Embankments founded on sulphide clay
-some aspects related to ground improvement by vertical drains

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Licentiate Thesis
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Stockholm, 2010
“There is only one thing about which I am certain, and this is that there is very little about which one can be certain.”

William Somerset Maugham (1874-1965)
The research project presented in this thesis was carried out from December 2007 to May 2010 at the Department of Civil and Architectural Engineering, Royal Institute of Technology (KTH). The supervisor for my research was Professor Stefan Larsson.

The funding for my research was provided in full by the Sven Tyréns Foundation. Sincere thanks are directed to them for providing me with the opportunity to conduct PhD studies in the field of geotechnical engineering.

Similarly, sincere thanks are directed to my supervisor, Stefan Larsson, whose encouragement and engagement in my work have been invaluable. I hope our interesting and fruitful discussions will continue for a long time to come.

I would also like to show my appreciation to all of my colleagues at Tyréns AB for the great collaboration we have had over the years. Special thanks are directed to our great field engineers Hans-Ola Engström, Ingemar Engström, Hans Frelin, Håkan Henriksson and Johan Sjölén for their devoted fieldwork and in situ measurements.

My colleagues at the Division of Soil and Rock Mechanics at the Royal Institute of Technology are also acknowledged. A special thanks to Professor Håkan Stille for reviewing the manuscript and providing some valuable comments on the text.

The personnel at the Ådalsbanan Railway project and at the E4 Enånger-Hudiksvall project are also gratefully acknowledged. I would like to mention Peter Zackrisson and Kjell-Ola Berg in particular for their support and assistance throughout the project.

Warm thanks are also due to Rolf Larsson at the Swedish Geotechnical Institute and to Bengt-Arne Torstensson at BAT AB for our meaningful discussions concerning the mechanical behaviour of clay and the interaction of clay and vertical drains.

Last but not least, I would like to express my most sincere thanks to my beloved and admired friend, Anna, for sharing her life with me.

Borlänge, Sweden June 2010

Rasmus Müller
Abstract

In this thesis, some aspects concerning building embankments founded on sulphide clay are studied, with special reference to ground improvement by preloading in combination with prefabricated vertical drains (PVD’s). The main purpose of the research was to increase empirical knowledge of the mechanical behaviour of sulphide clays subjected to embankment loadings and of the interaction between vertical drains and sulphide clays. Important aspects related to ground improvement with PVD’s in more general terms are also treated, in particular how various uncertainties regarding the properties of the clay and the clay-drain interaction imposed in the design phase can be addressed. The benefits of using the observational method for handling these uncertainties are discussed, and a description of how the method was used in an embankment project is presented. The results from the research are presented in one conference paper and two papers submitted to peer-reviewed international journals, which are appended.

The design of PVD’s involves describing the consolidation characteristics of the clay and the interaction between the drains and the clay. Primarily, the rate of consolidation is determined by the hydraulic conductivity (permeability) of the clay in the horizontal direction. Hence, accurate determination of this material property is of paramount importance in making reliable design predictions. As conventional laboratory tests for assessing the consolidation characteristics of a clay only provide information about its properties in the vertical direction, one is often left to make assumptions about the horizontal properties based on empirical correlations. Reliable empirical knowledge of these correlations for a certain clay is therefore vital. A large number of CRS tests were performed on horizontal and vertical samples of sulphide clay in order to investigate the correlation between the horizontal and vertical hydraulic conductivity and coefficient of consolidation. The results show that there is very small anisotropy in these parameters and that the scatters in the results are large. For design purposes, sulphide clays should therefore be assumed to be isotropic in this respect. In order to handle the variation in properties, several parallel tests should be made and partial factors of safety should be introduced in the design. Introducing partial factors of safety in the design of PVD’s is one of the main topics suggested for further research. Regarding the clay-drain interaction, a study of the disturbance effects (smear effects) during the installation of drains in sulphide clays was performed. Back-calculations of measurements of pore pressure dissipation were made via a parameter study. It was shown that smear affects the consolidation rate to some extent but that the natural (undisturbed) hydraulic conductivity is more significant.

The undrained shear strength $s_u$ of a clay is dependent on the preconsolidation pressure $\sigma'_p$. As the clay consolidates under a loading, the effective stress increases, possibly to magnitudes surpassing the initial preconsolidation pressure and thereby leading to increased undrained shear strength of the clay. The relation between $s_u$ and $\sigma'_p$, i.e. the ratio $s_u/\sigma'_p$ for a sulphide clay, was investigated based on results from a large number of in situ tests and laboratory tests. There were large scatters in the measurements, but $s_u/\sigma'_p=0.25$ is suggested as being relevant in the direct shear zone for design purposes in sulphide clays.
Sammanfattning

Några aspekter kring bankuppfyllnad på sulfidlera studeras i denna avhandling. Speciellt inriktas studien mot djupstabilisering via förbelastning i kombination med prefabricerade vertikaldräner (PVD’s). Forskningen syftade huvudsakligen till att öka den empiriska kunskapen kring det mekaniska beteendet hos sulfidlera under belastning från bankonstruktioner och interaktionen mellan PVD’s och sulfidlera. En del generella aspekter relaterade till djupstabilisering med PVD’s behandlas också, speciellt hur osäkerheter i lerans egenskaper och interaktionen lera-dränkan hanteras. Fördelarna med att nytta observationssmetoden för att hantera dessa osäkerheter diskuteras och hur metoden tillämpades i ett projekt där en järnvägsbank uppfördes på vertikaldränerad sulfidlera presenteras. Resultaten från forskningsprojektet presenteras i en bilagd konferensartikel och två bilagda artiklar skickade till referentgranskade internationella vetenskapliga tidskrifter.

Vid projektering av djupstabilisering med vertikaldräner krävs kunskap om lerans konsolideringsegenskaper och interaktionen mellan dräner och lera. Konsolideringstakten beror primärt av lerans hydrauliska konduktivitet (permeabilitet) i horisontalled. För att kunna utföra en tillförlitlig design krävs därför att denna egenskap bestäms noggrant. Vid konventionella laboratorieförsök bestäms enbart lerans konsolideringsegenskaper i vertikalled, och man är därför ofta tvungen att uppskatta egenskaperna i horisontalled baserat på empiriska korrelationer. Tillförlitliga empiriska kunskaper kring dessa korrelationer är därmed viktigt. För att öka den empiriska kunskapen kring korrelationen mellan vertikal och horisontell hydraulisk konduktivitet och konsolideringskoefficient för sulfidlera, utfördes ett stort antal CRS försök på vertikala och horisontella prover av sulfidlera. Resultaten från försöken visade att egenskaperna i vertikalled och horisontalled skiljer mycket litet och att spridningen i försöksresultaten är stora. För praktiska ändamål föreslås att sulfidlera anses vara isotropt i detta avseende, att parallellförsök bör utföras och att partialkoefficienter introduceras i designproceduren. Hur partialkoefficienter bör hanteras vid design av PVD’s, är ett av de huvudsakliga målen för den fortsatta forskningen inom området. En studie av interaktionen mellan sulfidlera och dräner, dvs. storleken på störningseffekterna vid installationen, utfördes via en parameterstudie där uppmätta portryckstvillingning jämfördes med olika parameteruppsättningar. Studien visade att störningseffekterna påverkade konsolideringstakten men att den naturliga (ostörda) hydrauliska konduktiviteten i sulfidleran hade större inverkan.

En leras odränerade skjuvhållfasthet $c_u$ är beroende av förkonsolideringstrycket $\sigma'_c$. När lera konsoliderar under en last ökar effektivspänningen vilket ofta leder till att det ursprungliga förkonsolideringstrycket och därmed $c_u$ ökar. Relationen mellan $c_u$ och $\sigma'_c$, dvs. kvoten $c_u/\sigma'_c$ för en sulfidlera undersöktes via ett stort antal försök i fält och i laboratorium. Spridningen i mätningarna var stor men för designändamål föreslås $c_u/\sigma'_c = 0.25$ vara representativt i den direkta skjuvzonen för sulfidlera.
List of publications

This thesis is based on the work presented in the following papers:


Paper III Müller R, Larsson S and Westerberg B. Stability for a high embankment founded on sulphide clay. Accepted for publication in *Proceedings of the Institution of Civil Engineers - Geotechnical Engineering*.

In all three papers, the analyses of measurement data, calculations and writing were done by Müller. The co-author Larsson continuously supervised the work, thereby greatly contributing to the final products. The co-author Westerberg provided valuable comments on the manuscript for Paper III and contributed his knowledge of sulphide soils.
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Chapter 1 - Introduction

1.1 Background
Vertical draining of fine-grained soils for ground improvement purposes was first suggested and applied in the US in the late 1920’s (Hansbo 1979). In the early days of the method (the 1930’s), there was some pioneering work done by a Swedish engineer, Walter Kjellman, who patented a type of prefabricated cardboard drain and equipment for its installation (Torstensson 2008). This type of drain is the precursor of all the prefabricated band drains frequently used across the world today. Later on, methods for designing vertical drains were developed by Barron (1948). However, the design of vertical drains today is based mainly on the subsequent, and more practical, simplifications and additions to the theory proposed by Hansbo (1979).

