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VACUUM CONSOLIDATION FIELD TEST ON A PSEUDO-FIBROUS PEAT

Dissertation submitted to the University of Dublin, Trinity College,
for the degree of Doctor of Philosophy

by

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March 2012
Declaration

I declare that this thesis has not been submitted as an exercise for a degree at this or any other university and it is entirely my own work.

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Juan Pablo Osorio-Salas
Summary

Several different ground improvement methods have been used to improve soft highly compressible organic soils such as peat. Vacuum consolidation is a ground improvement technique developed in the 1950's, in which the atmospheric pressure is used as a surcharge to improve the geotechnical properties of the soil deposit. Since the 1980's, vacuum consolidation has become increasingly popular and has been successfully implemented in several countries, mainly for the improvement of soft highly compressible clays and hydraulic fills for land reclamation projects. Other soils such as peat and soda ash-tailings have also been improved using vacuum consolidation, although there are only a few publications on its use on these materials.

A vacuum consolidation field test was designed and successfully implemented in a 10 x 10 m area at Ballydermot bog, in order to investigate the field performance of vacuum consolidation in peat deposits, and to evaluate the viability of implementing this technique as a method for the construction and improvement of roads over peat in Ireland. The field test was instrumented and monitored for a period of 11 months. To evaluate the effect of the prefabricated vertical drains spacing, the test area was subdivided in two, one with a spacing of 0.85 m and another with a spacing of 1.20 m, both in a square grid.

Two vacuum generating systems were tested during the project. The first system used a jet pump to create the vacuum, reaching vacuum pressures up to 80 kPa, though in an intermittent manner. The second system used a liquid ring pump as the main vacuum generator, and a more stable pressure was maintained, with vacuum pressures up to 71 kPa.

In general, the settlement and pore water pressure behaviour in the two different PVD spacing areas was very similar through the entire project, indicating that the vacuum pressure was adequately transmitted to both sections. The settlement of both subareas followed each other closely during the entire testing period at all depths. A uniform pore water pressure reduction was observed at all depths, for both spacing subareas, even beneath the bottom of the PVDs.

The field test was back-analysed using the Soft Soil and the Soft Soil Creep models from PLAXIS. Acceptable results were obtained for the ground vertical displacements, although
the models seem to overestimate the heave. The Soft Soil Creep model tends to produce higher values of compression than the Soft Soil model, as it considers the effect of creep. The pore water pressures calculated using both models did not follow the behaviour shown by the measured values. The design parameters for the back-analysis were determined by means of conventional oedometer tests, showing that the geotechnical properties of the peat obtained in conventional tests can be used in the design of vacuum consolidation projects.

From the laboratory test results, a statistical study was conducted in the use of different temperatures (80°C and 105°C) to determine the water content in peat. It was shown that there is no significant difference on the water contents obtained under either temperature.

The results of the oedometer tests were analysed using four different models, namely: The End of Primary approach, Janbu's resistance concept, the Soft Soil model and the Soft Soil Creep model.

The End of Primary approach adequately modelled the stress–strain behaviour of Ballydermot peat. Janbu's resistance concept correctly described the stress–strain behaviour of all tests, and the strain–time behaviour of most load increments. However, the long term tests did not behave linearly as expected, but after certain time the linearity of the curve was distorted showing scatter behaviour. The Soft Soil and the Soft Soil Creep models correctly depicted the stress–strain behaviour observed on the oedometer tests. The strain–time behaviour was adequately described by the Soft Soil Creep model, but not by the Soft Soil model as this model does not consider creep. However the latter was able to determine the final vertical strain value.
Dedication

To my godson Tomás, and my cousins Silvia and José Andrés, who showed me that with courage and strength anything in life can be overcome.

A mi ahijado Tomás, a mis primos Silvia y José Andrés, quienes me enseñaron que, con fuerza y coraje, toda las dificultades se puede superar.
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I would like to thank Bord na Móna for allowing me to use the facilities at Ballydermot bog. Thank you to all the staff at the bog for their help and curiosity on why would you pump a bog down?
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1. INTRODUCTION

1.1 BACKGROUND AND MOTIVATION

Peat deposits cover great areas in Ireland and around the world. According to Hobbs (1986), about 17.2% of the overall land surface of Ireland is covered with peat (Figure 1.1); although for some counties the peat coverage is significantly greater, for example, 36.1% in County Leitrim (Hammond, 1981). Hence, a significant part of the road network of the country has been constructed across peat lands. Currently, there is an increasing demand for more and improved transport infrastructure to meet the modern traffic demands, safety requirements and to reduce maintenance cost.

Due to its high compressibility, creep behaviour and low shear strength, peat has generally been considered a problematic soil in geotechnical engineering. Several ground improvement methods have been used worldwide for treating problematic soft soils such as peat. These methods can be classified under six categories (Terashi and Juran, 2000): replacement, densification, consolidation, grouting, admixture stabilization, thermal stabilization, reinforcement and miscellaneous as the combination of two or more of the previous.

Peat ground improvement techniques such as surcharging and chemical stabilisation have been researched in Ireland. Hanrahan (1964; 1976; 1981) presented research studies in peat preconsolidation using a gravel embankment as a temporary surcharge and the installation of vertical drains to accelerate the consolidation process and produce a more uniform increase in the shear strength of the peat with depth. Hebib (2001) and Hebib and Farrell (2003) presented a study into methods of stabilising peats by the addition of binders, although this approach has not proven to be cost effective in practice.
Vacuum consolidation is a ground improvement technique in which the atmospheric pressure is used as a surcharge to improve the geotechnical properties of the soil deposit. The technique was originally developed in the early 1950’s by Kjellman (1952); however, due to technical and economical limitations the method was not widely used for several decades.

Since the 1980’s, vacuum consolidation has become increasingly popular and has been successfully implemented in several countries, particularly in Asia, mainly for the improvement of soft highly compressible clays and hydraulic fills for land reclamation projects. Other soils such as peat and soda ash-tailings have also been improved using vacuum consolidation, although there are only a few publications on its use on these materials.

1.2 OBJECTIVES

The aim of this thesis is to investigate the field performance of vacuum consolidation in peat deposits, and to evaluate the viability of implementing this technique as a method for the construction and improvement of roads over peat in Ireland.
The main objectives of this project are as follows:

(i) To implement a full scale vacuum consolidation test on a peat bog, in order to identify the setup requirements and procedure, and the technical issues that may be encountered in the field.

(ii) To examine the compressibility behaviour of peat subjected to vacuum loading in the field.

(iii) To evaluate the possibility of using traditional laboratory tests to determine the parameters for the design of vacuum consolidation projects in peat.

(iv) To back analyse the field test using a finite element model, and to assess the performance of the model to simulate the observed behaviour of the peat.

1.3 THEESIS OUTLINE

This thesis is composed of five chapter outlined as follows:

Chapter 2: describes the vacuum consolidation methodology and reviews the current knowledge obtained from field applications and laboratory studies on different materials. Also, a brief introduction vacuum consolidation modelling is presented. Finally, a discussion on the one-dimensional compressibility of peat is given.

Chapter 3: This chapter describes the test site, the findings of two cable percussion boreholes, two manual boreholes and three vane tests; and the results of the laboratory tests carried out on the samples recovered. Rainfall records and water table variations are also presented. The laboratory tests included classification tests and oedometer tests. Finally, the results of the oedometer tests are analysed using four different models.

Chapter 4: describes the design, implementation and monitoring of the field test at Ballydermot bog. Subsequently, the field results are discussed, and a finite element model is used to back-analyse the test, using the parameters determined in the laboratory. The
field and model results are compared, and the ability of the model to predict the field
behaviour is assessed. Finally, the technical difficulties encountered while implementing
the test are described and an analysis of its effect in the project is given.

Chapter 5: presents the main conclusions from the thesis, and makes recommendations for
future work.

Appendix A: presents the results of the field exploration, the laboratory tests and the
numerical models on the oedometer tests.

Appendix B: shows the differences between the stress paths under vacuum loading and
embankment loading.
2. LITERATURE REVIEW

2.1 INTRODUCTION

The construction and improvement of roads over peat has always been a challenging task in geotechnical engineering. Bog roads often undergo considerable distortion due to the low shear strength and high compressibility of the peat foundation, which may pose a significant safety hazard and requires high maintenance budgets to keep roads in service.

Preloading soft, highly compressible soils is probably one of the oldest and most widely used ground improvement techniques to increase the shear strength and reduce post-construction settlements of the ground (Mitchell, 1981), being earth fills the most commonly used type of preload. In the early 1950's Kjellman (1952) proposed using the atmospheric pressure as a temporary surcharge for improving clay soils. This technique of soil improvement has been successfully implemented since the 1980's in several countries in a diversity of projects and soil types.

This chapter initially describes the vacuum consolidation methodology and reviews the current knowledge obtained from field applications and laboratory studies on different materials. Also, a brief introduction to numerical modelling of vacuum consolidation is presented. Finally, a discussion on the one-dimensional compressibility of peat is given.

2.2 VACUUM CONSOLIDATION

Vacuum consolidation was originally proposed by Kjellman (1952) of the Royal Swedish Geotechnical Institute as a mean of improving the geotechnical properties of soft compressible clayey soils. The technique consists in applying a vacuum pressure to an isolated soil mass, thus reducing the pore pressure and increasing the effective stress, while the total stress remains constant, which means the overall stability is not affected.
2.2.1 Historical background

The traditional vacuum consolidation method (Figure 2.1) consists of a system of vertical drains driven down from the surface to a prescribed depth, a filter layer usually comprised by a sand or gravel bed, a collector pipe connected to a vacuum pump for transmitting the vacuum and collecting the water and air discharged from the soil mass. The treated area is isolated with an air-tight membrane placed on top of the filter layer and tightly connected to the ground around it, free from cracks and root channels. It is essential that the area is securely sealed to avoid leaks and loss of vacuum. Originally, sand or cardboard wick drains were used as vertical drains; and welded strips of sheet plastic, sheet rubber or thin layers of clay, bentonite or asphalt as a sealing membrane (Kjellman, 1952).

![Figure 2.1: Principle of Swedish vacuum method (Kjellman, 1952).](image)

However, apart from some isolated applications, the method was not widely used for several decades due to technical and economical limitations (Halton et al., 1965; Holtz and Wager, 1975; Johnson et al., 1977; Qian et al., 1992), namely the inability to maintain an effective vacuum pressure during treatment and its high cost compared with the alternative of surcharge fills. Vacuum consolidation was made more cost competitive by the advances in geosynthetics, prefabricated vertical drains (PVD) and by more effective vacuum pumps (Terashi and Juran, 2000; Dam et al., 2006).

Since the 1980s, vacuum consolidation has been successfully implemented in several countries in Asia, Europe and North America in projects such as ports, airport runways, roads, bridges and by-passes, warehouses, sewage treatment plants and oil storage stations.
Chapter 2

LITERATURE REVIEW

The technique has been mainly used to improve highly compressible clayey soils, but there are only a few publications on its use in peat soils (see Table 2.1, §2.2.4).

2.2.2 Theoretical background

The principles and mechanism of vacuum consolidation has been studied and explained in the literature by several authors (Kjellman, 1952; Cognon, 1991; Qian et al., 1992; Park et al., 1997; Leong et al., 2000; Masse et al., 2001; Chu and Yan, 2005a; Indraratna et al., 2005a; Rujikiatkamjorn and Indraratna, 2007b; Indraratna et al., 2010a).

2.2.2.1 Principle of vacuum consolidation

Significant differences exist between surcharge loading and vacuum loading. Normally, the atmospheric pressure, \( p_a \), is disregarded in geotechnical engineering as the calculations are usually based on effective stresses, and the atmospheric pressure variation is negligible. However, to understand the principles and mechanisms of vacuum consolidation the atmospheric pressure should be taken into account. Chu and Yang (2005a) described the mechanisms of both, surcharge and vacuum preloading, using the spring analogy (Figure 2.2). For convenience of explanation the pressures in Figure 2.2 are given in absolute values and \( p_a \) is the atmospheric pressure.

The process of consolidation under load is presented in Figure 2.2(a). As shown, in the instant the increment of load \( \Delta p \) is applied, the water takes the load, generating an increase in the pore water pressure (PWP), \( \Delta u \), equal to the surcharge \( \Delta p \). Gradually, the excess pore water pressure dissipates and the load is transferred from the water to the spring (representing the soil skeleton). The increase in the effective stress equals the amount of pore water dissipation, \( \Delta \sigma' = \Delta p - \Delta u \). At the end of the consolidation process, the excess PWP has dissipated, \( \Delta u = 0 \), and the increase in the effective stress is the same as the surcharge applied, \( \Delta \sigma' = \Delta p \). It should be noted that the above process is not affected by the atmospheric pressure, \( p_a \).
Figure 2.2: Spring analogy of consolidation process (a) under fill surcharge; (b) under vacuum load (Chu and Yan, 2005a).

The mechanism of vacuum consolidation can also be illustrated using the spring analogy (Figure 2.2(b)). When a vacuum load is applied to the system, the pore water pressure in the soil reduces. As the total stress applied does not change, the effective stress in the soil increases. In the instant when the vacuum load $-\Delta u$ is applied, the pore water pressure in the soil is still $p_a$. As the PWP reduces, the spring starts to compress, representing the gain in effective stress in the soil skeleton. The amount of the effective stress increment equals the amount of pore water pressure reduction, $\Delta \sigma' = \Delta u$. This pore water pressure reduction cannot exceed the value of the atmospheric pressure, $p_{at}$, and in practice is normally about 80 kPa, due to the efficiency of the pumping system.
If an idealised soil profile is considered, with the water table and a single drainage boundary at the ground surface, the PWP and effective stress distribution with depth at a given time during consolidation can be plotted in Figure 2.3(a) and (b) for surcharge and vacuum preloading, respectively.

For the surcharge preloading case (Figure 2.3(a)), the effective stress after loading is (Terzaghi et al., 1996):
where $\sigma_0'$ is the initial effective stress, $\Delta \sigma'$ the effective stress increment, $\Delta \sigma_v$ the surcharge, $\Delta u$ the excess pore water pressure, $u_0$ the initial hydrostatic PWP and $u_t(z)$ the new PWP distribution. As mentioned above, the excess pore water pressure is initially equal to the surcharge, but as it dissipates the load is transferred to the soil skeleton increasing the effective stress.

Replacing (2.2) and (2.3) into (2.1), the effective stress after loading can be expressed as:

$$\sigma_v(z)' = \sigma_0' + \Delta \sigma_v - u_t(z)$$  \hspace{1cm} (2.4)

Under vacuum load (Figure 2.3(b)),

$$\sigma_v(z)' = \sigma_0' + \Delta \sigma'$$  \hspace{1cm} (2.5)

$$\Delta \sigma' = \Delta u$$  \hspace{1cm} (2.6)

$$\Delta u = u_0 - u_t(z)$$  \hspace{1cm} (2.7)

hence, the effective stress after vacuum preloading is

$$\sigma_z(z)' = \sigma_0' + u_0 - u_t(z)$$  \hspace{1cm} (2.8)

It should be noted that in the case of vacuum consolidation, the increment in effective stress cannot exceed 98 kPa. However, if a higher load is required a combined vacuum-surcharge method can be adopted. Figure 2.4 represents the conventional consolidation process by surcharge preloading and the vacuum-surcharge preloading (Indraratna et al., 2005a).

Mohamedelhassan and Shang (2002) considered that the excess pore water pressure generated by a combined vacuum-surcharge preloading may be evaluated by the law of superposition as the sum of the excess pore water pressure generated by the surcharge and
the PWP reduction induced by the vacuum preloading. Accepting this hypothesis, the effective stress under the combined method can be written as:

\[
\sigma_v(z)' = \sigma_0' + \Delta \sigma_{sur} + \Delta \sigma_{vac}
\]  

(2.9)

Replacing (2.2) and (2.6) into (2.9), the effective stress after combined vacuum-surcharge preloading can be expressed as:

\[
\sigma_v(z)' = \sigma_0' + \Delta \sigma - \Delta u_{sur} + \Delta u_{vac}
\]  

(2.10)

If equation (2.10) is simplified for the special cases (i.e. vacuum only or surcharged only), equations (2.4) and (2.8) are obtained.
2.2.2.2 Lateral displacement

Embarkment loading will cause both settlement of the underlying soft soil and generally outward lateral displacement (Figure 2.5(a)). This lateral displacement is mainly caused by the shear stresses induced by the embarkment load, and if these shear stresses are big enough shear failure may be caused within the soil. By contrast, the vacuum pressure technique tends to apply an isotropic incremental consolidation pressure to the soft soil, which will induce settlement and inward lateral displacement (Figure 2.5(b)). Even with an isotropic pressure increment, the ground may not respond isotropically because deformation is also influenced by the initial stress state. The inward deformation may cause some surface cracks around the improvement area, but normally there is no possibility of general shear failure (Chai et al., 2006a). Several authors have reported the phenomenon of lateral inward movement on vacuum consolidation projects (Qian et al., 1992; Shang and Zhang, 1999; Chu et al., 2000; Hayashi et al., 2003; Song and Kim, 2004; Tran and Mitachi, 2007).

