,262189867	126,2189867
15.11.2016 11:27:30	27,5	11,603	1,261314865	126,1314865
15.11.2016 12:12:00	72,0	11,118	1,049126967	104,9126967
15.11.2016 14:09:00	189,0	10,748	0,887251664	88,72516636
15.11.2016 15:02:00	242,0	10,668	0,852251598	85,2251598
15.11.2016 15:42:00	282,0	10,627	0,834314064	83,43140643
16.11.2016 08:36:00	1296,0	10,412	0,740251388	74,0251388
16.11.2016 10:43:00	1423,0	10,414	0,74112639	74,11263896
16.11.2016 12:48:00	1548,0	10,404	0,736751381	73,67513814
16.11.2016 15:38:00	1718,0	10,438	0,751626409	75,16264093
17.11.2016 10:59:00	2879,0	10,339	0,708313828	70,83138281
18.11.2016 11:31:00	4351,0	10,314	0,697376308	69,73763076
21.11.2016 14:31:00	8851,0	10,326	0,702626317	**70,26263174**

Pore pressure

Zero reading at surface [mA]: 4,028

Time	Time after installation [min]	Reading [mA]	Calculated pressure [bar]	Calculated pressure [kPa]
15.11.2016 11:00:00				
15.11.2016 11:00:30	0,5	7,38	1,065625	106,5625
15.11.2016 11:01:00	1,0	7,604	1,135625	113,5625
15.11.2016 11:01:30	1,5	7,705	1,1671875	116,71875
15.11.2016 11:02:00	2,0	7,834	1,2075	120,75
15.11.2016 11:02:30	2,5	7,923	1,2353125	123,53125
15.11.2016 11:03:00	3,0	8,002	1,26	126
15.11.2016 11:03:30	3,5	8,095	1,2890625	128,90625
15.11.2016 11:04:00	4,0	8,164	1,310625	131,0625
15.11.2016 11:04:30	4,5	8,227	1,3303125	133,03125
15.11.2016 11:05:00	5,0	8,288	1,349375	134,9375
15.11.2016 11:05:30	5,5	8,339	1,3653125	136,53125
15.11.2016 11:06:00	6,0	8,39	1,38125	138,125
15.11.2016 11:10:30	10,5	8,605	1,4484375	144,84375
15.11.2016 11:11:00	11,0	8,605	1,4484375	144,84375
15.11.2016 11:11:30	11,5	8,616	1,451875	145,1875
15.11.2016 11:12:00	12,0	8,616	1,451875	145,1875
15.11.2016 11:12:30	12,5	8,616	1,451875	145,1875
15.11.2016 11:16:00	16,0	8,202	1,3225	132,25
15.11.2016 11:16:30	16,5	8,155	1,3078125	130,78125
15.11.2016 11:17:00	17,0	8,106	1,2925	129,25
15.11.2016 11:17:30	17,5	8,064	1,279375	127,9375
15.11.2016 11:18:00	18,0	8,03	1,26875	126,875
15.11.2016 11:21:00	21,0	7,873	1,2196875	121,96875
15.11.2016 11:21:30	21,5	7,85	1,2125	121,25
15.11.2016 11:22:00	22,0	7,834	1,2075	120,75
15.11.2016 11:22:30	22,5	7,801	1,1971875	119,71875
15.11.2016 11:23:00	23,0	7,764	1,185625	118,5625
15.11.2016 11:23:30	23,5	7,748	1,180625	118,0625
15.11.2016 11:28:00	28,0	7,555	1,1203125	112,03125
15.11.2016 11:28:30	28,5	7,559	1,1215625	112,15625
15.11.2016 11:29:00	29,0	7,49	1,1	110
15.11.2016 11:29:30	29,5	7,425	1,0796875	107,96875
15.11.2016 11:30:00	30,0	7,431	1,0815625	108,15625
15.11.2016 11:30:30	30,5	7,346	1,055	105,5
15.11.2016 12:11:00	71,0	6,62	0,828125	82,8125
15.11.2016 14:07:00	187,0	5,815	0,5765625	57,65625
15.11.2016 15:04:00	244,0	6,174	0,68875	68,875
15.11.2016 15:41:00	281,0	6,173	0,6884375	68,84375
16.11.2016 08:38:00	1298,0	5,614	0,51375	51,375
16.11.2016 10:45:00	1425,0	5,621	0,5159375	51,59375
16.11.2016 12:50:00	1550,0	5,603	0,5103125	51,03125
16.11.2016 15:35:00	1715,0	5,607	0,5115625	51,15625
17.11.2016 10:57:00	2877,0	5,455	0,4640625	46,40625
18.11.2016 11:29:00	4349,0	5,407	0,4490625	44,90625
21.11.2016 14:32:00	8852,0	5,414	0,45125	**45,125**