The Swedish contributions to the field of vertical drains have been substantial over the years, but, somewhat surprisingly, the extent of the use of this method in Sweden is rather limited compared to other ground improvement methods applicable to fine-grained soils, e.g. lime-cement columns or embankment piling. Perhaps this is due (in the author’s opinion) to the need for a relatively high degree of empirical knowledge and experience from vertical drains in any particular soil, in order to be able to describe the soil-drain interaction accurately and make a reliable design. Because of the number of assumptions regarding the soil-drain interaction required in the design and the uncertainties in predicting the behaviour of a vertically drained construction, there is always a need for measurements and analyses in the building phase. This could be seen as a potential obstacle or source of complication, which is why vertical drains may seem less attractive than competing methods under certain circumstances. Another less advantageous aspect is time-related. For rational use of vertical drains, there is always a need for time in order for the soil to consolidate. In most projects, a minimum of six to eight months may be required, which is not always available. In many cases, one or two years of consolidation time is used.

Despite the less favourable aspects of the nature of ground improvement using vertical drains, the method has many advantages compared to competing methods. It is indeed one of the longest serving methods available, so the experience gained from projects all over the world is extensive. Despite the measurements needed and the engineering efforts required during the building phase of a construction on vertically drained subsoil, ground improvement by vertical drains is very economically advantageous compared to e.g. lime-cement columns or embankment piling. Vertical drains are also favourable from an environmental point of view, as there is no need for energy-consuming production or transportation of large amounts of bulk material, which is the case for competing methods. The purpose of vertical draining a soil is mainly to improve the properties of the soil itself rather than introducing any foreign material to improve the ground. This, together with gradually increasing knowledge of the drain-soil interaction resulting from the measurements and analyses made during the building phase, offers the geotechnical Engineer a great opportunity for finding creative solutions and the possibility of optimizing the design during the process.
In this thesis, some aspects related to the behaviour of vertical drains in sulphide clay are presented. Vertical drains are especially suitable in this type of soil as excavation should be avoided due to the potential negative environmental aspects and the notion that the function of chemical stabilization methods can be questioned (see Chapter 2). Empirical knowledge related to the performance of vertical drains in a particular soil is vital in order to produce reliable, safe designs without the introduction of high factors of safety. There are, to the author’s knowledge, no previously published studies on this subject for sulphide soils from Sweden or elsewhere in the world. Hence, there is a great need for studying the behaviour of vertical drains in sulphide soils. Furthermore, there is a lack of empirical knowledge about several aspects related to the mechanical behaviour of the sulphide soils. Previous studies have shown that many of the empirical relationships and testing procedures usually adopted for fine-grained soils from other parts of Sweden may not be valid for sulphide soils. In general, there is a great need for directing research efforts to the basic mechanical behaviour of sulphide soils and to studying the functions of different ground improvement methods.

1.2 Outline of the thesis

This thesis consists of an introductory section, one appended paper accepted for publication in a peer-reviewed international journal, one appended paper submitted to a peer-reviewed international journal and one appended conference paper. The main purpose of the introductory section is to present a summary and the conclusions drawn from the research work presented in Papers I-III and to present suggestions for further research in the area of vertical draining in general and of sulphide soils in particular. In addition, Chapters 2, 3 and 4 give the reader a broader context in the areas of sulphide soils, vertical drains and the observational method, thereby providing more background to the appended papers.

In Chapter 2, a brief description of the sulphide soils encountered around the Gulf of Bothnia is given. The description is based on a literature survey which is by no means exhaustive, but it gives the reader an overview of the sulphide soils encountered in Sweden. It considers the origin and occurrence of these soils and explains how they are classified for geotechnical engineering purposes. Some special and characteristic aspects of sulphide soils compared with other types of Swedish fine-grained soils are discussed and the potential negative environmental impacts that can arise when working with sulphide soils are presented.

Chapter 3 gives a compilation of the results from a literature survey of some important areas in the field of ground improvement with vertical drains. The main purpose of the method is given and a brief description of the history of the vertical drainage of soils is presented. The common analytical methods for designing vertical drains and different approaches to how the disturbed zone, i.e. the smear zone, can be treated in the design are described. Furthermore, the effects of drain resistance and how vertical drains can be handled in numerical analyses are treated.

In Chapter 4, the synthesis of the observational method is described. The method is often adopted, more or less rigorously, in projects involving vertical drains. In the appended papers, a high railway embankment, the “Veda embankment” (Fig. 1.1), founded on vertically drained sulphide clay, is the main object of study. How the observational method was adopted in this project is also presented.
In Chapter 5, a summary of the contents and the most important conclusions from the appended papers is given.

In Chapter 6, some aspects of ground improvement with vertical drains are discussed in more general terms, and suggestions for future research in the area are proposed.

1.3 Purpose of the thesis

The purpose of the research presented in this thesis was mainly to increase the knowledge about the behaviour of vertical drains in sulphide soils. In this sense, the characteristics determining the consolidation rate are vital. As the consolidation of vertically drained soils is governed mainly by the flow of pore water in the horizontal direction from the soil towards the drain, hydraulic conductivity in the horizontal direction in the soil is of great importance. If conventional tests for determining hydraulic conductivity and/or the coefficient of consolidation are performed, only the properties in the vertical direction of the soil are assessed. The properties in the horizontal direction are therefore often estimated from measurements of vertical properties via empirical relations. The consolidation rate is also affected by the disturbance effects (smear effects) in the soil, introduced during the installation of the vertical drains. It is very cumbersome to determine these effects in terms of size, shape and degree of disturbance, so one is almost always left to make engineering judgements of the quantities of these parameters. Empirical correlations between the horizontal and vertical hydraulic conductivities and between the horizontal and vertical coefficients of consolidation of sulphide clays were investigated through a series of CRS
Chapter 1 - Introduction

laboratory tests. The results were compared with experiences from other types of clays based on results from previously published studies. The parameters describing the smear zone were also investigated. In Sweden, the design of vertical drains is done according to a design guide for vertical drains (Swedish Road Administration 1989). In the design guide, relations between the vertical and horizontal coefficients of consolidation are proposed and quantities of the smear effects are suggested. The validity of these propositions for sulphide soils has been investigated. The aforementioned aspects are treated in Paper I and Paper II.

The increase in undrained shear strength \( s_u \) that occurs as a result of consolidation of clay is correlated to the increase in preconsolidation pressure \( \sigma'_p \). This increase is important when the stability of a construction is predicted. The increase in \( s_u \) depends on the consolidation rate as discussed previously, and on the ratio \( s_u / \sigma'_p \). Values of this ratio have been presented in numerous studies on different types of clays. In the research presented in this thesis, results from laboratory tests and field tests during construction of the Veda embankment were interpreted in order to investigate what ratio is relevant for the sulphide clay at the site and compared with the suggestions for other types of clays. This is presented in Paper III.

The research presented is aimed at direct applicability for practical engineering purposes in the design of constructions involving vertical drains in sulphide clay and the treatment of uncertainties in various parameters required in the design.

1.4 Research contribution

The research mainly contributes to the empirical knowledge of some aspects of the mechanical behaviour of sulphide clays and of the interaction between vertical drains and sulphide clay.

In Paper II, a methodology on how to assess horizontal hydraulic conductivity and the horizontal coefficient of consolidation from CRS tests on vertical and horizontal samples of a clay is proposed. The use of this methodology renders an opportunity to estimate the anisotropy in hydraulic conductivity and the coefficients of consolidation via rather simple standard laboratory tests. The results from such tests can be used in parallel with empirical relations stated in design codes etc. The empirical relations between the horizontal and vertical hydraulic conductivity and the horizontal and vertical coefficients of consolidation have been studied via a large number of CRS laboratory tests on vertical and horizontal samples of sulphide clays from two areas in north-eastern Sweden. The results from these tests show only small degrees of anisotropies and indicate that they are of the same order of magnitude suggested from previous experience from tests on other types of soft marine clays. The results also indicate quite large scatters in the parameters interpreted from the tests, which makes it difficult to perform a reliable design based on a limited number of tests. Suggestions on how these uncertainties can be treated for design purposes are given. Furthermore, the relationship between the hydraulic conductivity change index and the initial void ratio has been studied for the sulphide clays. It can be concluded that the empirical correlation suggested in studies on other types of clay is also valid for the sulphide clays.

In Paper I, the disturbance effects during installation of vertical drains in the sulphide clay at Veda have been back-calculated based on measurements of dissipation of excess pore pressures. It can be concluded that these effects have a detrimental effect on the function of
vertical drains in the sulphide clay. It is also shown that reliable knowledge of the horizontal hydraulic conductivity is of greater importance compared to how the disturbed zone is modelled.

In Paper III, a methodology on how to use the observational method for predicting the stability of embankment constructions on vertically drained clay in the design phase and control of the stability during the building phase is proposed. Suggestions on how measurements should be made during the building phase and how various uncertainties in the design can be treated are given. In this sense, empirical knowledge of the ratio $s_u/\sigma_p$ is useful, and results from evaluations of the ratio from a large number of in situ and laboratory tests on the sulphide clay at Veda are presented.
Chapter 2 – Sulphide soils in Sweden

2.1 General

About 12.6 million hectares, or approximately 0.1% of the Earth’s entire land surface, is covered with acid sulphate soils. Most of the occurrence of this type of soil is in tropical areas in Asia, Africa and Latin America (Beek et al. 1980). In Europe, acid sulphate soils, or as denoted in this thesis, sulphide soils, are only common in areas around the Gulf of Bothnia in Sweden and Finland.