Mohamedelhassan and Shang (2002) designed and manufactured an apparatus for the parametric study of one-dimensional vacuum consolidation (see §2.2.5.2). From this study it was concluded that the nature of the consolidation pressure has no bearing on the compressibility, coefficient of consolidation or hydraulic conductivity of the soil. However, the authors cautioned about that the lateral displacement of soil is completely different between the surcharge and vacuum preloading techniques, and recommended further study to be carried.
Chai et al. (2005) conducted a series of laboratory oedometer tests, in a specially adapted apparatus, with one-way drainage conditions under either vacuum pressure or surcharge loading, on reconstituted samples prepared with different initial effective stresses (see §2.2.5.3). An analysis of the results showed that the settlement induced by the vacuum pressure is less than the settlement induced by the surcharge pressure, for the same magnitude load, in samples where the initial effective stress is less than the applied load during the test. However, for the case where the initial effective stress was the same as the applied load, the settlement induced by vacuum pressure and surcharge load were almost the same. Therefore, if the vacuum pressure is larger than the stress required to maintain a $k_0$ condition (no horizontal strain), there will be inward lateral displacement which will result in less settlement than that of an equivalent surcharge load. Otherwise, there will be no lateral deformation and the vacuum pressure will induce the same settlement as an equivalent surcharge load. This situation is illustrated in Figure 2.6 and the condition for inward lateral displacement to occur can be written as presented in equation (2.11).

\[ \Delta \sigma_{\text{vac}} > \frac{k_0 \cdot \sigma_{\text{vo}}}{1 - k_0} \]  

\( (2.11) \)

where $k_0$ is the at-rest horizontal earth pressure coefficient, $\sigma_{\text{vo}}$ is the in situ vertical effective stress and $\Delta \sigma_{\text{vac}}$ is incremental vacuum pressure.

The authors defined the stress ratio $k$ as follows:
for which, if \( k \leq k_0 \), there will be no lateral displacement and vice versa.

Qiu et al. (2007) makes a distinction between the vacuum pressure, which is the pressure below the atmospheric pressure, and the pore water pressure reduction. Figure 2.7 shows a schematic, denoted by the absolute pressure, of the idealized fluid pressure and the vacuum pressure profiles with depth of a prefabricated vertical drain (PVD). From this, it can be seen that the vacuum pressure is the highest at the ground level, but it reduces rapidly; and there is no vacuum pressure in the fluids below the depth \( z_0 \). The shaded portion in Figure 2.7 schematically illustrates the fluid pressure reduction in the PVD.

\[
k = \frac{\Delta \sigma_{\text{vac}}}{\Delta \sigma_{\text{vac}} + \sigma_{\text{vo}}}
\]

(2.12)

According to Qiu et al. (2007) it is important to differentiate between the characteristics of soil consolidation induced by the vacuum pressure and the pore pressure reduction. Since the vacuum pressure applies an isotropic incremental consolidation pressure the part of the soil that has a pressure lower than the atmospheric undergoes isotropic consolidation. For the pore pressure reduction, \( \Delta \mu \), that is greater than the atmospheric, there will be a corresponding vertical effective stress increase of \( \Delta \mu \). Owing to the lateral displacement...
restriction, the soil element can only generate vertical displacement. Therefore, the pore pressure reduction that is greater than the atmospheric pressure induces the soil to undergo one-dimensional compression.

2.2.2.3 Degree of consolidation (DOC)

The degree of consolidation (DOC) is an important parameter in evaluating the effectiveness of soil improvement, and is often used as a design specification in a soil improvement contract (Chu and Yan, 2005a). The DOC is normally calculated as the ratio of the current settlement over the ultimate settlement. However, for a soil improvement project, the ultimate settlement is unknown and has to be predicted. Asaoka’s method (1978) is commonly used to estimate the ultimate settlement in vacuum consolidation projects (Masse et al., 2001; Yan and Chu, 2005; Seah, 2006). As an alternative, pore water pressure data can be used to assess the DOC. The pore water pressure dissipation ratio can be calculated easily as the ratio between the amount of pore water pressure dissipation to the initial pore water pressure.

2.2.2.3.1 Degree of consolidation by the Asoka method

In the Asaoka analysis procedure, the time settlement curve of the settlement gauge is first plotted. A series of settlement values \( s_1, s_2, ..., s_i \) is selected, such that \( s_i \) is the settlement at time \( t_i \) and that the time interval, \( \Delta t = (t_i - t_{i-1}) \) is constant (Figure 2.8). The next step is to plot the points \((s_i, t_i)\) as shown in Figure 2.9. These points should lie on a straight line defined as (Arulrajah and Bo, 2008):

\[
s_i = s_0 + \beta s_{i-1}
\]  

(2.13)

Where \( s_0 \) and \( \beta \) are two constants which depend on the selected time interval, \( \Delta t \).

The ultimate settlement can then be predicted at the intercept of this line and a 45° line (Arulrajah and Bo, 2008). In the case of placement of additional fill, the straight line will be deviated after the point the additional fill is placed (Figure 2.9). When the settlement is
relatively small compared to the thickness of the soil deposit, the shifted line becomes almost parallel to the initial line (Asaoka, 1978).

Figure 2.8: Settlement and time interval selection on Asaoka’s method (Masse et al., 2001).

Once the ultimate settlement is determined, the DOC can be estimated by:

\[ U_{asa} = \frac{s_t}{s_{ult}} \]  

(2.14)

where \( s_t \) is the settlement at time \( t \) and \( s_{ult} \) is the ultimate primary settlement.
2.2.2.3.2 Degree of consolidation by the PWP monitoring method

In the pore water pressure procedure, once the pore water pressures at different depths are measured during preloading, the initial and final pore water pressure distributions with depth can be plotted. For generality, a combined fill surcharge and vacuum load case is considered. The typical pore water pressure distribution profiles for a combined loading case are shown schematically in Figure 2.10.

Using this profile, the average DOC \( U_{avg} \) can be calculated as (Chu and Yan, 2005b):

\[
U_{avg} = 1 - \frac{\int [u_t(z) - u_s(z)] dz}{\int [u_o(z) + \Delta \sigma - u_s(z)] dz} \tag{2.15}
\]

where:
- \( u_o(z) \): initial pore water pressure at depth \( z \).
- \( u_t(z) \): pore water pressure at depth \( z \) at time \( t \).
- \( u_s(z) \): suction line.
- \( \Delta \sigma \): the stress increment due to surcharge at depth \( z \).

The suction line is estimates as:

\[
u_s(z) = \gamma_w z - s \tag{2.16}\]
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where:
\( \gamma_w \): unit weight of water.
\( z \): depth.
\( s \): vacuum applied.

### 2.2.2.4 Horizontal coefficient of consolidation \( (c_h) \) and horizontal permeability \( (K_h) \)

The horizontal coefficient of consolidation, \( c_h \), and the horizontal permeability, \( K_h \), can be back-analysed from field pore water pressure measurements of vacuum consolidation projects (Bergado et al., 1998a) in the same manner as it is back-analysed for embankment loading (Nicholson and Jardine, 1981; Arulrajah and Bo, 2008).

Considering the case of radial drainage only, the average degree of consolidation \( (U_h) \) from Barron’s solution (1948) under ideal conditions (no smear and no well resistance) is as follows:

\[
U_h = 1 - \exp \left( \frac{-8T_h}{F(n)} \right) \tag{2.17}
\]

where

\[
c_h = \frac{T_h d_e^2}{t} \tag{2.18}
\]

\[
F(n) = \frac{n^2}{(n^2 - 1)} \ln(n) - \frac{3n^2 - 1}{4n^2} \tag{2.19}
\]

where \( n \) is the drain spacing ratio

\[
n = \frac{d_e}{d_w} \tag{2.20}
\]

\( d_e \) is the equivalent diameter of the unit cell determined as (Craig, 2004):
\[ d_e = 2 \cdot \sqrt{\frac{1}{\pi}} \cdot S \]  \hspace{1cm} (2.21)

And \( d_e \) is the equivalent drain diameter determined as (Chai and Carter, 2011):

\[ d_w = \frac{w + t_d}{2} \]  \hspace{1cm} (2.22)

Where \( w \) and \( t_d \) are the width and thickness of the PVD.

After determining the average degree of consolidation, \( U_{avg} \), from equation (2.15), it is now possible to obtain the horizontal coefficient of consolidation, \( c_h \), from equation (2.18). Subsequently, the horizontal permeability can be determined as (Nicholson and Jardine, 1981; Bergado et al., 1998a):

\[ K_h = m_v c_h \gamma_w \]  \hspace{1cm} (2.23)

where \( m_v \) is the coefficient of volume compressibility and \( \gamma_w \) is the unit weight of water.

### 2.2.2.5 Comparison vacuum and surcharge preloading

The general characteristics of vacuum preloading in comparison with conventional embankment preloading are as follows (Qian et al., 1992; Indraratna, 2009):

- The effective stress increase is isotropical and, therefore, the lateral deformation is compressive. Consequently, the risk of shear failure can be minimised even at a higher rate of embankment construction. However, any ‘inward’ movement towards the embankment toe should be carefully monitored to avoid excessively high tensile stresses.
- The vacuum head can propagate to a greater depth of subsoil via the PVD system and the suction can propagate beyond the tips of the drain and the boundary of the PVD.
- The volume of the surcharge fill can be reduce to achieve the same degree of consolidation, depending on the efficiency of the vacuum system used in the field (i.e. air leaks).
• Since the height of the surcharge can be reduced, the maximum excess pore pressure generated by vacuum preloading is less than the conventional surcharge method (Figure 2.4).

Additional advantages of the vacuum consolidation method, compared with embankment loading, were mentioned by (Chai et al., 2005; Soares Marques and Leroueil, 2005):

• For projects in which stability is a major concern, the use of vacuum preloading may reduce the number of stages of loading and consequently construction duration.
• Since fill material may not be required, or may be required in a reduced volume, the fill disposal is also reduced or non-existent, which is environmentally preferable.
• The vacuum consolidation method is environmentally friendly since it does not put any chemical admixtures into the ground.

Furthermore, Indraratna et al. (2010b) presented the carbon dioxide (CO₂) emissions of three vacuum consolidation projects and concluded that the vacuum consolidation emissions are lower than for a bored pile solution, making it an environmentally friendly method for future infrastructure development.

Yan and Chu (2003) stated that the cost of soil improvement using vacuum preloading is only two thirds of that by fill surcharge, based on local prices of electricity and materials. This may be considered as an initial approximation for comparison, but a strict analysis of costs must be performed in every case.

2.2.3 Techniques of vacuum consolidation

Since its inception great contributions have been made in the development and advancement of the vacuum consolidation technique, in both, practical applications and theoretical explanations of its principles. Presently, most vacuum consolidation systems employed are basically similar to the one developed by Kjellman (1952), though every system may have some specific features. According to Indraratna et al. (2007), the vacuum consolidation systems can be categorised in two major groups, a membrane system and a membraneless system. Dam et al. (2006) classified the vacuum consolidation systems according to the
condition where the technique was applied, into on-land and under-water vacuum consolidation systems. For the purpose of this thesis, the former classification will be used.

2.2.3.1 Membrane system

The membrane vacuum consolidation system is basically the original vacuum consolidation method, as proposed by Kjellman (1952) and described in §2.2.1, improved with the use of new materials and construction techniques.

Figure 2.11 shows a typical cross-section of a membrane vacuum consolidation project. The usual construction process is as follows:

- Placing geotextile membrane and/or a granular bed above the ground surface to provide a working platform, if necessary.

- Installation of the prefabricated vertical drains and the instrumentation to a prescribed depth. In case the soft deposit to be improved is underlain by a more permeable layer, i.e. a sand or gravel layer, the PVDs are installed so that they only partially penetrate the...
soft deposit, normally leaving a 1.0 m cushion between the tip of the PVD and the permeable layer, to avoid any vacuum loss.

- Embedding horizontal flexible-perforated pipes, linked between them, in the granular bed to transmit the vacuum and collect the air-water mixture from the soil mass (Figure 2.12).

- Excavation of peripheral trenches, around the treated area, to a depth of about 0.50 m below the water table.

- The granular bed is then covered with an air-tight membrane. The membrane is keyed to the bottom of the trenches, and the trenches are filled with clay or any impervious material to isolate the treated area from the atmosphere. Outside the treated area, the water table works as a seal from the atmospheric pressure. It is essential that the area is securely sealed to avoid leaks and loss of vacuum.

- The drainage system is then connected to the collector pipe, which in turn is connected to the vacuum pump.

- If the vacuum consolidation is to be combined with surcharge fill to induce a higher consolidation of the soil mass, the fill is then placed on top of the membrane. If vacuum consolidation is to act alone, then the membrane is protected by covering with clay material or ponded with water to reduce environmental effects such as UV-rays or animal attacks.

In vacuum consolidation projects, generally, a high efficiency vacuum pump system is used to provide suction to the soil and to discharge the air-water out through the system of pipes and drains. According to Qiu et al. (2007), in early vacuum preloading applications, vacuum pumps connected to the collection system were used for providing negative pressure and separating the air-water mixture drawn out from the treatment area. However, owing to the complication of air-water separation and the badly sealed in situ boundary conditions, the vacuum preloading was not widely applied to practice. In China, it was not until vacuum pumps were replaced by jet pumps that the vacuum preloading method was applied successfully in situ. According to Dam et al. (2006) the vacuum pumps has been replaced by a 48 mm diameter jet pump (7.5 kW) propelled by a centrifugal pump, a system able to generate a vacuum pressures greater than 90 kPa.
For the Menard Vacuum System (Masse et al., 2001), a vacuum station consists of a specifically designed high-efficiency vacuum pump acting solely on the gas phase in conjunction with a conventional vacuum pump allowing liquid and gas suction, and generating up to 80 kPa of vacuum pressure. In Japan, Maruyama Industry Ltd. and Hazama Co. group (N&H group) successfully employed a specially designed vacuum pump system, which can separate water and air collected in a series of tanks by means of build-in discharge water pumps, thus sustaining high under-sheet vacuum pressure (up to 90 kPa) during treatment (Dam et al., 2006).

Shang et al. (1998) used several vacuum pumps with a capacity of 7.5 kW, each capable of generating up to 80 kPa vacuum pressure over an area of 1000 – 1500 m². In contrast, Tang and Shang (2000) were able to maintain the same vacuum pressure over an area of 600 m², due to the presence of a 2-3 m thick silty sand layer on the surface (Figure 2.13). Shang and Zhang (1999) treated a 2025 m² area of soda-ash tailing using four vacuum pumps with a capacity of 30 kW. According to the authors, more vacuum pumps per unit area are needed for soda-ash tailings than applications in soft clayey soils to accommodate the higher hydraulic conductivity of the tailings.
According to Dam et al. (2006), large treatment areas can be subdivided into individually seal blocks. In Chinese practice, the block size normally varies at 6000 – 10000 m$^2$, although it is feasible up to a maximum of 30000 m$^2$. For each block, one or more vacuum pumps are equipped providing that an area for individual vacuum pump is standardized at 1000 – 1500 m$^2$. However, in case of presence of thick pervious layer near the surface causing lateral leakage of vacuum, the individual treatment area should be reduced.

On the other hand, Zhu and Miao (2002) reported that in order to maximise the vacuum beneath the sealing membrane, the jet pumps in the vacuum-generating equipment are placed on the same level or slightly lower the sealing membrane, thus reducing the energy loss in the discharge piping system and increasing the vacuum beneath the sealing membrane. This has allowed to increase the area which can be improved by a single set of vacuum-generating equipment to 2500 – 3600 m$^2$ from the original 1000 – 1500 m$^2$, and the vacuum pressure under the membrane has been increased to 90 – 95 kPa from the original 80 – 85 kPa.

For the Menard Vacuum Consolidation system design, the standard area for a vacuum pump station is 5000 – 7000 m$^2$. For the N&H group, the individual treatment area is specified at 2000 – 2500 m$^2$, which will be supported by a single vacuum pump (Dam et al., 2006).
In membrane vacuum consolidation projects, there are some technical and operational factors affecting the effectiveness and economy of the technique (Cognon et al., 1994): (i) integrity (airtight) of the membrane, (ii) seal between the membrane edges and the ground, (iii) soil stratification including permeable layer within the soil deposit, and (iv) depth of ground water level.

Chai et al. (2008) presented two typical situations under which it is difficult to maintain the air-tightness using this technique:

- A high air-water permeability layer at ground surface. To avoid air leakage, the air-tightening sheet must be embedded below this layer, or an air-water cut-off wall penetrating through this layer and into the underlying lower permeability layer must be built around the perimeter of a preloading area. In practice, these options are costly and sometimes their effect is not guaranteed.
- If vacuum consolidation is combined with surcharge preloading, after placing the embankment fill, any damage to the air-tightening sheet cannot be identified and repaired, and consequently the effect of vacuum pressure will be reduced.

Tang and Shang (2000) encountered the first difficulty while working in the Yaoqiang Airport runway. A silty sand layer, 2-3 m thick, imposed difficulties in sealing the area and reaching the design vacuum pressure. The installation of in-situ deep mixing slurry cut-off walls along with reduced coverage area of vacuum pumps enabled the soil improvement objectives to be achieved by vacuum preloading treatment. Figure 2.13 shows a schematic of the construction of the cut-off wall along with a cross section of the treated area.