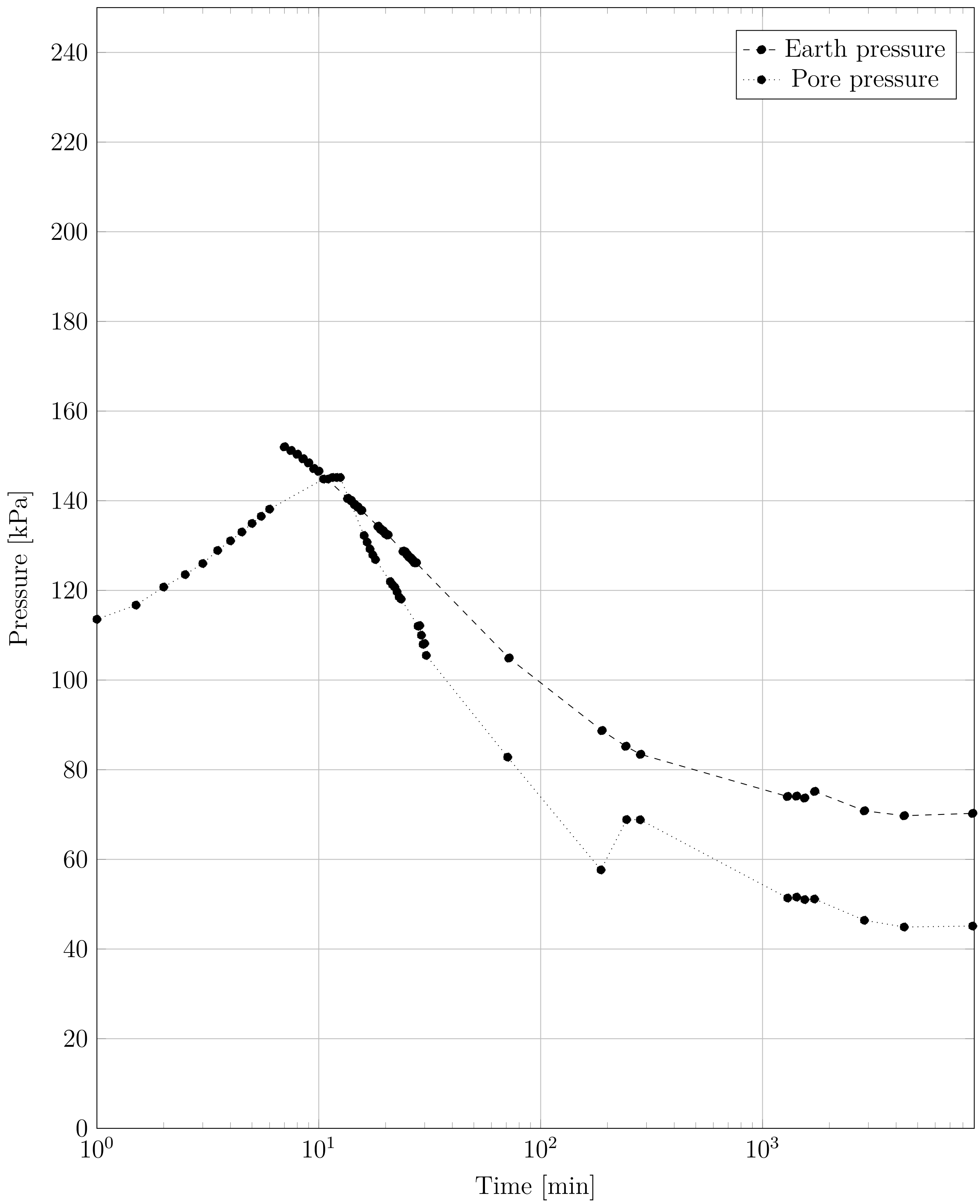


Figure E.1: Earth and pore pressure with time.

Appendix F

Laboratory Investigations

This appendix contains preliminary laboratory results provided by Samson Degago at the Norwegian Public Roads Administration. The first page presents key index parameters. Values for depth around 5 m are used in the correlation methods presented in Chapter 3. The next page presents the results of an oedometer test for a sample from a depth of 6.45 m. The value of OCR found from this test is used for the correlation methods values presented in Chapter 3. Please note that the test indicates poor sample quality.