2.2 Origin and occurrence around the Gulf of Bothnia

Sulphide-rich sediments are formed as organic matter decomposes into fine-grained sediments in a reducing environment. Oxygen-free or low-oxygen water environments can be formed when there is a limited exchange of water. In the waters of the Gulf of Bothnia, these environments were formed after the last deglaciation some 10,000 years ago. After the deglaciation, there was a period of a warmer climate, and large amounts of organic matter were deposited along with fine sediments on the floor of the Baltic Sea, or the Litorina Sea as it is called at that stage. The extension of the Litorina Sea and the present coastal line of Sweden and Finland are shown in Fig. 2.1. In the sediments, the organic matter was decomposed under anaerobic conditions, forming hydrogen sulphide ($\text{H}_2\text{S}$). The sulphide reacted with iron present in the sediments, forming iron monosulphide ($\text{FeS}$) and pyrite ($\text{FeS}_2$). The occurrence of these iron sulphides are common to the sulphide soils found in Sweden and gives these sediments a black colour and a characteristic odour (Georgala 1980). Over the years, the sediments have risen above the sea level and new land has formed as a result of the isostatic land uplift, which has been occurring since the withdrawal of the ice sheet. Today, areas with sulphide soils, commonly known as “svartmocka”, partly cover land areas around the Gulf of Bothnia, see Fig. 2.2. As seen from Figs. 2.1 and 2.2, the occurrence of sulphide soils is more or less restricted to the maximum extension of the Litorina Sea.
2.3 Geotechnical classification and characterization

The content of this section is derived mainly from Larsson et al. (2007).

2.3.1 Classification

Soils with the characteristic black colour from their FeS content that are found in the areas indicated in Fig. 2.2 are typically denoted as sulphide soils. Sulphide soils, like other fine-grained soils, are divided into sulphide clay, silty sulphide clay, clayey sulphide silt and sulphide silt, depending on the grain size distribution. Due to difficulties distinguishing the clay and silt content in this type of soil, this classification is uncertain and the soils are often simply denoted sulphide soil. Classification regarding the organic content of a sulphide soil is done in the same manner as any other soil, i.e. a sulphide soil is denoted organic sulphide soil or sulphide gyttja if the organic content is more than 2% or 6%, respectively. When characterizing sulphide soils, the total sulphur content, total iron content and organic content in relation to total dry weight of the soil are usually determined. Typical ranges for properties of the Bothnian sulphide soils are total sulphur content of 0.1-2%, total iron content of 2-5%, and organic content of 2-7% (Westerberg and Andersson 2009).

In Fig. 2.3, the microstructure of a silty sulphide clay, with an organic content of 1.5%, clay content of 27% and liquid limit of 75%, originating from an area in Luleå is presented (Larsson 1990). The grain matrix is constructed mainly from silt particles with clay particles attached to the surface. No visible fibres or other organic substances can be observed. In some of the pictures, relatively large embedded spherical particle aggregates are seen. X-ray diffraction showed that these aggregates contain large amounts of sulphur and iron. In a study by Pusch (1973), similar aggregates were found, presumably originating from organic matter or organic activity.
2.3.2 Sampling and handling of samples

Conventional sampling methods are usually employed for retrieving specimens of sulphide soils, i.e. some sort of auger sampler for “disturbed sampling” or piston sampler for “undisturbed sampling” (SIS 2007). From auger samples, visual distinguishing of different material and layers can be difficult due to the black colour of the sulphide soil. This can cause difficulty when choosing representative samples for characterization of the soil. Samples of sulphide soils are very sensitive to contact with air as the soil oxidizes after only a short time when subjected to air, so that the chemical and physical properties of the soil may change. Samples should be stored in a place with 100% relative humidity and at a temperature corresponding to the temperature in the soil where the samples were retrieved. Long storage times should be avoided.

2.3.3 Evaluation of laboratory tests and field tests

The FeS and FeS$_2$ content in sulphide soils affects their geotechnical properties. The geotechnical testing methods usually adopted for investigating and evaluating the properties of fine-grained soils in Sweden are based on experiences from soils that are common in other parts of Sweden. Such experience may therefore not be directly applicable to sulphide soils.
Regarding compression tests, e.g. incremental or CRS oedometer tests, studies by Eriksson (1989, 1992) show a great influence of the loading rate and temperature on the results from oedometer tests. Furthermore, the usually good agreement between pre-consolidation pressure $\sigma'_p$ evaluated via Casagrande’s method from incremental oedometer tests and $\sigma'_p$ evaluated from CRS oedometer tests according to Sällfors (1975) may not always be valid for sulphide soils.

The empirically established correction factors (commonly used for fine-grained soils) for deriving characteristic values of the undrained shear strength $s_u$ from measurements via fall cone tests or vane shear tests $\tau_u$ are based on the liquid limit. These are not applicable to sulphide soils. In the study by Larsson et al. (2007), they suggest the following relation with a correction factor of 0.65 independent of the liquid limit

$$s_u = \tau_u \cdot 0.65$$  \hspace{1cm} (2.1)

They also suggest that evaluations of $s_u$ based on the results from CPT testing should be made according to

$$s_u = \frac{q_t - \sigma_{vo}}{20} \left( \frac{OCR}{1.3} \right)^{-0.2}$$  \hspace{1cm} (2.2)

where $q_t$ is the total pressure measured with the CPT probe, $\sigma_{vo}$ is the total stress in the soil and $OCR$ is the over consolidation ratio.

### 2.3.4 Environmental aspects

If a sulphide soil is exposed to oxygen, e.g. if it is excavated or if the groundwater table is lowered as a result of ditching or draining, sulphate ions, hydrogen ions, iron ions and other metal ions (mainly cadmium, nickel, manganese, cobalt and zinc) that are common in the soil are mobilized and pH is lowered (Sohlenius and Öborn 2004; Sohlenius et al. 2009). This could lead to negative environmental impacts and harm the water environment and land-living organisms. In a study by Pousette (2007), a guideline for classifying sulphide soils according to their acidification potential is proposed. A sulphide soil should be classified as potentially acidifying if the total amount of sulphur is higher than 0.06% of dry substance and the ratio Fe/S is lower than 60. Furthermore, the content of iron sulphides may affect the possibility of chemically stabilizing sulphide soils and may also affect constructions sensitive to corrosion (Larsson et al. 2007).
Chapter 3 – Ground improvement by vertical drains

In this chapter, a literature survey concerning different aspects in the field of vertical draining of soil is presented. The text is focused predominantly on prefabricated vertical band drains (PVD’s) as this type of drain is most commonly used today. Some comments, remarks and conclusions made by the author are also incorporated in the text.

3.1 Purpose

Vertical drains in combination with a temporary surcharge is a ground improvement method used in fine-grained soils (mainly clays) with low hydraulic conductivity (permeability). The introduction of vertical drains shorten the distance between the free-draining boundaries in the clay mass and thereby reduce the consolidation time, i.e. increase the drainage rate of excess pore pressures induced in the clay when subjected to an external loading. As the clay consolidates under a load, the water content and void ratio decrease, leading to compression (settlement) and improvement of the clay, e.g. increased strength and stiffness. In a natural unimproved clay deposit, the drainage boundaries are mainly horizontal. The drainage boundaries are the ground surface and, if the clay overlies soil with a significantly higher hydraulic conductivity (e.g. sand), the bottom of the deposit. As a natural clay is often stratified with layers of more permeable soil, these layers can sometimes also serve as boundaries provided that they can drain the excess pore pressures freely. When vertical drains are introduced in the clay, they serve as vertical drainage boundaries, leading to a reduction of the drainage length from perhaps several metres or even tens of metres to a couple of decimetres, thus speeding up the consolidation process (Fig. 3.1). If vertical draining is combined with a temporary preloading (surcharge), the clay is additionally improved and development of secondary deformation (creep) can be significantly reduced.

Fig 3.1 Sketch of the drainage boundaries in a clay; a) without PVD’s; b) with PVD’s
3.2 Brief history

Vertical drains as a soil improvement method were first introduced in 1925 by an American, Daniel E. Moran (Hansbo 1977). In 1926 he received a patent for a method of installing vertical sand columns and also suggested the first application, the stabilization of mud soil beneath an embankment in California. However, sand columns were first used on a larger scale in 1934. In the late 1930’s the former director of the Swedish Geotechnical Institute, Walter Kjellman, introduced the first prefabricated vertical drain type on the market, the “cardboard wick” (Fig. 3.2). Ever since these pioneering works, the method has been used all over the world and is today one of the most common methods for improving soft fine-grained soils. Over the years, the installation methods, machinery, equipment and drain material have gradually been improved. In the beginning, sand drains were used most frequently but since the late 1970’s PVD’s have surpassed the former and are now applied almost exclusively.