The second difficulty was discussed by Bergado et al. (1998b), while examining the results of a vacuum-surcharge test for the Second Bangkok International Airport. During this test, some leakages occurred, that could not be repaired, which reduced the final settlement to 0.96 m from the 1.60 m expected.

In recent years, corrugated-flexible-perforated pipes (Figure 2.14(a)) have been replaced with drain panels (Figure 2.14(b)). This is to ensure the drainage channels will still function well under a high surcharge pressure, as in the case of combined fill and vacuum
preloading. The drainage panels also provide better channels for distributing vacuum pressure and water discharging. Some drainage panels also have slots for direction connection with PVDs and thus improve the efficiency of the system (Chu et al., 2008).

![Figure 2.14: Horizontal pipes used for vacuum preloading (Chu et al., 2008)](image)

**2.2.3.2 Membraneless system**

As mentioned in the previous section, when the total area to be treated is too large, it needs to be divided into blocks for individual vacuum treatment. This procedure reduces the efficiency of the method as one section is carried after another (Indraratna et al., 2007). Recently, to overcome this difficulty, a membraneless vacuum system (also known as vacuum-drain consolidation) was developed where each individual drain is connected directly to the vacuum line using a tubing system (Figure 2.15). Using this technique, vacuum consolidation can be conducted for a soil deposit under water, a situation that the conventional vacuum consolidation method finds difficult to apply (Chai et al., 2008).

In vacuum-drain consolidation a special PVD is used, consisting of a PVD, a drainage hose and a cap connecting the PVD and the hose, and it is named cap-drain (CPVD). A CPVD unit is shown in Figure 2.16. The method for installing CPVD is the same as that for installing PVD.

This technique has been applied recently in projects such as two test sections in reclaimed land at Tokio Bay in Japan (Chai et al., 2010), the ground improvement works at Suvarnabhumi Airport (Seah, 2006), and a land reclamation project at Yamaguchi Prefecture, Japan (Chai et al., 2008).
Figure 2.15: Membraneless vacuum consolidation typical cross-section (Saowapakpiboon et al., 2010)

Figure 2.16: Structure of CPVD (a) illustration (b) an actual CPVD (Seah, 2006; Tanchaisawat et al., 2007).
The vacuum-drain consolidation method uses a surface or subsurface soil layer as a sealing layer and there is no need to place an air-tightening sheet on the ground surface, and therefore, no worry about air leakage caused by damage to the sheet (Figure 2.17).

\[ H_s = \frac{(P_{vac})_{cap}}{\gamma_w Q_a} k_{air} \cdot A \]  

where \( \gamma_w \) is the unit weight of water, \( k_{air} \) is the permeability to air flow of the sealing layer, \( A \) is the area of treatment, and \( Q_a \) is the capacity of a vacuum pump.

Figure 2.17 illustrate two different situations under vacuum-drain consolidation. Figure 2.17(a) presents a homogenous soft soil deposit, with a sealing layer placed on top and the...
cap of the PVD embedded inside. Figure 2.17(b) shows the situation where there is a sand layer in the middle of a clayey deposit. To avoid vacuum pressure losses through this sand layer, a sealing sheet is pasted on the filter of the drain passing through the layer. To effectively use this technique, an intelligent construction system is needed. One of the methods is to install a piezocone on the tip of the mandrel for installing CPVD. The piezocone readings during current CPVD installation can be used to determine the location of the sealing sheet on the adjacent CPVD (Chai et al., 2008).

As the vacuum pressure is applied to the drain through the hose, the vacuum pressure will have the maximum value just below the cap, at the bottom of the sealing layer (Figure 2.17). In this layer, the vacuum pressure will vary from \( (p_{vac})_{cap} \) at the bottom to zero at the ground surface.

According to Saowapakpiboon et al. (2008), the advantages of vacuum-drain consolidation over the traditional membrane technique are:

- No liner needed that stay behind in the soil or has to be taken away.
- Direct connection of every drain to vacuum pump without flow resistance.
- No border trench needed.
- No damage possible due to settlements.
- Standard drain machines can be used.
- No skilled labour needed.
- Better control on functioning due to separate testing of the drain sections.
- No drainage layer needed.

Unlike in the membrane system where any leak can affect all the drains, in this system each drain acts independently. However, the requirement of extensive tubing for hundreds of drains can affect the installation time and costs (Chu et al., 2008). Furthermore, since this system does not provide an airtight condition for the entire area, the efficiency of the system can be low. The vacuum pressure applied may only be 60 kPa or lower (Seah, 2006; Chai et al., 2008; Saowapakpiboon et al., 2010); however, Chai et al. (2010) reported vacuum pressure of 80 – 90 kPa for two test section at Tokyo Bay. Also, this method only
works when the soil layer to be improved in predominantly clay with low permeability (Chu et al., 2008).

2.2.4 Field applications

The vacuum consolidation method has been successfully implemented in many countries worldwide. According to Yee and Ooi (2010), by 2010 an estimated 40 vacuum consolidation projects with an area of more than 6 million m² had been successfully treated. In China alone, the total treated area was more than 2 million m² by 1996 (Dam et al., 2006). Masse et al. (2001) reported applications in Germany, Thailand, South Korea, Canada, French Indies, Malaysia and France; over a period of 12 years. Table 2.1 presents a compilation of several vacuum consolidation projects found in the literature from 1958 to 2010.

From Table 2.1 it can be seen that the majority of vacuum consolidation projects have been performed in soft clayey soils and hydraulic fills. However, other materials such as peat and soda-ash tailings have also been improved, though to a lesser extent. A description of selected vacuum consolidation projects including peat in the soil profile is given here.

2.2.4.1 Pilot test for highway embankment, France (Cognon et al., 1994)

A vacuum consolidation pilot test was conducted in Ambes, France, for the construction of a highway embankment in very compressible saturated peat and clayey soils. The soil profile indicated the presence of 1.7 m of peat with a water content ranging from 400% to 900%, underlain by 2.0 m of highly organic, compressible clay with a water content ranging from 140% to 210% (Figure 2.18(a)).

The project called for the construction of a 2.15 m high embankment, to protect the highway from floods. Different solutions, including soil replacement, pile foundation and surcharge preloading were considered and discarded for economic reasons. The conventional preloading alternative required a gently sloping embankment with a 65 m base width, making it extremely expensive.
### Table 2.1: Vacuum consolidation projects in the literature

<table>
<thead>
<tr>
<th>PROJECT</th>
<th>SOIL TYPE</th>
<th>SYSTEM / PUMPS / VACUUM</th>
<th>AREA (m²)</th>
<th>COUNTRY. YEAR</th>
<th>REFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Runway extension, Philadelphia International Airport</td>
<td>Hydraulic clay and silt fill over soft clayey silt</td>
<td>Membraneless, deep well pumps, sand drains / 15 vertical turbine pump / 50 kPa</td>
<td>139445</td>
<td>USA, 1958</td>
<td>(Halton et al., 1965)</td>
</tr>
<tr>
<td>Field test section, Second Bangkok International Airport</td>
<td>Soft to very soft clay</td>
<td>Membrane, sand drains / deep well vacuum pump / 60 - 70 kPa</td>
<td>1600</td>
<td>Thailand, 1983</td>
<td>(Woo et al., 1989)</td>
</tr>
<tr>
<td>Field test, Reclamation land</td>
<td>Clayey soil dredge from seabed</td>
<td>Membraneless, horizontal drains / Vacuum pump and negative pressure tanks system / 80 - 90 kPa</td>
<td>350</td>
<td>Japan, undated</td>
<td>(Shinsha et al., 1991)</td>
</tr>
<tr>
<td>soda-ash tailings from chemical plant</td>
<td>Soda-ash tailing</td>
<td>Membrane, sand drains / Unspecified vacuum pump / 80 - 86 respectively</td>
<td>40000</td>
<td>Japan, undated</td>
<td></td>
</tr>
<tr>
<td>Tianjin New Harbour</td>
<td>Silty clay</td>
<td>Membrane, sand drains/ Unspecified vacuum pump</td>
<td>1250</td>
<td>China, undated</td>
<td>(Qian et al., 1992)</td>
</tr>
<tr>
<td>Northeast New Railway</td>
<td>Peat and silt</td>
<td>Membrane, surcharge, PVD / Unspecified / Unspecified</td>
<td>1950</td>
<td>Japan, undated</td>
<td></td>
</tr>
<tr>
<td>Factory in Lianyungang City</td>
<td>Marine clay</td>
<td>Membrane, surcharge, PVD / Unspecified / Unspecified</td>
<td>4000</td>
<td>China, undated</td>
<td></td>
</tr>
<tr>
<td>Pilot test Highway embankment, Ambes</td>
<td>Peat and highly organic clay</td>
<td>Membrane, surcharge, PVD / 25kW water-air pump backed by Venturi pumps / combined 150 kPa</td>
<td>300</td>
<td>France, 1990</td>
<td>(Cognon et al., 1994)</td>
</tr>
<tr>
<td>Oil tank farm, Ambes</td>
<td>Saturated alluvium (silt, peat and organic clay)</td>
<td>Membrane, surcharge, PVD / Unspecified / Unspecified</td>
<td>Unspecified</td>
<td>France, undated</td>
<td></td>
</tr>
<tr>
<td>Lemantin Airport</td>
<td>Compressible alluvium</td>
<td>Membrane, surcharge, PVD / Unspecified / Unspecified</td>
<td>Unspecified</td>
<td>France, undated</td>
<td></td>
</tr>
<tr>
<td>Field test Pier 300, Los Angeles</td>
<td>Hydraulic landfill (clayey and sandy soils)</td>
<td>Membrane,wick drains / Vacuum pump and negative pressure tanks system / 65 kPa</td>
<td>900</td>
<td>USA, undated</td>
<td>(Jacob et al., 1994)</td>
</tr>
<tr>
<td>Waste water treatment plant, Kimhae</td>
<td>Silty clay and clayey loam</td>
<td>Membrane, surcharge, PVD / Unspecified vacuum pump / 70 kPa</td>
<td>Unspecified</td>
<td>Korea, undated</td>
<td>(Park et al., 1997)</td>
</tr>
<tr>
<td>Reclaimed land at Xingang Port, Tiangang</td>
<td>Soft hydraulic fill, peat and soft organic clay</td>
<td>Membrane, surcharge, PVD / Several 7.5 kW vacuum pumps per subdivision (1000 - 1500m² per pump) / 80 kPa</td>
<td>Total = 480000; Subdivisions = 5000 to 30000</td>
<td>China, 1988</td>
<td>(Shang et al., 1998)</td>
</tr>
<tr>
<td>Field tests Second Bangkok International Airport (Suvarnabhumi Airport)</td>
<td>Soft Bangkok clay</td>
<td>Membrane, surcharge, PVD / Vacuum pump and water collection system / 60 kPa</td>
<td>1600 each (two tests)</td>
<td>Thailand, undated</td>
<td>(Bergado et al., 1998b)</td>
</tr>
<tr>
<td>Field test in a soda-ash tailing discharge dock, Dalian Harbour</td>
<td>Soda-ash tailing</td>
<td>Membrane, PVD / Four 30 kW vacuum pumps / 80 kPa</td>
<td>2025</td>
<td>China, 1993</td>
<td>(Shang and Zhang, 1999)</td>
</tr>
<tr>
<td>Pilot test at Port of Antwerp</td>
<td>Dredged silt</td>
<td>Membraneless, horizontal drains / Vacuum pump and submerged discharge pump / 80 kPa</td>
<td>Unspecified</td>
<td>Belgium, 1991</td>
<td>(Van Mieghem et al., 1999)</td>
</tr>
<tr>
<td>Pilot test and full treatment at Yaoqiang Airport runway</td>
<td>Alternate layer of silty sand, silty clay, silt and clay</td>
<td>Membrane, slurry cut-off wall, PVD / One 7.5 kW vacuum pump per area of 600m² / 70 - 80 kPa</td>
<td>1600</td>
<td>China, 1990</td>
<td>(Tang and Shang, 2000)</td>
</tr>
<tr>
<td>Oil storage station, Tianjin</td>
<td>Soft clay</td>
<td>Membrane, surcharge, PVD / Jet pump with centrifugal pump / 80 kPa</td>
<td>Total = 50000; Two sections of 30000 and 20000</td>
<td>China, undated</td>
<td>(Chu et al., 2000)</td>
</tr>
<tr>
<td>Review of 20 different project</td>
<td>Different materials, but mostly clays</td>
<td>Membrane - Surchage</td>
<td>Different</td>
<td>Several, 12 years</td>
<td>(Masse et al., 2001)</td>
</tr>
<tr>
<td>Embankment stabilization, Ishikari</td>
<td>Peat, peaty clay, silty sand, silty clay and sand</td>
<td>Membrane, surcharge, PVD / Unspecified pump / 60 - 70 kPa</td>
<td>Total = 20870; divided in 13 block</td>
<td>Japan, undated</td>
<td>(Shiono et al., 2001)</td>
</tr>
</tbody>
</table>
### Table 2.1 (cont.): Vacuum consolidation projects in the literature.

<table>
<thead>
<tr>
<th>PROJECT</th>
<th>SOIL TYPE</th>
<th>SYSTEM / PUMPS / VACUUM</th>
<th>AREA (m²)</th>
<th>COUNTRY, YEAR</th>
<th>REFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Review of different projects in China</td>
<td>Different materials</td>
<td>Membrane - Surcharge</td>
<td>Different</td>
<td>China, several</td>
<td>(Zhu and Miao, 2002)</td>
</tr>
<tr>
<td>Embankment construction at route 337, Sapporo</td>
<td>Peat, clayey peat, clay, fine sand, clay, sand</td>
<td>Membrane, surcharge, PVD / Two vacuum pumps / 60 kPa</td>
<td>3200</td>
<td>Japan, undated</td>
<td>(Hayashi et al., 2002)</td>
</tr>
<tr>
<td>Road at Tianjin Port</td>
<td>Silty clay slurry, silt, silty clay</td>
<td>Membrane, PVD / Jet pump with centrifugal pump / 80 kPa</td>
<td>Total = 18590; two halves sections</td>
<td>China, undated</td>
<td>(Yan and Chu, 2003)</td>
</tr>
<tr>
<td>Trial embankments with vacuum and heating</td>
<td>Clay, silty clay, glacial till.</td>
<td>Membrane, PVD and ground heating in one test / Unspecified pump / 80 kPa</td>
<td>169 each (two sections)</td>
<td>Canada, 1998</td>
<td>(Soares Marques et al., 2003)</td>
</tr>
<tr>
<td>Pilot test and full treatment at the Port of Huanghua</td>
<td>Hydraulic fill, alternate clayey silt and silty clay</td>
<td>Membrane, PVD / Unspecified vacuum pump / 80 kPa</td>
<td>Total = 800000; sections of 600 – 900 per pump</td>
<td>China, 1998</td>
<td>(Gao, 2004)</td>
</tr>
<tr>
<td>Sewage disposal plant in Jang-U</td>
<td>Silt, estuarine marine clay, sand-gravel mixture, weather rock</td>
<td>Membrane, surcharge, PVD / Unspecified vacuum pump / 80 kPa</td>
<td>Unspecified</td>
<td>Korea, undated</td>
<td>(Song and Kim, 2004)</td>
</tr>
<tr>
<td>Storage yard at Tianjin Port</td>
<td>Silty clay, muddy clay, soft to stiff silty clay</td>
<td>Membrane, PVD / Jet pump with centrifugal pump / 80 kPa</td>
<td>Total = 7433; Three sections of 2400, 3570 and 1394</td>
<td>China, undated</td>
<td>(Yan and Chu, 2005)</td>
</tr>
<tr>
<td>Road construction in Saga</td>
<td>Fill, clay, silty clay, sandy clay, gravelly sand</td>
<td>Membrane – Surcharge / unspecified pump / 60 – 70 kPa</td>
<td>Total = 16796; seven sections of 2040 to 3179</td>
<td>Japan, 2003</td>
<td>(Chai et al., 2006)</td>
</tr>
<tr>
<td>Review of different projects worldwide</td>
<td>Different materials</td>
<td>Membrane – Surcharge and Membraneless – Surcharge</td>
<td>Different</td>
<td>Worldwide, several</td>
<td>(Dam et al., 2006)</td>
</tr>
<tr>
<td>Pilot test in reclaimed landfill</td>
<td>Hydraulic landfill</td>
<td>Membraneless, horizontal drains / Unspecified vacuum pump / 60 kPa</td>
<td>15000</td>
<td>Korea, undated</td>
<td>(Lee et al., 2006)</td>
</tr>
<tr>
<td>Ground improvement works at Suvarnabhumi Airport</td>
<td>Fill, very soft to soft clay, medium stiff clay, stiff to very stiff clay and medium to dense sand</td>
<td>Membrane, surcharge, CPVD / Unspecified vacuum pump / 80 kPa</td>
<td>Over 400000 using both methods</td>
<td>Thailand, From mid ‘90s to 2006</td>
<td>(Seah, 2006; Sawapatkphiiboon et al., 2010)</td>
</tr>
<tr>
<td>Ground improvement works at Nansha Terminal of Guangzhou Port</td>
<td>Hydraulic fill, soft clay, sandy silt, silty clay and sandy clay</td>
<td>Membrane, PVD / Jet pump / 70 kPa</td>
<td>28200</td>
<td>China, 2003</td>
<td>(Qiu et al., 2007)</td>
</tr>
<tr>
<td>Test embankment, Kushiro City</td>
<td>Peat, clay, peat, sand, clay</td>
<td>Membrane, surcharge, PVD / Unspecified pump / 60 kPa</td>
<td>1350</td>
<td>Japan, undated</td>
<td>(Tran and Mitachi, 2007)</td>
</tr>
<tr>
<td>Reclaimed land project at Yamaguchi Prefecture</td>
<td>Reclaimed clayey layer, original clayey layer, sand</td>
<td>Membrane less, CPVD / Unspecified pump / 65 kPa</td>
<td>11100</td>
<td>Japan, 2004</td>
<td>(Chai et al., 2008)</td>
</tr>
<tr>
<td>Two test section at reclaimed landfill in Tokyo Bay</td>
<td>Reclaimed layer, clayey soil</td>
<td>Membrane less, CPVD / Unspecified pump / 80 – 90 kPa</td>
<td>3600 and 3782</td>
<td>Japan, 2006</td>
<td>(Chai et al., 2010)</td>
</tr>
</tbody>
</table>
The vacuum consolidation pilot test was undertaken along with a reference embankment test, in order to assess the efficiency of the technique. The test was conducted as follows:

- Placing a 0.8 m of sand blanket as draining layer at the surface.
- Installation of vertical drains in a square grid of 1.4 m x 1.4 m.
- Installation of horizontal drains and connections to vertical drains.
- Placing additional 0.5 m granular bed.
- Installation of air-tight PVC membrane on the surface, in a 390 m² area, and sealed along a peripheral trench with bentonite slurry.
- Vacuum application for 3 months with a 25 kW water-air pump, backed by Venturi pumps.
- The instrumentation and monitoring of settlements and PWP.
During the first day of operations, a settlement of 0.06 m was recorded, corresponding to a water flow of 600 m$^3$ per day per hectare. Figure 2.18(b) presents the monitored settlement during the test. According to the authors, it indicates that the settlement achieved is approximately equivalent to that induced by a 4.5 m high embankment; reaching about 80% degree of consolidation in the peat layer and 50% in the organic clay. At the end of the vacuum application, the ground surface rebounded about 0.03 m in the first 48 hours and then stabilized.