Prøveopphav: (B) Byggherre (E) Entreprenør (P) Produsent

Statens vegvesen

Region Øst

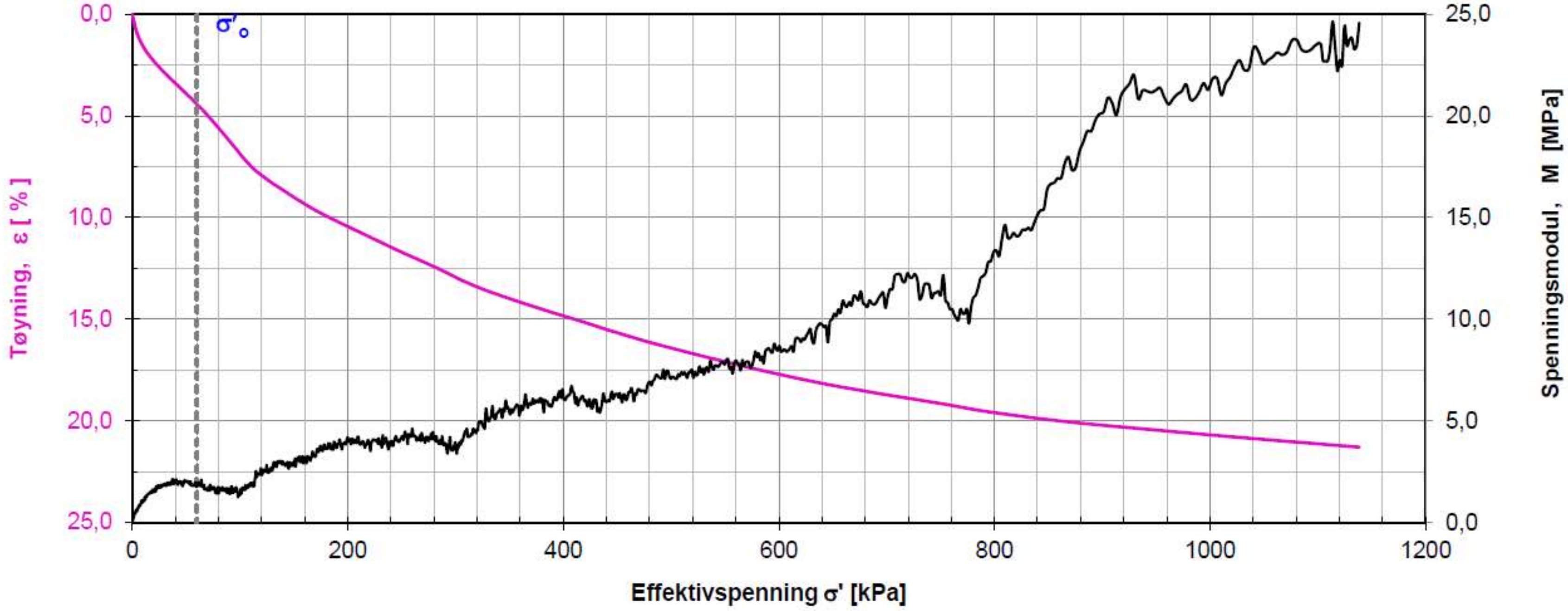
Borprofil, tabell

Oppdragsnr. 1160358 Navn Felles FoU Analyseår 2016 Prøvetype 54mm stål
Serienr. $5_{(B)}$ Hullnummer 16 Koordinater

Prøve	Delprøve	Dybde	Jordart	Densitet	Humusinnhold	Vanninnhold W	Flytegrense W_L	Utrullingsgrense W_P	Enkelt trykkforsøk C_{uuc}	Enkelt trykkforsøk Deformasjon	Konus, Uomrørt, C_{ufc}	Konus, Omrørt, C_{urfc}	Sensitivitet, St
		[m]		[kN/m3]	[%]	[%]	[%]	[%]	[kPa]	[%]	[kPa]	[kPa]	
1	A	5.15	Leire	18.5		38.2							
1	B	5.25							34.5	5			
1	C	5.35				40.1	38	21			31.9	2.9	11
1	D	5.45							37.6	5			
1	E	5.55									32.5	2.1	16
1	F	5.65				35.3							
2	A	8.15	Leire	19.4		33.5							
2	B	8.25							22.0	8			
2	C	8.35				34.1	21	17			18.6	0.3	64
2	D	8.45							30.0	7			
2	E	8.55				32.6					20.7	0.5	42
2	F	8.65											
3	A	11.15	Leire			28.8	21	15				0.3	
3	B	11.25											
3	C	11.35											
3	D	11.45											
3	E	11.55											
3	F	11.65											

Laboratorium: Sentrallaboratoriet Oslo - I henhold til H014 labprosess: 14.425, 14.426, 14.441, 14.442, 14.445, 14.471, 14.472

Laboratory investigation - CRS



Bp. 13, D= 6.45 m (75 mm), $\Delta\varepsilon_o$= 4.3%, $\Delta e_o/e_o$= 0.082 => Poor sample quality, OCR ≈ 1.6