A vast number of thoroughly documented and analysed case studies of embankments or fills intended for reclamation of land on soft fine-grained soils improved by vertical drains have been published, e.g. Hansbo et al. (1981); Bergado et al. (1991, 2002); Hird et al. (1995); Indraratna et al. (1997, 2003); Eriksson et al. (2000); Chai et al. (2001); Arulrajah et al. (2005); Shen et al. (2005); Bo et al. (2007); Larsson (2006); Rujikiatkamjorn et al. (2007); Lo et al. (2008); van Helden et al. (2008); Müller and Larsson (2008); Walker and Indraratna (2009).

Over the years, the original theories on the design of vertical drains developed by Barron (1948) and Kjellman (1949) have gradually been improved. This is discussed in more detail in Section 3.4.1.

The techniques of vertically draining a soil have also developed since the concept was introduced in the 1920’s. For instance, in order to reduce the required surcharge, the use of vacuum preloading (Bergado et al. 1998; Rujikiatkamjorn et al. 2007) and electro osmosis (Bergado et al. 2000) have been investigated in full-scale projects and laboratory studies, respectively. Another possible future improvement of the PVD technique is the combination of vertical drains and heating. According to Abuel-Naga et al. (2006), several studies have shown development of irreversible volume changes and improved properties (i.e. increased stiffness, strength and hydraulic conductivity) due to raised temperatures in clay and argue that heat could be combined with vertical drains.

Fig. 3.2 Kjellman’s “cardboard wick” drain; from Hansbo (1979)
3.3 Material and installation

There are several commercially available types of PVD’s on the market, principally composed of a plastic core surrounded by a geotextile sleeve. The drains typically have cross-sectional dimensions of 100 mm x 4 mm (Holtz 1987). Some examples are presented in Fig. 3.3. The drains are installed by using a drain rig in a square or triangular pattern with drain spacing of about 1.0 to 3.0 metres (Holtz 1987). As the drains are not very rigid, a mandrel is used during installation in order to steer the drains vertically through the soil (Fig. 3.4). As the mandrel is pushed downwards, the soil is displaced, thus creating a disturbed soil volume around the drain, the smear zone. When the intended depth is reached, the mandrel is withdrawn and the drain is held in place by an anchor plate. The installation process is schematically illustrated in Fig. 3.5. Some pictures from the installation of PVD’s at Njutånger, near the town of Hudiksvall, Sweden, are shown in Fig. 3.6.

![Fig. 3.3 Typical PVD’s; a) Membra drain; b) Layfield; c) Ce Teau](image)

![Fig. 3.4 Typical section of a diamond-shaped mandrel; from Holtz et al. (1991a)](image)

![Fig. 3.5 Installation process of a PVD; from Stapelfeldt (2006)](image)
3.4 Design

3.4.1 Analytical solutions

Conventional analytical calculations for designing vertical drains are generally based on the solution of the basic partial differential equation for consolidation by three-directional flow in soils presented by Barron (1948). In the theory, a soil volume dewatered by one drain is represented by a cylinder concentric to the drain (Fig. 3.7). The soil cylinder is assumed to have a cross-sectional area equal to the soil volume it drains (Fig. 3.8).
Barron’s general solution is rather complicated and can for practical purposes be simplified without losing any significant accuracy (Kjellman 1948; Hansbo 1977; Onoue 1988; Hird et al. 1992).

A simple and, for practical purposes, relevant solution for calculating the average degree of consolidation due to horizontal drainage \( \bar{U}_h \) was proposed by Hansbo (1979)

\[
\bar{U}_h = 1 - e^{-\frac{8 c_h t}{D^2 F}} \tag{3.1}
\]

where \( t \) is the time of consolidation; \( D \) is the equivalent diameter of the soil cylinder (Figs. 3.7 and 3.8) and \( c_h \) is the coefficient of consolidation due to radial drainage defined as

\[
c_h = \frac{k_h \cdot M_v}{\gamma_w} \tag{3.2}
\]

where \( k_h \) is the horizontal hydraulic conductivity of the undisturbed soil; \( M_v \) is the oedometer modulus in the vertical direction; \( \gamma_w \) is the unit weight of water.

\( F \) is defined as

\[
F = F_n + F_s + F_w \tag{3.3}
\]

where \( F_n = \ln(n) - 0.75 \); \( F_s = \left(\frac{k_s}{k_z} - 1\right) \ln\left(\frac{d_z}{d}\right) \); and \( F_w = \pi \cdot z \cdot (2 \cdot l - z) \frac{k_z}{q_w} \). These three terms account for the drain spacing, the smear effects and the well resistance, respectively. In Eq. 3.3, \( n = D/d \); \( d \) and \( d_z \) are the equivalent drain diameter and diameter of the disturbed zone, respectively (see Fig. 3.7); \( k_s \) is the horizontal hydraulic conductivity in the disturbed smear zone of the soil; \( z \) is the depth below ground surface; \( l \) is one half of the drain length.
for double drainage or the drain length for single drainage; and \( q_w \) is the discharge capacity of the drain.

The most commonly used formula for calculating the equivalent drain diameter is (Hansbo 1979)

\[
d = \frac{2(a + b)}{\pi}
\]

(3.4)

where \( a \) and \( b \) are the width and thickness of the drain, respectively (Fig. 3.4).

Hansbo’s solution, perhaps the most common formulation for designing PVD’s, is relevant when the drain spacing is much larger than the equivalent drain diameter. If \( n < 10 \), the more rigorous solution proposed by Barron (1948) should be employed. Fortunately, this is seldom the case in projects involving PVD’s but might be relevant for sand column drains installed at close spacing. In addition, Hansbo’s solution regards horizontal drainage only. If consolidation due to vertical drainage can not be ignored, i.e. if the drain spacing is of the same order as the distances between the horizontal drainage boundaries (e.g. limited clay thickness or clay with incorporated permeable layers), Carillo’s equation (Carillo 1942) can be adopted

\[
\bar{U} = U_h + U_v - U_h \cdot U_v
\]

(3.5)

where \( \bar{U} \) is the average degree of consolidation combining the effects of horizontal drainage \( U_h \) and vertical drainage \( U_v \). The latter is calculated by solving Terzaghi’s equation for vertical consolidation (a practical solution to the equation is presented in any textbook on soil mechanics, e.g. Lambe and Whitman 1979).

Furthermore, both Barron’s and Hansbo’s solutions are based on small strain theory and on Terzaghi’s theory of consolidation and are therefore restricted by the same simplifications and shortcomings. However, as soft clay soils may deform considerably as a result of a loading, neither the stiffness \( (M_v) \) nor the hydraulic conductivity \( (k_h) \) and as a consequence \( c_h \) are constant or can be assumed to be constant for larger stress intervals. Design based on Barron’s or Hansbo’s theories therefore implies choosing a fixed value of \( c_h \) representing a certain expected stress interval without regard to the change due to compression.

Over the years, several improvements of these theories have been proposed for solving some of the aforementioned shortcomings. For example, Runesson et al. (1985) presented a solution for situations where the drains only partially penetrate the clay layer. A consolidation equation valid for both Darcian and non-Darcian flow was introduced by Hansbo (1997b, 2001). A simple method considering both vertical and horizontal flow and the presence of PVD’s via an equivalent vertical hydraulic conductivity was presented by Chai et al. (2001). In a paper by Leo (2004), a closed form solution for coupled horizontal and vertical drainage was derived. Indraratna et al. (2005) published a theory based on Hansbo’s solution (Eq. 3.1) which is able to account for the change in stiffness and hydraulic conductivity due to compression. Furthermore, as the Barron and Hansbo theories assume the loading is applied instantaneously, development of methods accounting for a gradually increasing load has been proposed (Walker and Indraratna 2009; Conte and Troncone 2009).
As shown, none of the aforementioned proposed improvements solves for all the shortcomings of the Barron or Hansbo theories. Hence, if analytical calculations are to be made, we are still left to make a series of assumptions and simplifications. However, the use of computer programs for numerical analyses with more general and rigorous theoretical formulations for consolidation calculations reduces the necessary simplification. This is discussed in more detail in Section 3.4.4.

3.4.2 The disturbed “smear” zone

Many factors contribute to the size and shape of the disturbed smear zone. Most significant are factors related to the soil fabric, the size and shape of the mandrel and the installation process (e.g. Holtz et al. 1991b; Hird and Moseley 2000). The mandrels are usually circular, square-shaped, rectangular or diamond-shaped, as in Fig. 3.4. In the formulations of the equations for designing vertical drains by Barron or Hansbo, the smear zone is generalized as a concentric cylinder around the drain with the assumption of a constant value of $k_s$. This may be the case for circular or square mandrels, but the smear zone is better represented by an ellipse for rectangular or diamond-shaped mandrels (Chai and Miura 1999). Many researchers have studied the degree of disturbance, i.e. the ratio $k_s / k_h$. Values of the ratio between 1.5 and 10 for different types of clays, installation procedures and mandrel shapes etc. are usually suggested, e.g. Bergado et al. (1991, 1992); Hansbo (1986, 1987, 1997a); Hird and Moseley (2000). With the assumption of a constant value of $k_s$ within the smear zone, its extent can be approximated at 2-4 times the equivalent diameter of the mandrel, i.e. $d_s / d_m \approx 2-4$ (Holtz and Holm 1973; Jamiolkowski et al. 1983; Hansbo 1986, 1997a; Bergado 1991, 1992; Mesri et al. 1994; Hird and Moseley 2000).