Figure 2.19 shows the PWP, initial effective stress, maximum effective stress attained after 40 days, anticipated effective stress due to the designed 2.15 m embankment, and due to reference 4.5 m embankment. These results appear to be consistent with the percentage of consolidation established for each layer. In particular the effective stress after 40 days of vacuum application correspond to the final effective stress expected under the highway embankment.

After the pilot test, vacuum consolidation was selected as the best available solution for the preloading of the 17500 m$^2$ highway.

Figure 2.19: Measured pore pressures and effective stresses at Ambes site, France (Cognon et al., 1994).
2.2.4.2 Soil improvement for 480000 m$^2$ of reclaimed land at Xingang Port, China (Shang et al., 1998)

Vacuum consolidation was used at Xingang Port, Tianjing, China; for the improvement of 480000 m$^2$ of reclaimed land at The East Pier, located on the eastern side of the port (Figure 2.20). The project was undertaken in 29 months, from 15 June 1987 to 7 November 1989, 68 days ahead of the scheduled 31 months.

The general soil profile is shown in Figure 2.21 and consists of six layers: (i) extremely compressible 4.0 m clayey hydraulic fill, still in the process of self-weight consolidation at
the time of the project; (ii) 3.5 m layer of original peat deposit with a local silty clay and clay layer of 1.0 – 1.5 m in thickness; (iii) soft organic clay intercalated with thin lenses of silty sands, 4.0 m thick; (iv) 3.0 m of homogeneous organic clay – peat with a water content up to 60%. The hydraulic permeability was nearly isotropic in both the horizontal and vertical directions. The duration of soil improvement was dominated by the low hydraulic permeability of this layer. (v) Homogeneous organic clay with properties similar to the previous layer with slightly lower clay and water contents, 5.0 m thick; and finally (vi) silty clay and sandy silt was found.

The hydraulic fill was highly compressible and had extremely low shear strength, making it impossible to walk on the surface. The soil water content was higher than the liquid limit in all layers along with very low undrained shear strengths and large void ratios.

The treatment area was divided into six divisions based on the design loads of the structures to be built; the six divisions are shown in Figure 2.20(b) by Roman numerals I–VI. The vacuum pressure was applied on 72 individually sealed subdivisions, marked as 1–72. The treatment areas (subdivisions) ranged from 5000 to 30000 m². To compare with the results of vacuum preloading, four control areas (50 m × 50 m), marked S-1 to S-4 in Figure 2.20(b), were designated. Surcharge fill equivalent to 97 kPa was applied in three steps over these areas. The design criteria are summarized in Table 2.2.
Table 2.2: Design criteria of soil improvement at Xingang Port (Shang et al., 1998).

<table>
<thead>
<tr>
<th>Division (see Fig. 2b)</th>
<th>I</th>
<th>II</th>
<th>III</th>
<th>IV</th>
<th>V</th>
<th>VI</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design load $p_a$ (kPa)</td>
<td>50</td>
<td>50</td>
<td>8*</td>
<td>83</td>
<td>80</td>
<td>80</td>
</tr>
<tr>
<td>Method of treatment</td>
<td>Vacuum</td>
<td>Vacuum</td>
<td>Vacuum - preloading</td>
<td>Vacuum - preloading</td>
<td>Vacuum - preloading</td>
<td>Vacuum - preloading</td>
</tr>
<tr>
<td>Preloading pressure $p_r$ (kPa)</td>
<td>80</td>
<td>80</td>
<td>80 - 1*</td>
<td>80 - 1*</td>
<td>80 - 1*</td>
<td>80 - 1*</td>
</tr>
<tr>
<td>$c_u$ (up to 10 m depth) (kPa)</td>
<td>215</td>
<td>215</td>
<td>220</td>
<td>220</td>
<td>220</td>
<td>220</td>
</tr>
<tr>
<td>Residual settlement under design load (cm)</td>
<td>≤20</td>
<td>≤20</td>
<td>≤30</td>
<td>≤35</td>
<td>≤35</td>
<td>≤35</td>
</tr>
</tbody>
</table>

* Below 10 m depth, $c_u = c_u^* = 0.2c_u$, where $c_u^*$ is the initial undrained shear strength and $p_r$ is the effective overburden pressure.

During treatment, extensive instrumentation, including surface settlement and vertical deformation with depth, lateral deformation, pore pressure, and vacuum pressure, was implemented in three testing areas (subdivisions 12–13, 44, and S-2), representative of the vacuum, vacuum plus surcharge, and surcharge treatments, respectively.

The operation works where conducted as follow:

- Pretreatment of the soil surface, including manual placement of two layers of twig mats 0.7 m of granular material.
- Prefabricated vertical drains were embedded to depths between 16 and 20 m. In some areas where the thick soft soil layer was encountered, the depth was extended to 25 m. The spacing of the vertical drains was 1.3 m arranged in a square pattern.
- The interconnected perforated pipe system was placed on top of the sand layer. All pipes were steel or PVC, 76 mm in diameter, 6 m long, and wrapped with filter cloth. The average spacing between the pipes was 6 m.
- Polyethylene membrane liners were anchored into trenches, using a clayey soil which was compacted manually, to provide airtight seals. To ensure a proper seal, the anchor trench reached the organic clay deposit underneath pretreatment the surface.
- After installations were completed, the pumping system was tested for 6 h. The vacuum preloading began in January 1988 and was completed in November 1989. A vacuum pressure of 80 kPa under the membrane was considered full capacity, and it was reached within 15 days in most subdivisions, being the shortest time 1 day and the longest 58 days. The vacuum pressure was uniformly distributed and stabilized after 60 days.
- Based on past experience, treatment was terminated when the settlement rate was less than 1 mm/day over a period of 10 days. The average treatment time was 135 days in divisions I and II where only vacuum was used, and 175 days in divisions III–VI where
combined vacuum and 1 m surcharge fill were used. In some subdivisions, the treatment was extended up to 247 days.
- A layer of refill material was placed after the preloading treatment to bring the surface elevation up to +5.4 m.

The vacuum pumps used in the project had a capacity of 7.5 kW and were capable of generating a vacuum pressure of 80 kPa over an area of 1000–1500 m$^2$.

At the end of the operation, the total settlement (i.e. pretreatment and vacuum preloading) ranged from 1.6 to 2.3 m. Figure 2.22 shows a contour map of the settlement distribution. The settlement during the pretreatment stage, after PVD installation, ranged from 0.6 to 1.2 m. The settlement generated during treatment in divisions I and II, where only 80 kPa vacuum pressure was applied, ranged between 1.0 – 1.2 m. In other divisions, where the total combined vacuum-surcharge pressure was 97 kPa, the settlement ranged between 1.1 – 1.4 m. Settlement in the four control areas (S-1, S-2, S-3, and S-4) in divisions IV and V did not differ from the settlement were vacuum was applied, indicating the vacuum and surcharge preloading have generated similar consolidation effects.

Figure 2.22: Contour map showing total settlement due to the combined effects of pretreatment settlement and vacuum preloading. All contour lines are in metres (Shang et al., 1998).

The lateral displacements of the soil after treatment were measured using inclinometers at testing area subdivision 44 and control area S-2 (Figure 2.20(b)). The applied pressure was
97 kPa in both tests. Figure 2.23 shows the distribution of lateral displacement with depth measured at the border of the treatment area. Vacuum preloading generated up to 300 mm of inward lateral displacement, in contrast with the surcharge preloading which induced an outward lateral displacement of up to 470 mm. From these results, it can be inferred that the vacuum method does not impose the threat of bearing capacity failure due to rapid surcharge loading. However, it should be noted that the inward lateral displacement could generate tension cracks adjacent to the treatment area.

![Lateral Displacement with depth at Xingang Port (Shang et al., 1998).](image)

After treatment, the average decreases in soil water content with depth were 17.3, 16.3, and 22.5% in subdivisions 12-13, 44, and S-2, respectively, with large standard deviations (up to 17%) attributable to the complexity of the soil conditions. The decreases in void ratio were 16.2, 14.9, and 19.9% in subdivisions 12–13, 44, and S-2, respectively, consistent with the decreases in soil water content. The decreases in water content and void ratio became insignificant below elevation −8 m in subdivisions 12–13. On the other hand, in subdivisions 44 and S-2, the effective treatment depth was extended to elevation −10 m.

The soil strength after treatment was evaluated via undrained triaxial tests (UU tests), field vane tests and the cone penetration test (CPT). In subdivisions 12–13, the results from the UU tests and field vane tests were in good agreement, showing increases ranging from
1700% at the top to approximately 30 - 40% at the bottom. The design criteria in Table 2.2 were fulfilled over the entire depth. The CPT results indicated significant increases in the cone resistance down to elevation -12 m, whereas no improvement was registered below elevation -13 m, which agrees with the water content and void ratio data.

In subdivision 44, good agreement was found between the results of UU tests and vane tests. The percent increase in shear strength ranged from 33% at the bottom (elevation -13 m) to 2327% at the surface (elevation +5.5 m). The design criteria listed in Table 2.2 were fully achieved. The results of the CPT tests were consistent with the decreases in water content and void ratio and increases in the undrained shear strengths. Subdivisions 44 and S-2, where 97 kPa was applied using combined vacuum-surcharge preloading and surcharge alone, respectively, showed nearly identical results. Therefore, it is confirmed that the design preloading pressure of 97 kPa was realized through the combined application of vacuum and 1 m surcharge fill.

2.2.4.3 Embankment stabilization in Ishikari, Japan (Shiono et al., 2001)

A vacuum consolidation experiment was conducted at the Ishikari lowland zone about 5 m above sea level, in Japan. The soil profile at the experimental site is shown in Figure 2.24(a). The authors referred to the method as ‘compact vacuum consolidation’ (CVC). The experiment was carried out in a large area along other stabilization techniques, such as embankment loading (Figure 2.24(b)).

The vacuum consolidation test area covered 20870 m², and was divided into 13 blocks for practical purposes. Vertical drains were installed to a depth of 20 m and a granular bed was placed on the surface with a thickness varying from 0.7 to 0.9 m. An embankment was continually placed on the surface, starting 42 days after the vacuum pumps began operating. The 9.9 m embankment was completed 108 days later with an average speed of 16.5 cm/day (Figure 2.25(a)). This speed contrasts with the speed in the surcharge area which was 6 cm/day on the 36th day after having laid sand mats. On day 88, on the surcharge area, approximately 3.0 m height had been achieved and the works stopped at this stage because signs of instability were observed.
Figure 2.24: Experimental area at Ishikari, Japan: (a) Soil profile, (b) plan view (Shiono et al., 2001).

Figure 2.25: Experiment results against time: (a) fill height; (b) vacuum pressure and (c) settlement and volume of water extracted (Shiono et al., 2001).
Figure 2.25(b) shows the vacuum at the pump and below the air tight sheet. Initially the vacuum under the sheet varied between 60 and 70 kPa. It was noticed that the vacuum level increased after the filling, which could be attributed to an increase in the air-tightness.

Settlement on each layer and dehydration (volume of water extracted) are shown in Figure 2.25(c). The settlement before the embankment was 1.42 m; afterwards, there was rapid increase reaching 4.05 m when pumping stopped. The combined compression of peat and peaty clay accounted for 51% of the total subsidence, when these layer account for less than half the profile. The dehydration rate was almost proportionally inverse to the settlement. The authors reported that for the surcharge, the settlement was about 0.5 m after the 3.0 m, rapidly increasing thereafter. The maximum settlement measured was approximately 1.2 m at 480 days.

![Figure 2.25: Vacuum at the pump and below the air tight sheet.](image)

**Figure 2.26**: Pore water pressure variation at Ishikari (Shiono et al., 2001).

Figure 2.26 show the variation on the pore water pressure for each layer. The PWP decreased immediately after the pumps started operating on the peat, peaty clay and slity clay; increasing again immediately after filling began, and finally decreasing again when filling stopped. After the pumps were stopped the rate of PWP dissipation decreased on these layers. On the silty sand however, the increase in pore pressure was insignificant during filling and remaining nearly constant after it. For the surcharge area, the PWP increased steadily until a few days after the 3.0 m fill was completed, and subsequent dissipation was small. On the silty sand layer, there was a small increase and subsequent decrease to about -15 kPa, indicating drainage through this layer.
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Lateral displacement during vacuum consolidation is shown in Figure 2.27. Immediately after pumping started, lateral inwards deformation appeared in the peat layers. The top layer kept deforming inwards even after filling; however, the peaty clay, silty sand and the top of the silty clay gradually deformed outwards. On the other hand, on the surcharge area outwards deformation occurred from the beginning and increased as the filling went on.

![Figure 2.27: Horizontal displacement at Ishikari (Shiono et al., 2001).](image)

2.2.4.4 Embankment construction in Sapporo, Japan (Hayashi et al., 2002)

A construction test using vacuum consolidation was performed, on National Highway Route 337 on the suburbs of Sapporo in Japan, to build a high embankment over peaty ground. Figure 2.28 presents the soil profile at the site of the test. The water content for the peat and clayey peat layers ranged from 200% to 700% and the CPT resistance from 100 – 300 kPa.

Vertical drains were installed in a square pattern with 0.8 m spacing, to a depth of up to 19.9 m. Two vacuum pumps were used to improve an area of approximately 3200 m². Ground behaviour was observed using devices such as a settlement plate, a differential settlement gauge, a piezometer and a borehole inclinometer.
The embankment construction commenced 21 days after the pumps began operation with an average construction speed of 13 cm/day. The embankment height was 10.0 m.
Figure 2.29 shows that settlement progresses even after stopping of the vacuum pumps, though the progress is gradual. By soil layer, the settlement of the underlying clayey peat layer lags behind that of the other layers. The excess PWP in the peat layer before construction of the embankment was approximately -50 kN/m². The pressure in the lower soil layers, however, remained at around -20 kN/m².

The design value of negative pressure was set at 60 kN/m². However, the value was only fulfilled for the surface peat layers. In the deeper layers, only negative pressures of approximately one third of the design value were observed. The authors consider that this may be due to the effect of the fine sand layer between the upper and lower clay layers.

Inclinometer measurement immediately before construction of the embankment, upon completion of construction, at the time of pumping stoppage and 25 days after stopping of the pumps are shown in Figure 2.30. After the start of pumping and before construction, lateral displacement was towards the inside of the embankment, between the peat layer and the upper clay layer. The peat layer showed a more noticeable trend of displacement with a maximum value of approximately 150 mm. During construction of the embankment, lateral displacement occurred toward the outside of the embankment, but it was only about 250 m at the maximum. No behaviour was observed that indicated progress in shear deformation after stopping of the vacuum pumps.

![Figure 2.30: Lateral displacement with depth at Sapporo (Hayashi et al., 2002).](image)
The authors compared the improvement results from two other different sites with very similar ground conditions, using different countermeasures to accelerate consolidation (Table 2.3). The difference in embankment height and construction speed should be considered when making comparisons with the site to which the vacuum consolidation method was applied.

Regarding the number of days it took to achieve 90% consolidation (Figure 2.31), the settlement ceased in a short period of time in the cases of plastic drains with spacing of 0.7 and 0.9 m, as well as for the vacuum consolidation method. However, when the spacing of the PVDs was 1.1 m, the time to achieve this value was very high and close to that of the non-countermeasure case, indicating almost no consolidation acceleration effect.

Table 2.3: Construction conditions at different sites (Hayashi et al., 2002).

<table>
<thead>
<tr>
<th>Method</th>
<th>Spacing of vertical drain (m)</th>
<th>Embankment construction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Height (m)</td>
</tr>
<tr>
<td>Non-countermeasure</td>
<td>---</td>
<td>4.2</td>
</tr>
<tr>
<td>PVD</td>
<td>0.7</td>
<td>4.2</td>
</tr>
<tr>
<td></td>
<td>0.9</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.1</td>
<td></td>
</tr>
<tr>
<td>Vacuum consolidation</td>
<td>0.8</td>
<td>10.3</td>
</tr>
</tbody>
</table>

Figure 2.31: Number of day before reaching 90% consolidation for different drainage conditions in peat grounds (Hayashi et al., 2002).
Finally, the authors concluded that no improvement effect is achieved unless the PVD spacing is 0.9 m or shorter.

2.2.5 Laboratory testing

Laboratory testing on vacuum consolidation has not been as widely implemented as field applications. Nonetheless, in the last 15 years several laboratory investigations have been presented, introducing new and modified apparatuses specifically for testing the geotechnical properties of soils under vacuum loading, with and without radial drainage. A list of some different equipment, used in several studies, found in the literature is presented in Table 2.4. Selected laboratory studies are presented here.

Table 2.4: Vacuum consolidation equipment found in the literature

<table>
<thead>
<tr>
<th>Equipment</th>
<th>Diameter / height (mm)</th>
<th>Central drain</th>
<th>Load type</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Triaxial cell</td>
<td>100 / Unspecified</td>
<td>Yes</td>
<td>Cell and drain pressure difference</td>
<td>(Harvey, 1997)</td>
</tr>
<tr>
<td>Consolidation cell</td>
<td>70 / 25</td>
<td>No</td>
<td>Vacuum, surcharge and combined Vac-Surch</td>
<td>(Mohamedelhassan and Shang, 2002)</td>
</tr>
<tr>
<td>Large-scale consolidometer</td>
<td>450 / 950</td>
<td>Yes</td>
<td>Vacuum, surcharge and combined Vac-Surch</td>
<td>(Indraratna and Redana, 1998)</td>
</tr>
<tr>
<td>Oedometer</td>
<td>60 / 20</td>
<td>No</td>
<td>Vacuum or surcharge</td>
<td>(Chai et al., 2005)</td>
</tr>
<tr>
<td>Triaxial cell</td>
<td>74 / 150</td>
<td>Yes</td>
<td>Vacuum or surcharge</td>
<td>(Kawabata et al., 2006)</td>
</tr>
<tr>
<td>Large-scale consolidometer</td>
<td>450 / 900</td>
<td>Yes (multiple)</td>
<td>Vacuum, surcharge and combined Vac-Surch</td>
<td>(Chai et al., 2007a)</td>
</tr>
<tr>
<td>Triaxial cell</td>
<td>100 / 200</td>
<td>No</td>
<td>Vacuum, surcharge and combined Vac-Surch</td>
<td>(Mahfouz et al., 2007)</td>
</tr>
<tr>
<td>Large-scale consolidometer</td>
<td>305 / 500</td>
<td>Yes</td>
<td>Vacuum, surcharge and combined Vac-Surch</td>
<td>(Saowapakpiboon et al., 2010)</td>
</tr>
<tr>
<td>Large-scale consolidometer</td>
<td>450 / 950</td>
<td>Yes</td>
<td>Vacuum, surcharge and combined Vac-Surch</td>
<td>(Saowapakpiboon et al., 2011)</td>
</tr>
</tbody>
</table>

2.2.5.1 Modified triaxial cell - Imperial College (Harvey, 1997)

A triaxial cell (Figure 2.32) was used in Imperial College to conduct a series of experiments on soft sediments found at seabed to gain information on implementing vacuum consolidation at Chek Lap Kok Airport in Hong Kong.
In the testing procedure, a plastic wick of 6 mm diameter was surrounded by a 100 mm soil cylinder. Two soils were tested: firstly, pure, dry silt was rained into a water filled membrane; secondly, a silt-kaolin slurry at 52.5% water content was used. An isotropic confining stress was applied to each sample and a lower pressure was kept in the drain to implement the vacuum drainage principle. The PWP was monitored at three points on the periphery of the sample (Figure 2.32).

The clayey silt sample was prepared with 70% silt : 30% kaolin to model the soil found at Chek Lap Kok. When the pressure difference was applied, the soil around the drain stiffened considerably, while almost no consolidation was observed at the edge. This was
due to a non-linear reduction of soil permeability as consolidation progressed, shrinking
the pores around the drain, under increasing effective stress. Figure 2.33 shows the water
content in the vicinity of the drain and in the outer edge. It can be seen that the water
contents are lower close to the drain and towards the base of the sample.

2.2.5.2 One-dimensional vacuum consolidation apparatus (Mohamedelhassan and
Shang, 2002)

As mentioned in §2.2.2.2, Mohamedelhassan and Shang (2002) designed and
manufactured an apparatus for the parametric study of one-dimensional vacuum
consolidation (Figure 2.34).

The apparatus characteristics are:

- A stainless steel vacuum cell (Figure 2.35(a)).
- A soil sample size of 70 mm in diameter and 25 mm in height (Figure 2.35(b)).
- Independent application of vacuum (maximum 80 kPa) or surcharge, or a combined
  vacuum-surcharge pressure.
- A vacuum line with vacuum regulator, vacuum gauge and pressure transducer; allowing
  adjustment of desired vacuum level and ensuring a stable pressure during the test.
- A volume change tube placed in the vacuum line is connected to the top of the cell.
• The vacuum cell is mounted on a loading frame that allows the application of a vertical levered surcharge load through a hanger to the soil sample (Figure 2.34).
• The remaining instrumentation comprises: a second pressure transducer at the base of the cell to measure the changes in PWP, an electronic dial gauge on the loading piston to record settlement, and a thermometer to monitor the room temperature.
• All the instrumentation is connected to a data acquisition system to record measurements during the test.

![Diagram of vacuum cell](image)

Figure 2.35: (a) vacuum cell, (b) soil ring (Mohamedelhassan and Shang, 2002).

According to the authors, one of the challenges in the design of the vacuum cell was to ensure an airtight seal and yet allow a negligible friction between the piston and the wall of the cell. The airtight seal was achieved by using an O-ring as show in Figure 2.35(a), whereas a very smoothly finished piston and inner cell wall lubricated with silicon grease significantly reduced the friction.

Four sets of tests were conducted to examine the vacuum and surcharge combined one-dimensional consolidation model developed by the authors (see §2.2.6), and to evaluate the soil parameters during the consolidation process.
Table 2.5 shows the properties of the soils tested. Reconstituted soil samples were used in the first three sets of tests (W1 to W3), whereas reconstituted-consolidated samples were used in the fourth set of tests (O1).

The reconstituted soil specimen was prepared with the water content between 1 and 1.5 times the liquid limit. The sample was carefully placed in small patches in the soil ring and gently compacted with a thin rod to remove any noticeable air voids. However, it is possible that soil samples prepared by this method may have not been fully saturated at the beginning of testing.

Table 2.5: Soil properties of tested samples (Mohamedelhassan and Shang, 2002).

<table>
<thead>
<tr>
<th>Test</th>
<th>Clay (%)</th>
<th>Silt (%)</th>
<th>Sand (%)</th>
<th>W_L (%)</th>
<th>W_p (%)</th>
<th>PI (%)</th>
<th>Gs</th>
</tr>
</thead>
<tbody>
<tr>
<td>W1 to W3</td>
<td>28</td>
<td>60</td>
<td>12</td>
<td>31.5</td>
<td>19.9</td>
<td>11.6</td>
<td>2.73</td>
</tr>
<tr>
<td>O1</td>
<td>64</td>
<td>35.9</td>
<td>0.1</td>
<td>63 - 70</td>
<td>31 - 39</td>
<td>31 - 32</td>
<td>2.8</td>
</tr>
</tbody>
</table>

The reconstituted-consolidated samples were prepared using a specially designed consolidation column, 150 mm in diameter and 450 mm in height capable of consolidating clay slurry to a soil mass under a specific consolidation pressure. The slurry was mixed to a water content approximately twice the liquid limit (120%) and then gradually loaded up to 60 kPa in small load intervals over a period of 14 days. Four specimens were trimmed and laid on a smooth flat surface. Finally, the soil ring was gently pushed via its cutting shoe edge against the specimen and the excess soil removed. It is worth noting that the initial water contents of the four specimens vary slightly, as they were dependent on the specimen location in the consolidation column.

The testing procedure was as follows:

- Boil two porous stones in distilled water until saturated, saturate the lines in the base, and calibrate transducers with the standard triaxial test procedure.
- A saturated porous stone was placed at the base of the cell followed by a wet filter paper
- The soil ring, containing the sample, was then placed on the filter paper followed by another wet filter paper and the second porous stone.
• The upper part of the cell was put in place and the piston was lubricated and pushed carefully in it. As the O-ring passed into the upper part, the piston slid gently under its own weight and rested at the upper porous stone.
• A ring cap was placed on top of the cell and tightened to hold the cell together.
• The line between the upper porous stone and the volume change tube was saturated.
• The cell was then placed on the loading frame and connected to the vacuum line with the valve turned off.
• The instrumentation was connected to the data acquisition system.
• The vacuum pressure was adjusted to the desired value and the valve open.
• A surcharge pressure could be applied prior to or at any time during the vacuum testing.
• During testing the room temperature was monitored. Fluctuations were less than 1°C.

Two test series, W1 and W2, were performed to study the excess PWP and settlement with vacuum pressure. The loading program for these series is shown in Table 2.6.

<table>
<thead>
<tr>
<th>No.</th>
<th>Test series W1 pressures (kPa)</th>
<th>Test series W2 pressures (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Vacuum / Surcharge</td>
<td>Vacuum / Surcharge</td>
</tr>
<tr>
<td>1</td>
<td>20 / 0</td>
<td>30 / 0</td>
</tr>
<tr>
<td>2</td>
<td>0 / 20</td>
<td>0 / 30</td>
</tr>
<tr>
<td>3</td>
<td>20 / 20</td>
<td>30 / 30</td>
</tr>
<tr>
<td>4</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

As an example, the results obtained for series W1 are presented in Figure 2.36. Similar results were observed for series W2 and can be seen in the original paper.

From Figure 2.36(a), it is shown that surcharge loading generates a positive excess pore-water pressure that dissipates to zero with time; vacuum preloading generates a negative pore pressure that approaches the applied pressure with time; and a combined surcharge-vacuum preloading is a superposition of the first two above, i.e., the consolidation starts with an excess pore-water pressure equivalent to the surcharge pressure and ends with a negative pore pressure equivalent to the applied vacuum pressure. The settlements
presented in Figure 2.36(b) indicate that the nature of the consolidation pressure, either surcharge or vacuum, has no bearing on soil consolidation.

The soil consolidation parameters were further investigated in test series W3. The first sample W3.1, was consolidated at vacuum pressures of 15, 30, 45, and 60 kPa and unloaded to 15 kPa. The second sample W3.2, was loaded and unloaded with surcharge pressures of the same magnitude. The soil sample W3.3 was consolidated with a vacuum and surcharge combined pressure of 15 + 15 kPa, 30 + 30 kPa, 45 + 45 kPa, 60 + 60 kPa and unloaded to 15 + 15 kPa. The results for this series are shown in Figure 2.37.

Figure 2.37(a) shows that the nature of the applied consolidation pressure (vacuum, surcharge, or both combined) has no bearing on the compression behaviour of the soil. The soil demonstrated classic virgin compression behaviour. Figure 2.37(b) shows the coefficient of consolidation, $c_v$, versus the applied pressure for test series W3. The figure illustrates that a vacuum pressure and a surcharge pressure of the same magnitude produce similar coefficients of consolidation. The figure also shows that the coefficient of consolidation obtained by a combined vacuum-surcharge preloading is similar to the one obtained by a vacuum or surcharge pressure of the combined magnitude.

The behaviour of overconsolidated soils under different combined vacuum-surcharge pressures was studied in series O1. The loading program is presented in Table 2.7.
Figure 2.37: Tests series W3 results (a) e-log s curve, (b) coefficient of consolidation versus stress (Mohamedelhassan and Shang, 2002).

Table 2.7: Loading program test series O1

<table>
<thead>
<tr>
<th>No.</th>
<th>Test series O1 pressures (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Vacuum / Surcharge</td>
</tr>
<tr>
<td>1</td>
<td>30 / 2</td>
</tr>
<tr>
<td>2</td>
<td>30 / 15</td>
</tr>
<tr>
<td>3</td>
<td>30 / 30</td>
</tr>
<tr>
<td>4</td>
<td>30 / 40</td>
</tr>
</tbody>
</table>

Excess PWP measurements for test series O1 are presented in Figure 2.38. The figure shows that the pore water pressures measured at the bottom of the soil samples converge to the applied vacuum pressure by the end of the test, indicating that vacuum preloading is equally effective under various surcharge pressures.
Chapter 2

LITERATURE REVIEW

In general, the results showed that the vacuum pressure generates the same effect as that of a surcharge pressure under one-dimensional conditions. Hence, the soil compressibility characteristics obtained in conventional consolidation tests can be used in the design of vacuum consolidation. However, the authors cautioned that the lateral displacement of soil is different between the surcharge and vacuum preloading, as reported in case studies.

2.2.5.3 Ground deformations due to vacuum consolidation (Chai et al., 2005)

A Maruto Multiple Oedometer was used by the researchers to study the ground deformations induced by vacuum consolidation. The experimental set-up is shown in Figure 2.39. Each sample was 60 mm in diameter and typically 20 mm thick. The soil tested was reconstituted Ariake clay, which was preconsolidated under a pressure of 30 kPa. The physical properties of the sample are listed in Table 2.8. Before the start of each consolidation test, the soil sample was saturated to have a B value of greater than 0.9.

A series of laboratory oedometer tests with one-way drainage conditions were conducted under either vacuum pressure or surcharge loading, on samples with different initial

Figure 2.38: Excess PWP versus time for test series O1 (Mohamedelhassan and Shang, 2002).
effective stresses. The samples were subjected to different loading conditions as shown in Table 2.9. In tests with nonzero initial effective stress, the sample was first consolidated under a predetermined stress for 24 h and then an incremental surcharge load or vacuum pressure was applied, and the settlement and excess pore pressure at the bottom of the sample were monitored (top of the sample was a free drainage boundary). Considering that in the field the maximum achievable vacuum pressure is about 80 kPa, for all tests the incremental surcharge load and/or vacuum pressure was limited to this value.

![Diagram](image)

**Figure 2.39:** Laboratory set-up schematic (Chai et al., 2007b)

<table>
<thead>
<tr>
<th>Clay (%)</th>
<th>Silt (%)</th>
<th>Sand (%)</th>
<th>w (%)</th>
<th>w_L (%)</th>
<th>w_p (%)</th>
<th>e_0</th>
</tr>
</thead>
<tbody>
<tr>
<td>31</td>
<td>68.7</td>
<td>1.2</td>
<td>97.1</td>
<td>116.5</td>
<td>57.5</td>
<td>2.32</td>
</tr>
</tbody>
</table>

Table 2.8: Ariake clay geotechnical properties (Chai et al., 2005).

Comparison curves of the settlement versus time for cases 1-a and 1-b, 2-a and 2-b, and 3-a and 3-b are given in Figure 2.40 to Figure 2.42, respectively. From this results, it can be seen that when the initial vertical effective stress is low (0 and 40 kPa), the vacuum pressure induced settlement is less than that observed under the corresponding surcharge load (cases 1 and 2). For case 3 where the initial vertical effective stress is 80 kPa, the settlements induced by vacuum pressure and surcharge load are almost the same.

The authors in this study observed that when disassembling the apparatus, the specimens that were subjected to vacuum pressures higher than the initial effective stress, had separated from the confining ring. This showed the occurrence of radial inward movement.
in laboratory test specimens subjected to vacuum consolidation. This phenomenon was explained earlier in §2.2.2.2.

Table 2.9: Loading and drainage boundary conditions for laboratory tests (Chai et al., 2005).

<table>
<thead>
<tr>
<th>Case</th>
<th>Initial Stress (kPa)</th>
<th>Surcharge (kPa)</th>
<th>Vacuum (kPa)</th>
<th>Drainage condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-a</td>
<td>0</td>
<td>80</td>
<td>-</td>
<td>One way</td>
</tr>
<tr>
<td>1-b</td>
<td>-</td>
<td>-</td>
<td>80</td>
<td>One way</td>
</tr>
<tr>
<td>2-a</td>
<td>40</td>
<td>80</td>
<td>-</td>
<td>One way</td>
</tr>
<tr>
<td>2-b</td>
<td>-</td>
<td>-</td>
<td>80</td>
<td>One way</td>
</tr>
<tr>
<td>3-a</td>
<td>80</td>
<td>80</td>
<td>-</td>
<td>One way</td>
</tr>
<tr>
<td>3-b</td>
<td>-</td>
<td>-</td>
<td>80</td>
<td>One way</td>
</tr>
</tbody>
</table>

Figure 2.40: Settlement time curves of cases 1-a and 1-b (Chai et al., 2005).