Experimental investigations, however, have shown that the assumption of a constant value of $k_s / k_h$ in the smear zone is not valid. Onoue et al. (1991a) presented results from large-scale oedometer tests with radial drainage on reconsolidated Boston Blue Clay. During compression, simultaneous measurements of pore pressures were made at different distances to the central drain and subsequently $k_s / k_h$ were evaluated at different distances from the drain (Fig. 3.9).
They suggested a division of the soil into three zones (Fig. 3.10):

Zone I: the undisturbed zone. For the clay tested, $d_s/d_m = 6.5$ was suggested.

Zone II: a zone where the drain installation causes a decrease in void ratio and hence hydraulic conductivity.

Zone III: the remoulded zone with a reduction in horizontal hydraulic conductivity due to remoulding in addition to the decrease in void ratio.

In another study, measurements of hydraulic conductivity on samples of Ariake clay (Japan) retrieved at different distances from drains installed in the field are presented (Madhav et al. 1993). The results are shown in Fig. 3.11. They propose a division of the disturbed zone into an inner smear zone and an outer (transition) zone; $d_s/d_m = 12$ was suggested.
A different testing technique used to measure the extent and hydraulic conductivity reduction in the smear zone was adopted in a study by Indraratna and Redana (1998). Reconstituted alluvial Sydney clay (Australia) was consolidated to a low (20 kPa) preconsolidation pressure in a large-scale oedometer (Fig. 3.12).

Fig. 3.12 Large-scale oedometer with a centrally installed drain from which the oedometer samples were retrieved; a) cross section; b) seen from above; from Indraratna and Redana (1998)
Thereafter a sand drain was installed centrally in the oedometer, and samples were collected vertically and horizontally. The samples were placed in conventional oedometers and incrementally loaded. The hydraulic conductivities in the vertical and horizontal directions were evaluated at different distances from the drain and at different degrees of compression. Results from the hydraulic conductivity measurements are shown in Fig. 3.13 together with results from similar tests performed on reconsolidated Singapore kaolin clay presented by Sharma and Xiao (2000).

![Fig. 3.13 Evaluations of hydraulic conductivities; a) Indraratna and Redana (1998); b) Sharma and Xiao (2000)](image)

Both these tests confirm the existence of a transition zone with a successively decreasing hydraulic conductivity as the remoulded zone in the vicinity of the drain is approached. They also suggested $d_x/d_m = 5$ and 4 for the Sydney and Singapore clays, respectively. Another
important aspect regarding the smear zone highlighted in these studies is the reconsolidation effect. Studying Fig 3.13, it can be seen that the effect of disturbance is reduced significantly after 3-4 load increments (i.e. as the effective stress is tripled or quadrupled). This implies that the smear effects decrease during consolidation, which could have the effect of significantly improving the performance of vertical drains in practical engineering projects.

The results from the four studies mentioned do not indicate any obvious conclusions regarding either the degree of disturbance ($k_s/k_h \approx 1.5$ to 5) or the extent of the smear zone ($d_s/d_m \approx 4$ to 12), or the distribution of the hydraulic conductivity within the zone. However, the assumption of a constant degree of disturbance in the smear zone (c.f. Eq. 3.3) seems invalid and may lead to predictions of the consolidation rate that are not conservative (Basu et al. 2010). Basu et al. (2006) presents relatively simple and practically applicable analytical solutions for four different distributions of the hydraulic conductivity in the disturbed zone.

The equation for calculating $F$ (ignoring the influence of $F_0$) for use in Hansbo’s equation (Eq. 3.3) for one of the solutions, assuming a constant value of $k_s$ in the remoulded zone and a linearly increasing hydraulic conductivity in the transition zone (Fig. 3.14), is

$$F = \ln\left(\frac{n}{q}\right) + \frac{1}{\beta} \ln(m) + \frac{(q - m)}{(\beta \cdot q - m)} \ln\left(\frac{\beta \cdot q}{m}\right) - 0.75$$

(3.6)

where, $n = D/d$; $q = d_v/d$; $\beta = k_s/k_h$; $m = d_{uw}/d$; and with $D$, $d$, $d_v$, and $d_{uw}$ according to Fig. 3.14.

In reality, however, the extent of the smear zone is very difficult to assess even if extensive laboratory or field tests are performed. The characteristics of a transition zone are even harder to determine. In a study by Basu et al. (2010), an equivalent single smear zone is proposed with a larger extent than a smear zone that does not take the effect of the transition zone into account.

Fig. 3.14 The smear zone and transition zone according to Basu et al. (2006)
3.4.3 Effects of well resistance

The detrimental effect on the consolidation rate due to resistance to flow in sand column drains was discussed as early as Barron (1948). In the formulation by Hansbo (1979), this is taken into account via the well resistance factor $F_w$. As shown, the maximum effect of well resistance occurs in the middle of the vertically drained soil deposit for double drainage or at the bottom if drainage only occurs at the top, i.e. as $z = l$ (c.f. Fig. 3.7). The relative effect of well resistance compared with the effects of drain distance $F_d$ and smear $F_s$ depends on $k_s$ in relation to $q_u$. The relative effects of smear and well resistance for a few simple cases are presented in Table 3.1.

Table 3.1. Hansbo (1981); Effect of smear and well resistance on overall average consolidation degree achieved by vertical drains only

<table>
<thead>
<tr>
<th>Time of consolidation years</th>
<th>Sand drains*</th>
<th>Prefab drain*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(1)</td>
<td>(2)</td>
</tr>
<tr>
<td>0.25</td>
<td>33</td>
<td>34</td>
</tr>
<tr>
<td>0.5</td>
<td>55</td>
<td>56</td>
</tr>
<tr>
<td>1</td>
<td>80</td>
<td>80</td>
</tr>
<tr>
<td>2</td>
<td>96</td>
<td>96</td>
</tr>
</tbody>
</table>

* (1) no smear; no well resistance  
(2) smear; no well resistance  
(3a) smear and well resistance; drain length = 20 m  
(3b) smear and well resistance; drain length = 30 m

For sand column drains, the discharge capacity can be calculated as $q_u = k_w \cdot A_w$ where $k_w$ and $A_w$ are the hydraulic conductivity and cross-sectional area of the columns. According to Hansbo (1987), $q_u$ of sand column drains depends mainly on the grain size distribution of the sand and the possible clogging effect from intrusion of silt and clay particles as pore water enters the drain.

![Fig. 3.15 Results of discharge capacity tests for different drain makes; from Hansbo (1986)](image-url)
Regarding $q_w$ of PVD’s, Hansbo (1981, 1986 and 1987) argues that the discharge capacity is very much dependent on the lateral stresses against the drains imposed by the soil. A series of laboratory tests of some of the drains available on the market at that time are shown in Fig. 3.15 (from Hansbo 1986). It is clear that $q_w$ decreases as the lateral pressure increases. The reason for this is either that the filter sleeve is pressed into the drain channel or that the drain channels themselves are compressed (Hansbo 1987). Another possible detrimental effect on $q_w$ is the potential effect of folding the drain as the soil is compressed under a load. In a study by Holtz et al. (1991b), a vast number of drain brands were tested in a laboratory in order to evaluate $q_w$ of the drains at different lateral pressures. Some of the drains were also tested after first being compressed by 20% and as a result the drains were markedly folded, two examples of drains extruded after the tests are shown in Fig. 3.16. From the results of this study, most of the brands tested had $q_w > 150 \text{ m}^3/\text{year}$ even after folding and substantial lateral pressure.

In a study by Mesri and Lo (1991), a well resistance factor was introduced, $R = q_w / (k_h \cdot l^2)$. They also presented analyses of three embankment construction projects with respect to the influence of $R$ on the time required for 95% consolidation compared to ideal drain conditions (assuming $q_w = 1000 \text{ m}^3/\text{year}$). The results from the analyses indicated a threshold value of $R = 5$, above which well resistance can be ignored.
This implies that for many practical purposes concerning clay, i.e. with \( l < 10 \) metres and \( k_h < 10^{-8} \) m/s, the required \( q_w \) is about 150 m\(^3\)/year and well resistance can therefore be ignored. However, if the clay deposit is larger, if the soil has a higher hydraulic conductivity or if the PVD installed has very low discharge capacity, the influence of well resistance may need to be incorporated in the analysis.

Another important aspect is the filter criteria. Hansbo (1981) argues that very low hydraulic conductivities of filters can be used as the thickness of the filters is very small, and proposes that \( k_{\text{filter}} \approx 0.03 \text{ to } 0.1 \text{ m/year} (1 \text{ to } 3 \cdot 10^{-9} \text{ m/s}) \) is sufficient. A low hydraulic conductivity, hence dense structure of the filter, is preferable as it will reduce the intrusion of fine grains into the drain channel, thus reducing the risk of clogging.