Figure 2.41: Settlement time curves of cases 2-a and 2-b (Chai et al., 2005).
2.2.5.4 Laboratory studies in PVD incorporating vacuum loads

Indraratna and Redana (1998) developed a large-scale radial drainage consolidation cell, presented in Figure 2.43. Indraratna and his team have conducted several laboratory studies to model the improvement of soils using PVDs and vacuum consolidation (Bamunawita, 2004; Indraratna and Rujikiatkamjorn, 2004; Rujikiatkamjorn et al., 2008).

The large-scale consolidation cell consists of two half sections made of stainless steel (450 mm in internal diameter and 950 mm in height). In order to eliminate the friction along the cell boundary, a 1.5 mm thick ultra-smooth Teflon sheet was inserted around the periphery. As a result, the height and diameter (h/d) ratio of the cell can be much higher than a conventional oedometer, enabling the appropriate testing of a mandrel-driven, prefabricated vertical drain (PVD). The loading system is equipped with an air jack compressor system via a rigid piston. Water and air tightness of piston is achieved using an O-ring system on its periphery, and lubricated with grease to reduce friction. In the loading calibration, over 97% loading was transferred to soil sample. A displacement transducer connected to a data logger is located at the top of the piston. A vacuum pump capable of applying suction up to -100 kPa is used above the PVD. The cell is also equipped with a specially designed steel hoist and guider from which a PVD can be inserted vertically along the central axis of the cell.
The settlement was measured by a LVDT placed at the top of the piston. Six calibrated diaphragm-type piezometers were installed at 3 different depths (i.e., top, middle and bottom of the cell) to measure the pore-water pressures at various points. The fully saturated piezometer tips were kept in position using thin rigid stainless steel tubes. The LVDT and pore pressure transducers were connected to a common data logger.

Rujikiatkamjorn and Indraratna (2006a) used reconstituted alluvial clay from Moruya (New South Wales) for a series of tests. The geotechnical properties of sample are shown in Table 2.10.

<table>
<thead>
<tr>
<th>Clay (%)</th>
<th>Silt (%)</th>
<th>w (%)</th>
<th>w_p (%)</th>
<th>PI (%)</th>
<th>γ_s (kN/m^3)</th>
<th>e_0</th>
</tr>
</thead>
<tbody>
<tr>
<td>40 – 50</td>
<td>~ 60</td>
<td>45</td>
<td>17</td>
<td>28</td>
<td>17</td>
<td>1.14</td>
</tr>
</tbody>
</table>

Three different series of tests were conducted the improvement of soft clay via PVD and vacuum consolidation. In the first series of tests (Test SP) a typical surcharge load of 30 kPa was applied. For the second series (Test VP), only a vacuum pressure of 40 kPa was
applied. For the third series (Test SV), a vacuum pressure of 40 kPa together with a 30 kPa surcharge load was applied to further accelerate consolidation.

The test procedure was as follows:

- Clay preparation by mixing the material thoroughly using a mechanical mixer with a water content slightly greater than the liquid limit.
- The clay was placed and compacted in layers in the apparatus. An initial preconsolidation pressure of 20 kPa was applied for 7 days before the installation of the vertical drain.
- A 100 mm × 3 mm band drain (equivalent drain radius of 32.5 mm) was then installed vertically using a steel mandrel. After the installation, the mandrel was withdrawn and an initial precompression stress of 20 kPa was immediately applied for all tests.
- For Test SP the large clay sample was further subjected to additional surcharge loading in increments of 30 kPa.
- In the case of Test VP, the sample was loaded with an application of vacuum pressure of 40 kPa only.
- In the case of Test SV, the clay was loaded axially in increments of 30 kPa, together with a vacuum pressure of 40 kPa.
- The duration of each test was approximately 35 days.
- The corresponding excess pore pressure and settlement behaviours were recorded and plotted.

Figure 2.44 illustrate the measured average excess pore pressures and surface settlements for all tests. It is clearly shown that the combination of vacuum and surcharge pressure maximises the settlement, and this settlement depends on the magnitude of the applied surcharge and vacuum. In Figure 2.44(b), positive excess pore pressure is observed when the surcharge pressure is employed (Tests SP and SV). The measured excess pore pressures become negative for the tests with the vacuum application (Tests VP and SV). The measured ultimate negative average excess pore pressure is approximately 80% of the applied vacuum pressure due to vacuum loss along the drain length. This is in accordance with previous laboratory observations made by Indraratna et al. (2004).
2.2.6 Analytical and numerical models in vacuum consolidation

The traditional method for modelling the soil behaviour under vacuum consolidation, is to introduce a surcharge load in the surface of the ground equivalent to the vacuum load applied (Park et al., 1997). This method is able to predict the settlement in an acceptable manner, but fails to model the pore pressure behaviour due to the variation of the total stress introduced by the surcharge load (Indraratna et al., 2004). At present, there is no commercial software available that is capable of conducting vacuum consolidation analysis. Even though in-depth numerical modelling is out of the scope of this thesis, a brief introduction to the subject is presented.

Different analytical and numerical methods have been proposed for modelling soil behaviour under vacuum consolidation in the last decade. The majority of methods include vacuum and PWP distribution analysis along the drain, and settlement and degree of consolidation calculations.

Mohamedelhassan and Shang (2002) initially developed a combined vacuum and surcharge one-dimensional consolidation model based on laboratory tests (see §2.2.5.2) and Terzaghi’s consolidation theory. In this study, it was considered that the excess pore water pressure generated by a combined vacuum-surgecharge preloading (Figure 2.45(a)) may be evaluated by the law of superposition as the sum of the excess pore water pressure
generated by the surcharge (Figure 2.45(b)) and the pore water pressure reduction induced by the vacuum preloading (Figure 2.45(c)). The average degree of consolidation for combined vacuum and surcharge preloading can then be expressed by equation (2.25), where $c_{vc}$ is the combined vacuum-surcharge coefficient of consolidation.

$$U_{ave} = 1 - \sum_{n=0}^{\infty} \frac{8}{(2n + 1)^2 \pi^2} \exp \left[ - \frac{(2n + 1)^2}{4 \pi^2} \frac{c_{vc} t}{H^2} \right]$$  (2.25)

However, this theory does not consider the effect of introducing vertical drains in the ground. Indraratna et al. (2004) conducted a series of tests in a large-scale consolidation cell (see §2.2.5.4) introducing PVDs. Piezometer readings along the drains confirmed that the suction head propagates to the bottom of the drain but is less than the maximum applied at the top (Figure 2.46). Based on these observations, Indraratna et al. (2005b) proposed analytical solutions for vacuum consolidation with vertical drains, in both the axisymmetric and equivalent plain strain conditions of a single cell unit, and incorporated these solution in a single drain finite element model.

Other authors have also study radial consolidation including vacuum loading. Most of the proposed solutions employ conversions to the two dimensional plane strain condition, although some studies have presented three dimensional analysis (Mahfouz et al., 2006; Rujikiatkamjorn and Indraratna, 2007a). Lee et al. (2006) introduced a numerical simulation for the analysis of vacuum consolidation with horizontal drains. Tran and
Mitachi (2007) used a plain strain analysis to model a vacuum-embankment test on peaty ground.

![Diagram of suction distribution along the vertical drain](image)

**Figure 2.46**: Suction distribution along the vertical drain (a) measured in laboratory testing; and (b) assumed in analytical model (Indraratna et al., 2005b).

Normally the analytical and numerical models involve the creation and implementation of appropriate subroutines in commercial numerical software such as ABAQUS, CRISP or Mathcad (Bamunawita, 2004; Rujikiatkamjorn, 2005; Tran and Mitachi, 2008; Chai et al., 2010), or the development of own software (Saowapakpiboon et al., 2010); for conducting the calculations.

Detail explanation of some analytical and numerical solution for vacuum consolidation can be found in the literature given above and in Qian et al. (1992), Indraratna et al. (2010a; 2012), Chai et al. (2005; 2006a; 2008), Chu and Yan (2005a; 2005b), Yan and Chu (2005), Rujikiatkamjorn et al. (2007), Rujikiatkamjorn and Indraratna (2006a; 2006b), Indraratna (2009), Saowapakpiboon et al. (2011) and Geng et al. (2012).

### 2.3 ONE-DIMENSIONAL COMPRESSIBILITY OF PEAT

Laboratory studies on the compressibility of peat have been carried worldwide by several authors such Hanrahan (1954; 1964), Berry and Poskitt (1972), Landva and La Rochelle (1983), Lefebvre et al. (1984), Fox et al. (1992), Fox and Edil (1996), Mesri et al. (1997),

It is well accepted that the one-dimensional compression of mineral and organic soils, such as peat, is due to (i) volume changes associated with the dissipation of excess pore pressures, and (ii) volume changes due to creep (O'Loughlin, 2001). It is to note that many authors consider that these two phenomena occur simultaneously (Bjerrum, 1967; Hobbs, 1986; Zeevaert, 1986; Kim and Leroueil, 2001).

**Figure 2.47:** In situ settlement-time curves showing tertiary compression for (a) 3.0 m embankment, (b) 2.0 m embankment (Candler and Chartres, 1988).
Furthermore, workers such as Dhowian and Edil (1980) and Candler and Chartres (1988) have reported what has been called tertiary compression in peat, for both field and laboratory studies. Figure 2.47 shows the response of two trial embankments on peaty soil, where the three mentioned phases are evident. den Haan (1994) defined tertiary compression as a decrease of slope $m$ in a plot of log strain rate versus log time, after a relative constant stretch at $m \approx 1$.

Mesri et al. (1997), after a study on Middleton peat, mentioned that the laboratory environment may enhance the anatomical and chemical degradations of plant cell walls by aerobic microbial activities and anaerobic fungi and bacteria; thus, due to the loss of structural integrity of cell walls and cell inclusions, increase compression during the secondary consolidation stage can be expected.

Fox et al. (1999) presented the results of two tests, also on Middleton peat; one of the samples (N1) was tested under normal conditions, while the other sample (N2) was subjected to gamma radiation prior to testing to minimize biodegradation during secondary compression. Figure 2.48 shows the results for these tests, where it can be seen that secondary compression for specimen N1 shows $C_\alpha$ increasing with time and then decreasing at the end of the test, while $C_\alpha$ for specimen N2 remains essentially consistent with time.

![Figure 2.48: e - log t curves of Middleton peat for untreated (N1) and irradiated (N2) samples (Fox et al., 1999).](image-url)
2.3.1 The end of primary approach

The most traditional method of calculating the magnitude of settlement is the End of Primary (EOP) approach. The EOP method considers that the secondary compression begins at the end of primary consolidation. This assumption simplifies the estimation of settlement in that the primary settlements can be estimated in the traditional fashion and the secondary compression can be included as a separate calculation. This approach would not include the additional settlement that would arise from the secondary compression that would occur during primary consolidation (Farrell, in press). Nonetheless, the standard oedometer test on peats would include some secondary compression which may reduce the error in practice (Hobbs, 1986).

The EOP method uses the normal $e - \log \sigma'$ (Figure 2.49) to estimate the settlement resulting from an increase in the vertical effective stress. It is generally more useful to obtain the settlement in terms of the strain ($\varepsilon$). For the primary consolidation, the strain is calculated as follows:

![Figure 2.49: Schematic $e - \log \sigma'$ curve (Terzaghi et al., 1996).](image)
If $\sigma'_{vo} + \Delta \sigma'_v \leq \sigma'_p$

$$\varepsilon = \frac{\Delta e}{1 + e_o} = \frac{C_R}{1 + e_o} \log \left( \frac{\sigma'_{vo} + \Delta \sigma'_v}{\sigma'_{vo}} \right)$$

or

$$\varepsilon = \frac{\Delta e}{1 + e_o} = \frac{C'_R \log \left( \frac{\sigma'_{vo} + \Delta \sigma'_v}{\sigma'_{vo}} \right)}{\log \left( \frac{\sigma'_{vo} + \Delta \sigma'_v}{\sigma'_{vo}} \right)}$$  \hspace{0.5cm} (2.26)

If $\sigma'_{vo} < \sigma'_p < \sigma'_{vo} + \Delta \sigma'_v$

$$\varepsilon = \frac{\Delta e}{1 + e_o} = \frac{C_R}{1 + e_o} \log \left( \frac{\sigma'_p}{\sigma'_{vo}} \right) + \frac{C'_C}{1 + e_o} \log \left( \frac{\sigma'_{vo} + \Delta \sigma'_v}{\sigma'_p} \right)$$

or

$$\varepsilon = \frac{\Delta e}{1 + e_o} = \frac{C'_R \log \left( \frac{\sigma'_p}{\sigma'_{vo}} \right)}{\log \left( \frac{\sigma'_{vo} + \Delta \sigma'_v}{\sigma'_p} \right)} + \frac{C'_S \log \left( \frac{\sigma'_{vo} + \Delta \sigma'_v}{\sigma'_p} \right)}{\log \left( \frac{\sigma'_{vo} + \Delta \sigma'_v}{\sigma'_p} \right)}$$  \hspace{0.5cm} (2.27)

where:

$\sigma'_{vo}$ is the initial vertical effective stress

$\Delta \sigma'_v$ is the vertical effective stress increment

$\sigma'_p$ is the preconsolidation pressure

$e_o$ is the initial void ratio

$\Delta e$ is the change in void ratio due to $\Delta \sigma'_v$

$C'_C$ is the compression index

$C'_S$ is the swell index

$C'_C$ is the compression ratio

$C'_R$ is the recompression ratio

$C'_C$ is the slope on the virgin section of the $e - \log \sigma'_v$ curve Figure 2.49.

$$C'_C = \frac{\Delta e}{\log \left( \frac{\sigma'_{vo} + \Delta \sigma'_v}{\sigma'_{vo}} \right)}$$  \hspace{0.5cm} (2.28)
Similarly, $C_R$ is the slope on the recompression section of the $e - \log \sigma_v$ curve. However, $C_R$ values are highly sensitive to soil disturbance and the oedometer procedure (Terzaghi et al., 1996). In practice, it is usual to use the swell index $C_S$, from the rebound section of the $e - \log \sigma_v$ curve, as a good approximation of $C_R$ (Figure 2.49).

$$C_R \approx C_S = \Delta e \log \left( \frac{\sigma_v' + \Delta \sigma_v'}{\sigma_v'} \right)$$

(2.29)

As can be seen from equation (2.27), the EOP approach requires the estimation of the preconsolidation pressure. Field and laboratory study have shown that virgin peat do have an 'apparent' yield point, possibly due to capillary suction forces when in the acrotelm or due to the weight of snow (Farrell, in press). The preconsolidation pressure may be obtained using the normal Casagrande construction (1936) or Janbu’s method (1969).

Considering that the void ratio and strain, due to secondary compression under a constant effective stress, generally varies linearly; the settlements can estimated from the $e - \log t$ curve (Figure 2.50), in the EOP method, as follows:

$$\varepsilon = \frac{\Delta e}{1 + e_o} = \frac{C_o}{1 + e_o} \log \left( \frac{t_2}{t_1} \right)$$

or

$$\varepsilon = \frac{\Delta e}{1 + e_o} = C_{SEC} \log \left( \frac{t_2}{t_1} \right)$$

(2.30)
where:

$C_a$ is the secondary compression index, obtained as the slope $e - \log t$ curve after $t_p$ (Figure 2.50)

$C_{SEC}$ is the coefficient of secondary compression, calculated similarly as the slope of the $e - \log t$ curve

$t_p$ is the time of the end primary consolidation

As explained by Hobbs (1986), the correlation between $C_a$ and $C_{SEC}$ is:

$$
\Delta e = C_a(\Delta \log t) \quad \text{and} \quad \Delta H/H_o = C_{SEC}(\Delta \log t)
$$

since,

$$
e = \Delta e/(1 + e_o) = \Delta H/H_o
$$

(2.31)

hence,

$$
C_{SEC} = C_a/(1 + e_o)
$$

2.3.2 The resistance concept

Janbu (1969) introduced the use of the resistance concept in soil mechanics. During the 25th Rankine Lecture, Janbu (1985) explained how the resistance is a unifying concept, being widely applied in all fields of engineering, where action - reaction systems require analysis.

According to this concept, all media possess resistance against a forced change of existing equilibrium conditions. The resistance of a medium, or of an isolated part of it, can therefore be determined by measuring the incremental response to a given incremental action (Figure 2.51); therefore, by definition:

$$
\text{Resistance} = \frac{\text{Incremental cause (given)}}{\text{Incremental effect (measured)}}
$$

(2.32)
For a non-linear response the resistance is in general defined as the tangent to the action-response curve. For a linear action-response curve the resistance is a constant of proportionality, without a change in definition. Some examples of the resistance concept applied in engineering are: electrical resistance ($R$ or $\rho$), elastic resistance ($E$), dynamic resistance ($mass$), hydraulic resistance ($k^l$) and heat resistance ($C$).