### 3.4.4 Numerical analysis

In the last few decades, the use of numerical analysis for the design of projects involving vertical drains and for research purposes has increased dramatically. The analytical solution for consolidation under radial drainage (i.e. vertical drain design) according to Eq. 3.1, involves analysis of a unit cell under simplified boundary conditions (Fig. 3.7). This theory is quite easy to use and is therefore suitable for “hand calculations” or “spreadsheet calculations”. However, it is based on Terzaghi’s theory of consolidation and therefore bound to the same simplifications and shortcomings. As pointed out by e.g. Muir Wood (2004, p 345), this formulation is underpinned by a series of assumptions regarding material characteristics and boundary conditions: incompressibility of pore fluid and soil particles; validity of Darcy’s law; constant stiffness and hydraulic conductivity; one-dimensional deformation and flow. In addition, the theory involves a solution of the equations for consolidation rate (Eq. 1) and for the magnitude of compression due to consolidation (settlement) explicitly. Design based on the analytical solutions implies choosing a fixed value of \( c_h \) representing a certain expected stress interval without regard to the change due to compression.

In more rigorous solutions for solving problems involving consolidation, simultaneous solving for the consolidation rate and the magnitude of compression is performed. In software for numerical calculations, Biot’s equations for consolidation (Biot 1941) are usually implemented and coupled with material constitutive models, resulting in the basic equations. As argued by Hird et al. (1992), the most rigorous and mathematically most complex solutions are based on Biot’s consolidation theory or the further developments by Yoshikuni and Nakanodo (1974) or Onoue (1988). A detailed description of the derivation of Biot’s equations is given in Potts and Zdravkovic’ (1999, pp 305-324). The coupling allows simultaneous solving of the consolidation rate and the magnitude of compression, and the changes in stiffness and hydraulic conductivity due to compression can be modelled. As a result, real soil behaviour can be described more closely than with use of the analytical solutions.

In a study by Hird et al. (1992), calculations using a finite element program equipped with Biot’s theory were compared with conventional analytical calculations using Hansbo’s theory. The degree of consolidation at various distances from the drainage boundaries of a unit cell under axisymmetric condition was studied. A comparison with some analytical parametric studies presented in Jamiołkowski et al. (1983) was also made. All comparisons under these axisymmetric conditions were excellent.
However, consolidation analyses for e.g. embankments on soft ground are preferably done under the assumption of plane strain conditions. As noted by Hird et al. (1992), the flow pattern towards a drain is of an axisymmetric nature, thus some sort of matching procedure, i.e. translation, is required (Fig. 3.17). They proposed two different methods for doing this, either by a geometrical matching procedure (Eqs. 3.7a and 3.7b)

\[ B = D \cdot \sqrt{\frac{3}{2} \cdot \left[ \ln\left(\frac{n}{s}\right) + \left(\frac{k_s}{k_h}\right) \ln(s) - \frac{3}{4} \right]} \]  

(3.7a)

\[ Q_w = \left(\frac{4B}{\pi D^2}\right) \cdot q_w \]  

(3.7b)

or by a hydraulic conductivity matching procedure (Eqs. 3.8a and 3.8b), both incorporating effects of the smear zone

\[ k_{ps} = 2 \cdot k_s \cdot \left[ \frac{3}{\ln\left(\frac{n}{s}\right)} + \left(\frac{k_s}{k_h}\right) \ln(s) - \frac{3}{4} \right] \]  

(3.8a)

\[ Q_w = \left(\frac{4}{\pi D}\right) \cdot q_w \]  

(3.8b)

where \( B \) is the width of the plane strain unit cell; \( s = d_s / d \); \( Q_w \) is the plane strain discharge capacity of the drain and \( k_{ps} \) = the horizontal hydraulic conductivity in plane strain.

In the study, they conclude that the maximum deviation in the calculated degree of consolidation by axisymmetric and corresponding plane strain analyses is less than 11%. The validity of the matching procedures was later investigated (Hird et al. 1995) in a comparison with three embankment case histories. The matching procedure seemed valid for these reasonable wide ranges of field conditions.

Fig. 3.17 Unit cells; a) axisymmetric; b) plane strain; from Hird et al. (1995)
However, they also note that the procedure may be invalid if flow in the vertical direction is significant, e.g. in soils with seams of more permeable material at distances of similar magnitude to the distances between the drains.

The matching procedure proposed by Hird et al. (1992) has been adopted in analyses of constructions involving PVD’s presented in e.g. Chai and Miura (1999) and Arulrajah et al. (2005) and in the study presented in Paper I in this thesis.

In numerical software, the drains can be modelled either as pore pressure boundaries, i.e. by vertical boundaries introduced in the numerical model at which the excess pore pressure is always zero, or as very thin soil elements with a representative hydraulic conductivity. The first procedure is valid for cases where the well resistance can be ignored, whereas the second procedure is of a more general nature. However, the very thin elements introduced by adopting the latter procedure may lead to numerical difficulties.
Chapter 4 – The observational method

In this chapter, a brief description is given of the origin and main ingredients of the observational method as presented by Peck (1969) in his famous Ninth Rankine Lecture. In addition, the design and construction procedures adopted in the Veda embankment project are presented. In this project, which is the main object of study in the appended papers, a trial embankment was built and the observational method was used.

4.1 General description

According to Peck (1969), engineers working in the field of soil mechanics owe Karl Terzaghi for the formulation of the observational method and its development into a systematic design procedure. Peck states that, as Peck and Terzaghi worked on their book *Soil Mechanics in Engineering Practise* in 1945, Terzaghi formulated the basics of the observational method for the first time. In the context of all the uncertainties facing an engineer working in the fields of soil and rock mechanics, Terzaghi saw only two methods for coping with them at that time, either by introducing excessive factors of safety or by making assumptions based on the general, typical experience. In one draft of the introduction to the book, Terzaghi considered the first method “wasteful” and the second “dangerous”. He therefore postulated a third method, the “experimental method”, in which the design was to be based on whatever information could be secured. Potential differences between reality and the assumptions made would then be inventoried and measurable quantities computed. During the construction process, the computations would be compared with measurements, gradually closing the gaps in knowledge and constituting a basis for necessary modifications of the design.

In Peck (1969), it is stipulated that the observational method “…often permits maximum economy and assurance of safety, provided the design can be modified as construction progresses.”

Peck (1969) summarized the following eight ingredients for applying the observational method:

1. Exploration sufficient to establish at least the general nature, pattern and properties of the deposits, but not necessarily in detail.
2. Assessment of the most probable conditions and the most unfavourable conceivable deviations from these conditions. In this assessment geology often plays a major rôle.
3. Establishment of the design based on a working hypothesis of behaviour anticipated under the most probable conditions.
4. Selection of quantities to be observed as construction proceeds and calculation of their anticipated values on the basis of the working hypothesis.
5. Calculation of values of the same quantities under the most unfavourable conditions compatible with the available data concerning the subsurface conditions.
6. Selection in advance of a course of action or modification of design for every foreseeable significant deviation of the observational findings from those predicted on the basis of the working hypothesis.

7. Measurement of quantities to be observed and evaluation of actual conditions.

8. Modification of design to suit actual conditions.

This design methodology should not be adopted if the nature of a project is such that alteration of the design cannot be made. Otherwise the methodology has great potential for a safe, economically favourable and time-saving design.

The Eurocode for Geotechnical Design, EN 1997-1 (CEN 2004), states: “When prediction of geotechnical behaviour is difficult, it can be appropriate to apply the approach known as the ‘observational method’, in which the design is reviewed during construction.”

The observational method is one of the four applicable design methods in the code and the requirements for using the observational method are based on the eight ingredients in Peck (1969).

The observational method is highly suitable for the design of constructions involving vertical drains in clay as there are, indeed, many uncertainties in the behaviour of the clay itself and the governing features regarding the interaction of the clay and the drains. Furthermore, the most important quantities that determine the behaviour of a construction on vertically drained clay can readily be predicted in the design phase and measured in the building phase. In a project involving vertical drains, these quantities are:

- the deformations in the subsoil
- the development of excess pore pressures in the subsoil
- the shear strength increase in the subsoil as a result of consolidation

In Papers I and III appended to this thesis, the application of the observational method in a project involving an embankment on vertically drained sulphide clay is presented. Application of the observational method in projects involving vertical drains was earlier reported by e.g. Choa (1994), Tan and Chew (1996), Eriksson (1997), Arulrajah et al. (2004, 2005, 2008) and Torstensson (2008).

4.2 Observational method adopted in the design of the Veda embankment

4.2.1 General

The Veda embankment was built to navigate across an approximately 300 m wide valley situated near to the river Ångermanälven, 55 kilometres north of the city of Sundsvall in Sweden. A length section of the embankment is shown in Fig. 4.1. This section is part of the Ådalsbanan railway line, which will connect to the new Botnia Line, scheduled to open in 2011. The embankment is situated close the High Coast bridge, from which the photo in Fig. 1.1 was taken, prior to the removal of the surcharge preloading. The subsoil strata in the valley generally consist of soft sediments of silt, clay (partly sulphide clay) and sand. The embankment was made of crushed rock fill (mainly granite and gneiss) derived from a nearby tunnelling project, and rises to a height of 16 m above the original ground surface. Further
information regarding the construction and geotechnical conditions can be found in the appended papers.

The embankment load imposes a considerable stress increase in the soil, which in turn leads to deformation and stability problems. In order to address the stability issues, support berms consisting of moraine fill were constructed and the embankment was built using staged construction, making it possible to account for the increase in undrained shear strength $s_u$ in the clay due to consolidation effects. Prefabricated vertical drains (PVD’s) were installed in order to accelerate consolidation. This method for improving the ground was preferred to other methods such as full excavation of the soft soils, installation of lime-cement columns or a pile foundation. The possibility of full excavation was rejected mainly due to the occurrence of sulphide clay in the subsoil. When sulphide soils are excavated and disposed of, the sulphide content in the soil may lead to negative environmental impacts, such as acidification of water areas and leaching of heavy metals. This is further discussed in Section 2.3.4.

Ground improvement via lime-cement columns was also rejected, due mainly to the substantial stresses imposed by the embankment loading compared with the compression strength of the columns. If this method had been chosen, the columns would have had to be installed at a very close spacing and consequently at a very high cost. The function and applicability of lime-cement columns in sulphide soils can also be questioned given uncertainties regarding how the sulphide content in the soil will affect the lime-cement-clay interaction. A pile foundation was not considered because of the high costs involved.

As experience with embankments of this particular magnitude on sulphide clay was limited, uncertainty about strength, deformation and consolidation characteristics of the subsoil prevailed in the design phase. Generally, pilot tests are recommended given the complexity and difficulty in predicting the parameters required to make an accurate design of ground improvement works involving PVD’s (Hansbo 1997a). In the present case, a trial embankment was built, and the design and construction processes were carried out using the observational method.

![Fig. 4.1. Length section through centre of embankment (from Paper III)](image)
4.2.2 The trial embankment

The trial embankment was 50 metres long and 6.7 metres high and built and instrumented in December 2005, one year before the start of construction on the main embankment. Fig. 4.2 shows photos of the trial embankment and the instrumentation.

Fig. 4.2 The Veda trial embankment; a) overview; b) measurement instruments
Photos H Henriksson
Chapter 4 – The observational method

The trial embankment is described in detail and results from the measurements are presented in Papers I and III.

Construction of the trial embankment served the purpose of increasing understanding of the interaction between the PVD’s and the sulphide clay and of the strength and deformation properties in the sulphide clay. In Paper III, measurements of the increase in undrained shear strength $s_u$ and the ratio $s_u/\sigma_p'$ between $s_u$ and the preconsolidation pressure $\sigma_p'$ during consolidation are presented and discussed. Measurements from the trial embankment were also used to calibrate a finite element model for use in the design of the Veda embankment, as described in Paper I. Furthermore, the trial embankment enabled a testing of the equipment and techniques proposed for measurements of the planned Veda embankment.

4.2.3 The Veda embankment

The construction works for the Veda embankment began in the autumn of 2006. Fig. 4.3 shows a picture taken during installation of the vertical drains and a picture during filling works. A layout plan and a typical section of the embankment are presented in Fig. 9 of Paper III. The staged construction sequence and measured excess pore pressures at different levels in the ground are presented in Fig. 8 in Paper III.

Below is a description of how the observational method was adopted in this case.

1. In order to fulfil the first ingredient of Peck’s methodology, site investigations in the area consisting of probing, sampling and pore pressure measurements were made. Laboratory tests on “undisturbed” samples of clay taken from various depths at 6 locations were also carried out.

2. The results from these tests were summarized and interpreted in order to produce a picture of the ground conditions and some basic material parameters of the soil layers in the ground (the most probable conditions). The knowledge gained from the trial embankment was also incorporated. More detailed descriptions are given in Paper III.

3. A soil model intended for preliminary design of the embankment was established based on interpretations of the site investigation. A design of the ground improvement method was then performed and predictions of the degrees of consolidation were made at different stages of the construction process. The stability of the construction was controlled via calculations at the various stages of construction and with assumptions regarding the increase in $s_u$ based on the predicted degrees of consolidation. This procedure is described in detail in Paper III. Predictions of the magnitudes of settlements and horizontal deformations in the subsoil were also made. Both analytical and numerical analyses were made in order to establish a design under the most probable conditions according to Peck’s methodology.

4. Measurable quantities in order to control the behaviour of the construction were chosen, and predictions of their values during construction based on the most probable conditions were made. The quantities in this case were:

   a. Settlements of the original ground surface.
   b. Horizontal deformations in the ground.
   c. Pore pressure development in the clay layers.
   d. The increase in shear strength in the clay layers.
The quantities were predicted at various depths below the ground surface and at different locations under the embankment and berms, see Figs. 4.4-4.5 and Paper III.

5. The most unfavourable values of the aforementioned quantities based on the knowledge at that time were anticipated.

6. Action plans for potential deviations from the real behaviour of the embankment and subsoil were drawn up. In this case, the consolidation rates (rate of pore pressure dissipation), the magnitudes of the settlements and the increase in $s_u$ were the most important aspects. The consolidation rate controlled both the settlement rate and (perhaps most important) the rate of shear strength increase. If the consolidation rate were slower than anticipated, the time between two loading stages would have been prolonged in order to satisfy the stability demands. The same yielded if the measured
$s_u$ were lower than predicted. If the magnitudes of settlement had been larger than predicted, there would have been a need for additional filling, which in turn would have led to higher stress levels in the ground and required a larger surcharge. This would have meant a need for additional calculations of the stability and further measurements of $s_u$ during the construction time. The measurements of horizontal deformations in the ground were seen as an indicator of the stability conditions. If these measurements had exceeded predictions, more intense measurements would have been made.

7. During the building stage of the embankment, settlements, horizontal deformations, pore pressures and the increase in undrained shear strength in the clay $s_u$ were monitored and compared with design assumptions and predictions. A selection of results from measurements of settlements and excess pore pressures along with respective predictions of the most probable outcomes (according to step 4) are shown in Figs. 4.4-4.5. Predictions and measurements of horizontal deformations and $s_u$ profiles and how they were obtained and used in the design, is given in Paper III.

8. The only unfavourable deviation from the predictions which led to an action was that a lower consolidation rate was observed between stage 4 and 5. Instead of filling for stage 5 immediately after completion of stage 4, a “rest period” of approximately two months was introduced and the stability demands could be fulfilled.

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Fig. 4.4 Predicted and measured settlements; a) Centre of embankment; b) Berm
Fig. 4.5 Predicted and measured excess pore pressures in the middle of the sulphide clay layer; a) Centre of embankment; b) Berm
Chapter 5 – Summary of appended papers

5.1 Paper I - Veda trial embankment – comparison between measured and calculated deformations and pore pressure development

In this paper, measurements on the trial embankment built at the Veda site in advance of the “real” embankment are presented. The design of the trial embankment was made according to the suggestions in the Swedish design guide for vertical drains (Swedish Road Administration 1989) based on limited field and laboratory tests. During and after the building process, pore pressure development, settlements and horizontal deformations in the subsoil were observed. The main aim of the study was to prepare soil models for use with finite element software, providing a sufficient description of the behaviour of the different clay layers. The soil models were intended to be used in the design phase and the observational phase of the Veda embankment project. The focus was on finding material parameters for the different clay layers found in the subsoil at the site, for instance the moduli and hydraulic conductivity (permeability). As the finite element modelling was made with plane strain conditions, the hydraulic conductivity matching procedure proposed by Hird et al. (1992) was adopted (cf. Eq. 3.8a). In this procedure, the hydraulic conductivity in the horizontal direction $k_h$ and the hydraulic conductivity and extent of the smear zone $k_s$ and $d_s$, respectively, are needed. None of these three variables had been investigated before building the trial embankment, and therefore a parameter study to find a set of parameters that best fit the measurements of pore pressures in the ground was made. A group of soil models with various sets of parameters, each representing different parts of the clay deposit, were suggested for use in the design of the Veda embankment.

The main conclusion of the paper is the importance of carefully modelling $k_h$ and the characteristics of the smear zone in the design of PVD’s. The trial embankment was designed essentially based on the suggestions in the Swedish design guide aiming at an average 95% degree of consolidation occurring one year after completion. However, the measurements indicated a development of only a 60-70% degree of consolidation during that time. Hence, design based merely on empirical suggestions in e.g. design codes can be very misleading and result in serious difficulties during and after the building process. Furthermore, $k_h$ has a greater influence on the calculated equivalent hydraulic conductivity than the smear characteristics. It is suggested that $k_h$ should be assessed via laboratory or field tests and that the smear effects should be handled via sensitivity analyses.

5.2 Paper II – Hydraulic conductivity and the coefficient of consolidation of two sulphide clays in Sweden

The theories used for the design of vertical drains, described in Chapter 3, propose knowledge about the hydraulic conductivity and coefficient of consolidation in the horizontal direction.
Conventional laboratory testing of clay offers interpretations of these parameters in the vertical direction only. Hence, one is often left to make assumptions based on empirical knowledge of a certain type of clay or simply based on a more or less qualified guess.

This paper presents results from a series of CRS laboratory tests performed on samples of sulphide clay retrieved from two different sites where large infrastructure projects involving embankments on vertically drained subsoil are currently being built. A methodology to investigate the hydraulic conductivity in both the vertical $k_v$ and horizontal $k_h$ directions via conventional CRS tests with vertical drainage of excess pore pressures is proposed. In order to evaluate $k_h$, samples were trimmed from the sides of the sample cylinders into oedometer rings with a slightly smaller diameter (45 mm) than conventionally used (50 mm), see Fig. 5.1. On samples from the same testing tubes, both vertical tests and horizontal tests were conducted. The main purpose of the tests was to investigate the anisotropy of hydraulic conductivity and consequently the coefficient of consolidation via the ratios $k_h/k_v$ and $c_h/c_v$.