The biggest advantage of the resistance concept is the basic and simple approach to the soil stress-strain-time behaviour evaluation, which makes it a powerful tool for practical engineering (Havel, 2004). Janbu's resistance concept has been not only applied to clay but...
also to other granular media such as silts, sands, moraine, fractured rocks and sedimentary rocks (Figure 2.52).

2.3.2.1 Stress – strain behaviour

The resistance concept can be applied to the observed stress – strain behaviour of soils. The behaviour is completely described by the vertical effective stress \( \sigma' \) versus strain \( \varepsilon \) curve, which must be presented on an arithmetic scale as shown in Figure 2.52 (Janbu, 1985). In this case, the applied stress is the action and the strain is the response. The tangent modulus \( M \) to the curve is the resistance against deformation, thus:

\[
M = \frac{d\sigma'}{d\varepsilon}
\]  

(2.33)

where \( M \) is the compression modulus and the swelling modulus, for loading and unloading, respectively. Figure 2.52 presents the development of the tangent modulus \( M \) with stress \( \sigma' \) together with stress – strain curves from oedometer tests on different materials.

![Figure 2.53: Stress – strain resistance concept applied to Eberg clay (Janbu, 1985).](image)

The tangent modulus \( M \) usually varies with the stress. It has been found that, for stresses above the preconsolidation pressure \( \sigma'_{pc} \), \( M \) varies linearly, as is shown in Figure 2.53.
Hence for $\sigma' \geq \sigma'_p$:

$$M = m(\sigma' - \sigma'_r)$$  \hspace{1cm} (2.34)

where $m$ is the modulus number and $\sigma'_r$ is the intercept on the $\sigma'$ axis and is the reference stress.

Combining the definitions given in equations (2.33) and (2.34):

$$d\varepsilon = \frac{d\sigma'}{m(\sigma' - \sigma'_r)}$$  \hspace{1cm} (2.35)

Integrating for normally consolidated soils, from $\sigma'_p$ to $\sigma' = \sigma'_p + \Delta\sigma'_r$:

$$\varepsilon = \frac{1}{m} \ln \left( \frac{\sigma' - \sigma'_r}{\sigma'_p - \sigma'_r} \right)$$  \hspace{1cm} (2.36)

According to Janbu (1985) equation (2.36) implies that the simple explanation of the past experiences of linear $e - \log \sigma'$ curves is that the tangent modulus is linearly dependent on the effective stress $M = m\sigma'$, with $\sigma'_r = 0$.

For over-consolidated soils $\sigma' < \sigma'_p$, a stress change from $\sigma'_{vo} + \Delta\sigma'_r < \sigma'_p$ will lead to a strain $\varepsilon$ equal to (Janbu, 1969):

$$\varepsilon = \frac{\Delta\sigma'_r}{M}$$  \hspace{1cm} (2.37)

Lefebvre et al. (1984) investigated the tangent modulus of a Quebec fibrous peat (Figure 2.54). Due to the high compressibility of peat, this author decided to use the natural strains (as defined by Hencky) rather than the linear strains. The change affected the value of the tangent modulus, reducing it greatly. However, the general behaviour remained the same as the one described by Janbu (1969). Using this method, Lefebvre et al. (1984) found preconsolidation pressures ranging from 7.5 to 23 kPa, which contrasts with the values
found using the \( e - \log \sigma' \) curve, which ranged between 3.2 and 7.8 kPa. Furthermore, the values found for the modulus number, \( m \), ranged between 2.56 and 3.57.

![Figure 2.54: Vertical strain and tangent modulus versus effective stress for Quebec peat (Lefebvre et al., 1984).](image)

### 2.3.2.2 Strain – time behaviour

For step loading in oedometer tests, where the load is applied instantaneously and left constant for some time during which the sample gradually compresses, the resistance concept can be applied in a similar manner as the one described above. In this case, the time is considered as the action and the strain as the response. Hence, the time resistance of the soil \( R \) was defined by Janbu (1969) as:

\[
R = \frac{dt}{d\varepsilon} \quad (2.38)
\]
Figure 2.55 presents a schematic of the time–strain curve, and the time–resistance curve, plotted in arithmetic scale. The $R - t$ curve is divided into three zones, namely: (i) a primary consolidation phase where the initial excess PWP gradually reduces, and the curve is represented by a second degree parabola; (ii) an intermediate transition zone where both, hydrodynamic primary consolidation and secondary creep compression settlement are taking place; and (iii) a zone where the excess pore pressure is zero and only secondary compression is occurring.

![Diagram](image)

**Figure 2.55:** Definition of time resistance $R$ and creep resistance number $r_c$, for the incremental loading oedometer (Janbu, 1985).

However, for $t \geq t_c$, it is likely that the gradients and hence the seepage forces are so small that the time–delayed compression, for all practical purposes, is governed by the intergranular shear stress and shear strains only (Havel, 2004). Irrespective of this, the $R - t$ curve will approach a straight line and thus can be written that:

$$R = r_s(t - t_r)$$  \hspace{1cm} (2.39)

where $r_s$ is the creep resistance number.
Introducing equation (2.39) into equation (2.38), and integration between \(t_c\) and \(t\):

\[
e_s = \frac{1}{r_s} \ln \left( \frac{t - t_r}{t_c - t_r} \right) \tag{2.40}
\]

where \(e_s\) is the creep strain or secondary compression.

Different authors have studied, and shown, that \(r_s\) depends on the level of the effective stress applied for clay soils (Figure 2.56). According to Havel (2004), the creep resistance number is very large in the over-consolidated range \((\sigma' < \sigma_{p}')\) and drops radically when the effective stress \(\sigma'\) approaches the preconsolidation pressure \(\sigma_{p}'\). The minimum value of the creep resistance \(r_s\) is at \(\sigma' = \sigma_{p}'\) and thereafter increases slightly with increasing \(\sigma'\). It can be seen from Figure 2.56, that the preconsolidation pressure can be clearly identified for both clays presented.

---

**Figure 2.56:** Creep resistance number variation versus effective stress for: (a) Barnehave clay (Janbu, 1969); and (b) Norwegian Onsey clay (Havel, 2004).
2.3.3 Numerical modelling: Soft Soil and Soft Soil Creep models

The Soft Soil (SS) and Soft Soil Creep models (SSC) were developed and implemented by PLAXIS bv to model the behaviour of near–normally consolidated clays, clayey silts and peats; with high compressibility as the special feature of these materials (Brinkgreve et al., 2010). These two constitutive models are briefly introduced here. A detailed description can be found in Brinkgreve et al. (2010), The et al. (1998), Vermeer and Neher (1999), Blommaart et al. (2000), Neher et al. (2001) and Waterman and Broere (2004).

Hebib and Farrell (2003) used the SS model to back analyse the behaviour of Ballydermot peat on an oedometer test, and reported that the model realistically represented the recorded behaviour in the laboratory. Tan (2008) employed the SSC to continuously model a roadway construction across cranberry bog areas with deep peat deposits, in U.S. Route 44, Massachusetts; and found that the modelling results matched well with the field measurements at different construction stages.

2.3.3.1 Soft Soil model (SS)

The SS model is based on the modified Cam Clay model (Neher et al., 2001). In the model, it is assumed that there is a logarithmic volumetric strain, \( \varepsilon_v \), and the mean effective stress, \( \sigma' \), formulated as (Figure 2.57):

\[
\varepsilon_v - \varepsilon_{v0} = \lambda^* \ln \left( \frac{\sigma'}{\sigma_0^*} \right) \quad (2.41)
\]

For unloading and reloading situations, the volume strain is supposed to be elastic (described by Hooke’s law and denoted by the superscript \( e \)), and is formulated as:

\[
\varepsilon_v^e - \varepsilon_{v0}^e = \kappa^* \ln \left( \frac{\sigma'}{\sigma_0^*} \right) \quad (2.42)
\]

where:

- \( \lambda^* \) is the modified compression index
- \( \kappa^* \) is the modified swelling index
The modified compression index ($\lambda^*$) and the modified swelling index ($\kappa^*$) can be obtained from an isotropic compression test or an one-dimensional compression test, when plotting the natural logarithm of the mean stress as a function of the volumetric strain, as shown in Figure 2.57 (Brinkgreve et al., 2010). It is important to note that, $\lambda^*$ and $\kappa^*$ are different from the Cam Clay compression index ($\lambda$) and swelling index ($\kappa$), as the latter parameters are defined in terms of the void ratio, $e$, instead of the volumetric strain, $\varepsilon_v$.

Equations (2.43) and (2.44) give relationships between the SS model parameters and the compression and swell indices ($C_C$ and $C_S$) described in §2.3.1.

\[
\lambda^* = \frac{C_C}{(1 + e_0)\ln 10}\quad (2.43)
\]

\[
\kappa^* \approx \frac{2C_S}{(1 + e_0)\ln 10}\quad (2.44)
\]

According to Waterman and Broere (2004), the ratio of the unloading/primary loading stiffness, $\lambda^*/\kappa^*$, cannot be smaller than 1 and should normally be between 2 and 10. For most practical cases the value of the $\lambda^*/\kappa^*$ ratio falls within the range of 3 to 7.
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2.3.3.2 Soft Soil Creep model (SSC)

All soils exhibit certain amount of creep, and primary compression is thus always followed by a certain amount of secondary compression (Vermeer and Neher, 1999). Secondary compression or creep plays an important role in the deformation behaviour of soft soils like peat and organic clay (Blommaart et al., 2000).

The SSC model is based on the modified Cam-clay soil model, extended with a viscoplastic component and combined with Biot's consolidation theory, to account for consolidation and time dependent deformation behaviour (Blommaart et al., 2000). The creep component is based on the work of Butterfield (1979) where the secondary compression is expressed as:

\[ \varepsilon_v = \varepsilon_{vc} + \mu^* \ln \left( \frac{\tau_c - t'}{\tau_c} \right) \]  

(2.45)

where:
- \( \varepsilon_v \): Total volumetric strain
- \( \varepsilon_{vc} \): Volumetric strain during consolidation
- \( \mu^* \): The modified creep index

The strain rate, \( 1/\dot{\varepsilon} \), can be determined by differentiating equation (2.45) with respect to time:

\[ \dot{\varepsilon} = \frac{\mu^*}{\tau_c - t'} \quad \text{or inversely} \quad \frac{1}{\dot{\varepsilon}} = \frac{\tau_c - t'}{\mu^*} \]  

(2.46)

Equation (2.46) allows the Janbu's construction to be developed (see §2.3.2.2) in order to determine modified creep index (\( \mu^* \)) from experimental data. However, this parameter can be determine by plotting the volumetric strain versus the natural logarithm of time (Figure 2.58(a)), as well as by using Janbu’s construction as described (Figure 2.58(b)). The use of Janbu’s method is recommended by PLAXIS bv as both \( \mu^* \) and \( \tau_c \) can be directly determined when fitting a straight line through the data (Brinkgreve et al., 2010).
Figure 2.58: Idealised consolidation and creep behaviour in standard oedometer test (Neher et al., 2001).

Up to this point, $\tau_c$ can only be determined from experimental data. In order to find an analytical expression to determine $\tau_c$, it is assumed that all inelastic strains are time dependent. Additionally, Bjerrum (1967) assumed that the preconsolidation stress depends on the amount of creep strain accumulated by time, and expressed the strain up to the end of consolidation, $\varepsilon_c$, by an equation of the form:

$$\varepsilon_{vc} = \varepsilon^e_{v} + \varepsilon^{cr}_{v} = \kappa^*\ln\left(\frac{\sigma^{'f}}{\sigma^{'o}}\right) + (\lambda^* - \kappa^*)\ln\left(\frac{\sigma^{'f}}{\sigma^{p_{pc}}_{o}}\right)$$

where:

$\varepsilon^e_{v}$: Elastic volumetric strain during consolidation
$\varepsilon^{cr}_{v}$: Time dependant volumetric strain during consolidation
$\sigma^{'o}$: Initial effective pressure before loading
$\sigma^{'f}$: Final effective loading pressure
$\sigma^{p_{pc}}_{o}$: Preconsolidation pressure before loading
$\sigma^{p_{pc}}_{f}$: Preconsolidation pressure after creep.

The preconsolidation pressure after creep is equal to:

$$\sigma^{'f} = \sigma^{p_{pc}}_{o} \cdot e^{\left(\frac{\varepsilon^{cr}}{\lambda^* - \kappa^*}\right)}$$

From equation (2.48) it can be seen that the longer the soil is left to creep the larger $\sigma^{'f}$ grows (Vermeer and Neher, 1999).
Combining equations (2.45) and (2.47), it follows that the total volumetric strains due to an increase in the effective stress from $\sigma'_0$ to $\sigma'$ and a time period $t_c + t'$ is:

$$\varepsilon_v = \varepsilon_{vc}^e + \varepsilon_{vc}^{cr} + \varepsilon_{vac}^{cr}$$

$$\varepsilon_v = \kappa^* \ln \left( \frac{\sigma'}{\sigma_0'} \right) + (\lambda^* - \kappa^*) \ln \left( \frac{\sigma_{pc}'}{\sigma_{po}'} \right) + \mu^* \ln \left( \frac{t_c - t'}{t_c} \right)$$

(2.49)

As can be seen from equation (2.49) the total volumetric strain is divided into an elastic part and a visco-pastic creep part, denoted by the superscripts $e$ and $cr$ respectively. The visco-plastic part is further separated into a part occurring during consolidation and a part after consolidation, denoted by the subscripts $c$ and $ac$. The terms from equation (2.49) are depicted in Figure 2.59.

![Figure 2.59: Idealised stress-strain curve from oedometer test with the strain increment divided into an elastic and a creep component (Neher et al., 2001).](image)

The time dependency of the preconsolidation pressure $\sigma'_p$ is now found by combining equations (2.47) and (2.49) to obtain (Neher et al., 2001):

$$(\lambda^* - \kappa^*) \ln \left( \frac{\sigma_p'}{\sigma_{pc}'} \right) = \mu^* \ln \left( \frac{t_c - t'}{t_c} \right)$$

(2.50)
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Assuming that the is increased stepwise and each load step is maintained for a constant period \( t_c + t' = \tau \), where \( \tau \) is precisely 24 h, the IC-line (Figure 2.59) with \( \sigma'_{p} = \sigma' \) is obtained. Equation (2.50) leads for \( OCR = \sigma'_{p} / \sigma' = 1 \) to:

\[
(\lambda^* - \kappa^*) \ln \left( \frac{\sigma'_{p}}{\sigma'_{pc}} \right) = \mu^* \ln \left( \frac{\tau_c + \tau - \tau_c}{\tau_c} \right) \tag{2.51}
\]

Considering that with respect to \( \tau \) the difference \( \tau - \tau_c \) is very small (Neher et al., 2001), equation (2.51) can be simplified as:

\[
\frac{\tau}{\tau_c} = \left( \frac{\sigma'_{p}}{\sigma'_{pc}} \right)^{\frac{\lambda^* - \kappa^*}{\mu^*}} \quad \text{or} \quad \tau = \tau_c \left( \frac{\sigma'_{p}}{\sigma'_{pc}} \right)^{\frac{\lambda^* - \kappa^*}{\mu^*}} \tag{2.52}
\]

Having derived an expression for \( \tau_c \) it is now possible to formulate the differential creep equation:

\[
\dot{\varepsilon}_v = \kappa^* \frac{\sigma'}{\sigma'} + \frac{\mu^*}{\tau_c + t'} \tag{2.53}
\]

Where \( \tau_c + t' \) can be eliminated by means of equation (2.50):

\[
\dot{\varepsilon}_v = \kappa^* \frac{\sigma'}{\sigma'} + \frac{\mu^*}{\tau_c} \left( \frac{\sigma'_{pc}}{\sigma'_{p}} \right)^{\frac{\lambda^* - \kappa^*}{\mu^*}} \tag{2.54}
\]

Finally, introducing equation (2.52) into equation (2.54):

\[
\dot{\varepsilon}_v = \kappa^* \frac{\sigma'}{\sigma'} + \frac{\mu^*}{\tau} \left( \frac{\sigma'_{pc}}{\sigma'_{p}} \right)^{\frac{\lambda^* - \kappa^*}{\mu^*}} \tag{2.55}
\]

Where \( \sigma'_{p} \) is defined as in equation (2.48).
The modified compression index ($\lambda^*$) and the modified swelling index ($\kappa^*$) are the same as described in §2.3.3.1. Equation (2.56) gives the relationship between the modified creep index and the secondary compression index described in §2.3.1.

$$\mu^* = \frac{C_0}{(1 + e_0)\ln 10} \quad (2.56)$$

In order to determine $\mu^*$ using the oedometer test, the load step needs to be left for a period of time long enough for the consolidation effect to disappear and the curve to become straight. If the oedometer test is stopped, or another load step is applied, before the consolidation is finished, the consolidation rate will then be added to $\mu^*$ resulting in an overestimation of the parameter (Waterman and Broere, 2004).

According to Waterman and Broere (2004), there is a creep ratio expressed as $(\lambda^* - \kappa^*)/\mu^*$ which needs to be considered. This ratio has a wide range of values, normally between 5 and 25, where high values represent stiff soils with little creep and small values represent soft soils with a considerable amount of creep. For most practical cases, the ratio falls within the range of 10 to 20, and if the creep ratio is over 25 the use of the creep model could be reconsidered. For these relatively stiff soils creep is of minor importance and the Soft Soil model could be used instead.