![Fig 5.1 Preparing the horizontal samples in the laboratory; a) the cutting device; b) trimming; c) the cutting device with the clay sample; d) mounting the clay sample in the oedometer ring](photos)

The laboratory-evaluated values of $c_h$ were also compared with evaluations based on in situ measurements of the consolidation rates. In addition, as a number of parallel tests were performed on samples of the “same” clay, it was possible to conduct studies of the variations...
in the evaluated properties. An investigation of the validity of previously presented empirical correlations linking the hydraulic conductivity with other properties of clay was also carried out.

The results from the laboratory tests show that the anisotropy in both hydraulic conductivity and the coefficient of consolidation was low. For the sulphide clays tested, an average value of $k_h/k_v \approx 1.3$ was found (Fig. 5.2).

The scatters in evaluated values of the initial hydraulic conductivities ($k_{v0}$ and $k_{h0}$) were significant. The quotients of maximum and minimum values from corresponding tests were between 1.7 and 3.3 with maximum deviations from the mean of 34% to 110% (Fig. 5.3). Regarding the coefficients of consolidation, the scatters were even larger. Quotients of maximum and minimum values from corresponding tests between 1.6 and 5.0 were measured. A conclusion from these findings underlines what an impossible task it is to determine these parameters for design purposes based on results from a single test or only a few tests. The paper concludes that several parallel tests should be made or that partial factors of safety should be introduced in the design.

![Fig. 5.2 Measured initial horizontal hydraulic conductivity vs. initial vertical hydraulic conductivity (from Paper II)](image-url)
5.3 Paper III – Stability for a high embankment founded on sulphide clay

Controlling the stability against shear failure is a very important issue to address in both the design phase and the building phase of an embankment construction. A case study describing how the stability was handled in the Veda embankment project is presented. The embankment was built in a staged construction mode on vertically drained sulphide clay, accounting for the increase in undrained shear strength $s_u$ during consolidation between the stages. As previous experience from building high embankments on sulphide clay and empirical knowledge of vertical drains in sulphide clay was limited, the aforementioned trial embankment (Paper I) was built and the observational method was employed in the project.

Building an embankment in stages allows for some consolidation during construction, hence increasing the effective stresses in the clay. If the effective stresses supersede the maximum past preconsolidation pressure in the clay, the shear strength in the soil increases. In the project, this was accounted for in the design phase via use of the undrained strength analysis (USA) methodology proposed by Ladd (1991). In order to adopt the USA methodology, reliable predictions of the increase in effective stresses during the building stages are required, i.e. the rate of the dissipation of excess pore pressures must be anticipated. Furthermore, reliable predictions of the increase in $s_u$ in relation to the present preconsolidation pressure $\sigma_p'$ must be made, i.e. the ratio $s_u/\sigma_p'$ must be established for the clay. Neither of these prerequisites was entirely fulfilled due to uncertainties in predicting the real behaviour of the sulphide clay. As discussed in Paper I, a soil model used for predicting the consolidation rate due to vertical draining of the sulphide clay was developed via measurements on the trial
embankment. The \( \frac{s_u}{\sigma'_p} \) ratio was investigated through field tests and laboratory tests, constituting a base for the predictions. The stability of the Veda embankment at the various construction stages was then predicted. In accordance with the observational method, the design predictions were controlled during the building phase of the embankment via continuous measurements of pore pressures and in situ tests of \( s_u \) at some stages.

The paper discusses the importance of assessing a certain quantity from several independent sources of measurements. In this case, the stability of the embankment was of concern, which is why \( s_u \) and the increase in \( s_u \) in the sulphide clay due to consolidation were of paramount importance. During construction, assessments of \( s_u \) at various stages were made via three different methods, directly via CPT tests and field vane tests and indirectly via pore pressure measurements. The measurements were also made at six different locations (Fig. 9 in Paper III). Comparison of the \( s_u \) profiles evaluated this way with the design predictions allowed evaluation of the stability of the construction during the building process. The large amount of measurements taken during construction showed that the ratio \( \frac{s_u}{\sigma'_p} = 0.25 \) is valid for a large stress interval (Fig 5.4). As shown, the scatter is considerable, which is why only one test or a few tests can be very misleading.

![Diagram](image-url)

**Fig. 5.4 Variation in \( s_u \) versus evaluated \( \sigma'_p \) (from Paper III)**
Chapter 6 – Discussion and proposed future research

6.1 Discussion

Designing and building an embankment or performing any other type of work involving placing a fill material on soft fine-grained soil involves controlling the construction in both the serviceability limit state (deformation) and the ultimate limit state (stability). In both cases, the most important aspect is the rate of consolidation, i.e. the rate at which the induced excess pore pressures dissipate. For serviceability limit state, the degree of consolidation directly determines the relative size of the remaining deformations due to primary consolidation at any point in time. If the occurrence of secondary consolidation (i.e. creep) is to be reduced by using a temporary surcharge, the rate of consolidation is decisive in determining when unloading can take place. For ultimate limit state, and if the stability depends on gaining strength due to the increase in effective stresses as a result of consolidation, correct predictions may be of even greater importance. Once the filling works have begun, there is not much one can do to speed up the consolidation process. Incorrect predictions of the time for reaching a certain degree of consolidation can lead to extended building times, possibly delaying a project, or lead to problems with post-construction deformations or insufficient safety margins against failure.

This thesis is concerned mainly with embankments on vertically drained sulphide clay. However, it highlights some of the aspects that should be taken into consideration when designing and building embankments on vertically drained soil in a more general manner as well. The art of geotechnical engineering is to a large extent a matter of handling various uncertainties, primarily arising from the fact that we are trying to master and understand a natural material that is far from steady and homogeneous, but instead likely to be rather disobedient.

The design of projects involving ground improvement of clay by means of vertical drains involves handling a vast number of more or less uncertain factors. First of all, the mathematical formulations and physical models at our disposal are (and always will be) limited due to simplifications and generalisations. Then there are uncertainties connected with quantification of the necessary parameters required as inputs in these mathematical expressions. For vertical drain design from the perspective of predicting the consolidation rate, these parameters are the hydraulic conductivity of the clay and its variation both spatially and during compression, its potential anisotropy and the detrimental effects due to disturbance effects during installation of the drain material. The difficulty of accurately determining the hydraulic conductivity of an “undisturbed” clay from tests on small samples of clay is considered in the appended Paper II. Lately, much of the research effort in the field of vertical draining has been devoted to trying to describe the extent and the characteristics of the smear zone and to finding mathematical formulations that account for various “special aspects” regarding the design of vertical drains (cf. Chapter 3). The aim should, of course, always be to try to describe every aspect in the best possible manner, but not without keeping focus on the fundamentals. If the natural horizontal hydraulic conductivity can not be described in an
acceptable way, the degree of perfection applied in modelling the smear zone or other aspects of the behaviour of vertical drains is wasteful. More effort should be directed towards refining and rationalizing the methods for measurement and evaluation of the horizontal hydraulic conductivity in clays. However, further study is needed primarily in how the uncertainties related to these evaluations and the other inevitable assumptions associated with vertical drains in clay should be handled. Perhaps the soil models and mathematical formulations currently available for describing soil behaviour are good enough in the context of our ability to quantify the necessary inputs and the connected uncertainties. It is believed that more research efforts need to be directed at identifying and quantifying the uncertainties involved in a geotechnical problem. The introduction of partial factors of safety in the design of vertical drains seems like a natural step in this process.

6.2 Future work

During the research work that forms the basis of this thesis, a number of questions and interesting topics for future research arose. As the design of vertical drains is inevitably connected with some degree of assumptions, empirical knowledge of the behaviour of the clay-drain system is vital. This is especially important for further studies on sulphide clay as the empirical knowledge and previous experience of vertical drains in this type of soil are very limited. Another potential topic for further study is how the various uncertainties are handled in the design and building phase of a structure founded on vertically drained soil. Below are some of the topics identified in the context of handling the uncertainties related to vertical drain design:

Primarily:
- How do we handle the uncertainties for practical engineering purposes? Should partial factors of safety be introduced? And in that case, how large should the factors be? Some means of statistical analyses, e.g. “Beta analyses”, could be used.
- The possibility of adopting a Bayesian updating procedure when judging the stability at the various loading stages during the building process. Bayes’ theorem is a way of rationally updating this uncertainty in light of new measurements (e.g. Ang and Tang 2008). This could be an interesting approach as knowledge about the behaviour of the construction is continuously increasing as results from measurements are gathered during the building process.
- A more in-depth study of the factors affecting vertical drain design for the proposed models on the market. Which parameters are most important and should therefore be investigated most closely in practice? This could be done via statistical sensitivity analyses of the parameters in the equations and quantification of the uncertainties in determining them, preferably applied to some relevant case examples.

Secondarily:
- How do we determine horizontal hydraulic conductivity and the coefficient of consolidation in the best possible manner? Evaluations from various types of tests, both in the field and in the laboratory, should be compared. A comparison with the proposed methodology presented in Paper II is also proposed.
- How should a construction be instrumented in order to evaluate the degree of consolidation both on average and in discrete layers of the soil? A comparison of different approaches for evaluation is proposed.
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