2.3.4 Permeability characteristics of peat

A striking characteristic of peat is the remarkable decline in permeability with reduction in void ratio or water content, falling by some three orders against a change in void ratio of half an order (Hobbs, 1986).

Tavenas et al. (1983a) conducted an investigation on the different equipment and methods used to measure the permeability of clays, and concluded that the triaxial apparatus renders the most reliable measurements; it was also concluded that the indirect methods of evaluating $k$ from oedometer tests produce significant errors due to the assumptions of Terzaghi’s consolidation theory. However, Mesri and Ajlouni (2007) showed that the permeability of fibrous peat can be directly measured by falling or constant head flow
measurements during an oedometer test, or can be also computed using porewater pressure measurements during the constant rate of strain oedometer test or compression measurements during the incremental loading oedometer test (Figure 2.60).

Hobbs (1986) compiled a total of 98 series of permeability tests carried out by several workers worldwide, in undisturbed mainly fibrous peats, during pauses in consolidation in the standard oedometer and Rowe cell (Figure 2.61). Hobbs (1986) was able to construct an envelope covering most of the results, and concluded that the permeability was not directly related to the pressure causing compression, but to the initial void ratio, reflecting the natural state of the peat. Tavenas et al. (1983b) also observed a significant relation between void ratio and permeability, and study different relationships proposed by several authors, concluding that none of these relations were generally valid. However, the authors suggested that the use of equation (2.57) is valid from a practical point of view.

\[
\log k = \log k_o - \frac{e_o - e}{C_k}
\]  

(2.57)

Where \( C_k \) is the permeability change index, and \( k_o \) is the initial coefficient of permeability at the initial void ratio, \( e_o \). According to Tavenas et al. (1983b) and Mesri et al. (1997), for soft clay and silt deposits the values of \( C_k = 0.5e_o \).
Figure 2.61: Vertical permeability during pauses in consolidation tests on undisturbed peat (Hobbs, 1986).

Lefebvre et al. (1984) and Mesri et al. (1997), reported that, for peat, equation (2.57) also described the permeability decrease with the reduction in void ratio. Lefebvre et al. (1984) reported $C_k$ values between 1.5 and 3.2 for the Quebec fibrous peat. Mesri et al. (1997) showed that for peat $C_k = 0.25e_0$. Furthermore, Mesri and Ajlouni (2007) compared the $C_k$ versus the initial void ratio for five different materials, including peat (Figure 2.62).

Figure 2.62: $C_k$ values versus $e_0$ for five different materials (Mesri and Ajlouni, 2007).

Another feature of peat permeability is the high degree of anisotropy, with horizontal permeability being usually greater than vertical permeability, particularly in fibrous peat.
According to Hobbs (1986), the fresher the peat the greater the ratio of horizontal to vertical permeability and highly humified peat may be expected to be fairly uniform.

Lefebvre et al. (1984) found that for the fibrous Quebec peat, the horizontal permeability was initially slightly lower than the vertical permeability, but at larger stress levels it becomes larger by a factor of 8. Hobbs (1986) mentioned that, in private communication, Petley (1983) reported that for Somerset Levels peats the horizontal to vertical permeability ratio was in the range of 1.7 to 7.5. For the Middleton peat, Mesri et al. (1997) found that $k_h/k_v = 10$. According to Mesri and Ajlouni (2007), for surficial fibrous peats $k_h/k_v$ is likely to be in the range of 3 to 5.

Nonetheless, Hobbs (1986) cautioned that laboratory tests may be regarded as carried on a microstructural scale, and that both the permeability and anisotropy increase on the field as the larger features become predominant.

2.4 SUMMARY

2.4.1 Vacuum consolidation

Vacuum consolidation has become a widely used ground improvement technique for reducing the post-construction settlement and increasing the shear strength of highly compressible soft soils. Field applications have been reported worldwide since the 1980s, predominantly in Asia and Europe. Although vacuum consolidation was proposed in the early 1950s, the principles and mechanisms of vacuum consolidation have only been studied and explained in the last two decades, particularly in the last ten years.

Significant differences exist between vacuum consolidation and the traditional surcharge approach. Vacuum preloading induces a reduction in the pore water pressure, increasing the effective stress, while the total stress remains constant. The effective stress increase tends to be isotropical, which will induce inwards lateral deformation. Consequently, the risk of shear failure can be minimised even if vacuum preloading is combined with surcharge fill. Furthermore, higher rate of embankment construction has been achieved when using vacuum consolidation. From a practical point of view, the surcharge fill can be
reduced in height by using vacuum preloading, to achieve the same desired rate of consolidation.

Two types of vacuum consolidation techniques are presently in use, membrane and membraneless system. The effectiveness and economy of the former depends on (i) the integrity (airtightness) of the membrane, (ii) the seal between the membrane edges and the ground, (iii) the soil stratification including permeable layer within the soil deposit, and (iv) the depth of ground water level. For the later the main factors are: (i) thickness of the sealing layer, (ii) required length of tubing, (iii) soil stratification including permeable layer within the soil deposit, and (iv) permeability of the soil to be improved.

Vacuum consolidation has been successfully used on large scale project, on very soft grounds, particularly soft clayey soils and hydraulic fills. However, other materials as peat and soda-ash tailings have also been improved, although there are few publications on its use on these materials.

Different pumping systems have been developed for practical applications, such as vacuum stations with vacuum pumps acting solely on the gas phase in conjunction with pumps allowing liquid and gas suction; and systems using jet pumps, capable of extracting the air-water mixture. The usual vacuum pressures reported for the different projects ranges between 70 – 80 kPa, with maximum values around 90 kPa. For projects conducted on peat, the usual values are around 60 kPa.

Several different apparatuses for testing vacuum consolidation in the laboratory have been developed in the last 15 years, including consolidation cells, triaxial cells and large-scale radial drainage consolidometers. These new equipment have allowed to further understand the behaviour of soils under vacuum loading.

Some laboratory investigations have concluded that the soil compressibility characteristics obtained in conventional consolidation tests can be used in the design of vacuum consolidation projects. Tests on large-scale consolidometer and field observations have shown the actual the distribution of vacuum along the length of the drain. The lateral deformation of soils under vacuum pressures has also been studied.
Chapter 2  LITERATURE REVIEW

Increasing attention has been paid to analytical and numerical modelling vacuum consolidation since the mid-2000s. Analytical consolidation models have been developed, including: a one-dimensional model, 2D axisymmetric and plane strain models of a single cell unit, and 3D models. These two and three dimensional models have been usually incorporated in numerical finite element analysis.

2.4.2 One-dimensional compressibility of peat

Peat is an organic, highly compressible material. The compressibility of peat is due to three factors: (i) primary consolidation associated with the dissipation of excess pore pressures, (ii) secondary compression due to creep, and (iii) tertiary compression related to the decay and loss of integrity of the vegetable matter. Due to the rapid dissipation of the excess pore pressure in peat, secondary compression is the major contributor of the observed long term settlements.

Different theories have been proposed and applied to model the behaviour of compressible soils and peats. Traditional, it is thought that primary consolidation is followed by secondary compression; however, some workers consider that these two phenomena occur simultaneously.

The permeability of peat, as that of mineral soils, is directly related to the initial void ratio. As the void ratio is reduced due to compression, the permeability reduces drastically. It has been shown that for peat, $k$ values can be measured using triaxial apparatus, falling head test or constant head tests; or can be computed using CRS or oedometer tests.

The permeability of peat is highly anisotropic, particularly for the less humified materials, with horizontal permeability being larger than vertical permeability. Furthermore, as peat is compressed due to loading, the horizontal permeability tends to become larger.

Finally, the permeability and anisotropy of peat in the field increase as the larger features become predominant.
3. VACUUM CONSOLIDATION TEST SITE

3.1 INTRODUCTION

A vacuum consolidation field test was carried out on a section of Ballydermot bog, County Offaly, in order to study the efficiency of vacuum consolidation and the response of peat. The test site is located approximately 65 km from the centre of Dublin (Figure 3.1).

This chapter describes the test site, the findings of two cable percussion boreholes, two manual boreholes and three vane tests; and the results of the laboratory tests carried out on the samples recovered. Rainfall records and water table variations are also presented. The laboratory tests included classification tests and oedometer tests. Finally, the results of the oedometer tests are analysed using four different models.

3.2 SITE DESCRIPTION

3.2.1 Site location

Ballydermot bog is situated in the border between Counties Kildare and Offaly, approximately 2.7 km north of Rathangan and 12 km south of Edenderry, in the Irish Midlands (Figure 3.2). The bog is harvested for milled peat which is used in the Edenderry Power Plant is located approximately 4.5 km to the west of the bog.

The Trinity College Dublin / National Roads Authority (TCD/NRA) vacuum consolidation field test is located at the south end of Ballydermot South bog by the Bord na Móna workshops and site office.
3.2.2 Site description

Ballydermot bog is a raised bog and part of the 2490 hectares Ballydermot-Lullymore Bog Complex comprising the bog units of Ballydermot North and South, Barnaran, Blackriver, Glashabaun North, Glashabaun South and Lodge, Codd North and South, and Lullymore (Hammond, 1969). The bog covers an area of 900 ha and is divided in Ballydermot North with 600 ha and Ballydermot South with 300 ha (Whitaker, 2004).

Figure 3.1: Location of site relative to Dublin (NASA - Earth Observatory, 2011).

Extensive peat extraction has taken place in Ballydermot for the last 60 years. In the mid-1940s, Ballydermot bog was developed and drainage commenced for the production of sod peat. In the mid-1990s the bog was changed from sod peat to milled peat production. According to Bord na Móna staff on site, the last year of sod peat production was 1995.
The bog was then developed for milled peat production between 1996 and 1997, which commenced in 1998 and is on-going. Presently, the bog is part of the Derrygreenagh milled peat group.

Figure 3.2: Ballydermot-Lullymore Bog Complex (Whitaker, 2004).

Figure 3.3 shows a panoramic view of the test area prior to construction works commencement for the pilot test. A satellite view of the area is presented in Figure 3.4. The drainage channels built by Bord na Móna to drain the bog for peat harvesting purposes are evident in this image. The test area is located between the series channels and an approximately 2.25 m deep ditch as shown in Figure 3.5. A detail view of the channels and the ditch is shown in Figure 3.6 and Figure 3.7, respectively.
Figure 3.3: Panoramic view of Ballydermot bog prior to TCD/NRA vacuum consolidation test.
Figure 3.4: Satellite view of TCD/NRA field vacuum consolidation test area (Google Imagery, 2011).
Figure 3.5: Placement of test area between drainage channels and ditch.
3.2.3 Site geology

Peat developed in Ireland in the post-glacial period within the past 10000 years. Three basic peat formations are recognised: (i) raised bogs of the Central Plain, (ii) blanket bogs of the Western seaboard and the upland regions, and (iii) fen peats (Hammond, 1981). Geographically, Ballydermot bog is a raised bog located in the central Irish lowland region, underlain by Carboniferous limestone.

Because of the extensive drainage and intense harvesting, the peat layer at Ballydermot bog has undergone a combined process of reduction in thickness due to harvesting and continuous settlement due to drainage, over the last 60 to 70 year. These combined processes have induced preconsolidation of the peat deposit, expressed in an increased...
preconsolidation pressure of the Ballydermot peat when compared with values for other bogs in the Irish midlands, as will be shown later in §3.4.3.1. Cuddy (1988) reported that in 1935 Sir Harry Godwin excavated a trial hole in Ballydermot bog, and found a peat depth of 8.7 m. Hanrahan (1953; 1954; 1964; 1967; 1976; 1981) conducted a 28 years investigation on the main road from Edenderry to Rathangan, which crosses Ballydermot bog and can be seen in Figure 3.4. The field investigation was carried in August 1953, and found that the average depth of the peat was approximately 7.62 m. This road is located approximately 200 m to the east of the test area, outside Bord na Móna grounds. Hammond (1969), after a peat stratigraphy study in Ballydermot bog, estimated that the original undrained average peat depth was 7.0 m. By 1988, Cuddy found an average depth of 5.64 m below the road surface. Hebib (2001) reported that due to the extensive drainage and peat production by Bord na Móna, only 4.0 m of peat remained. This was confirmed by the site investigation conducted for this project. According to Hammond (1969) and Hebib (2001) the peat layer is underlain by shelly marl and boulder clay.

3.2.4 Approximated soil profile

Figure 3.8 presents an approximated soil profile, based on the field investigation and soils classification described in §3.3 and §3.4, respectively. In the soil profile, all levels are related to Site Datum (S.D.) as defined in §3.3. The horizontal distance between the two boreholes is calculated by drawing an axis that crosses the two of them in Figure 3.9, and assuming origin in BH-1.

As it can be seen, the profile is mainly composed of three layers (i) a 0.8 m thick man-made fill, (ii) a 3.2 m pseudo-fibrous peat layer, and (iii) a 1.9 m gravel layer with very high contents of fines and sand, reducing with depth.

3.3 FIELD INVESTIGATION

The field investigation at Ballydermot bog comprised by two manual boreholes using spiral and gouge augers, two cable percussion boreholes and three field shear vane tests. A rain gauge and 24 stand pipes were installed to measure the variations of the ground water table and relate it to the rainfall prior to and during testing. The field investigation was
carried in accordance with BS 5930: 1999 (BSI, 2007). It is important to note that on 18\textsuperscript{th}
June 2009, after the exact location of the test area was selected, the surface was excavated
and 10 stand pipes were lost.

**Figure 3.8**: Approximated soil profile at Ballydermot bog.

**Figure 3.9**: Test area, stand pipes, boreholes and shear vane tests location with respect to Site Datum.
Chapter 3 DESCRIPTION OF TEST AREA

Figure 3.9 presents the location in plan-view of the test area, the remaining stand pipes after excavation, the two cable percussion boreholes and the three field shear vane tests, with respect to the selected Site Datum (S.D.). The S.D. was set outside the bog, on a concrete pavement in Bord na Móna grounds, and its coordinates are (0, 0, 100). A detailed explanation of the surface instrumentation system will be given in Chapter 4.

3.3.1 Manual boreholes

On 13th June 2009 two boreholes were carried out using a 1 inch spiral auger and a 1.5 inch gouge sampler (Figure 3.10) in the test area. At the time of boring the S.D. had not been set for the test yet; however, the 24 stand pipes were in place (see §3.3.5 and Figure 3.16(a)) and the location of the boreholes is related to these. MBH-1 was located approximately 2.0 m northwards from row 1 between pipes B1 and C1, while MBH-2 was done between rows 1 and 2, and lines C and D. Disturbed samples were recovered, bagged and returned to the Soils Mechanics Laboratory at Trinity College Dublin for testing (see §3.4).

Figure 3.10: Spiral and gouge auger for manual boreholes.

In MBH-1 two soil layers were found: (i) a 0.6 m man-made fill mainly composed of black peat with occasional plastic bags, geotextile pieces and stones, and (ii) a 3.54 m layer of fibrous peat. The peat became harder to sample and woody towards the bottom 0.6 m and
little stones were found in the last 0.15 m. The borehole was stopped at 4.14 m when the auger was pushed in and hit a hard surface producing a sound as it hit stones.

MBH-2 encountered: (i) Man-made fill 0.8 m thick with the same characteristics as above, and (ii) a 3.0 m layer of fibrous peat. As the auger was pulled out of the borehole at 3.8 m, a distinctive sound of water filling the hole was heard. No further advancement was possible from this depth.

3.3.2 Cable percussion boreholes

Two cable percussion boreholes were conducted, by Ground Investigation Ireland Ltd. and supervised by the author, at Ballydermot bog on the 10th and 11th of March 2010, as part of the detailed site investigation. Borehole 1 (BH-1) was done on the east side of the test area, at approximately 11.5 m from the southeast corner; while borehole 2 (BH-2) was located at roughly 7.8 m from the southwest corner (Figure 3.9).

Due to the soft nature of the peat surface, the drilling rig had to be carried into position by a peat digger and placed on a wood platform to provide a stable platform for the boring (Figure 3.11).
Chapter 3

DESCRIPTION OF TEST AREA

Undisturbed piston samples (100 mm diameter and 1.0 m long with sharpened edges) and bag samples were recovered from both boreholes and returned to the Soils Mechanics Laboratory at Trinity College Dublin for testing (see §3.4). The borehole log sheets, prepared by Ground Investigation Ireland Ltd, are included in Appendix A. It is to note that the approximated soil profile presented in §3.2.4 is based on the laboratory investigation conducted in the samples recovered, and not in the borehole log sheets.

BH-1 reached a depth of 6.3 m and four soil layers were found: (i) Peat fill 0.8 m thick, (ii) a 3.2 m layer of peat, (iii) a 2.0 m strata of stiff grey-brown sandy gravelly clay, and (iv) broken fragments of rock were extracted, indicating a boulder or possibly bedrock. Four piston samples were recovered at the peat layer and bag samples at all strata but the fill. Water was observed at 4.9 m and by the end of the day was at 5.6 m, both on the 10th of March.

BH-2 reached 7.0 m and five layers were described: (i) Peat fill 0.9 m thick, (ii) a 3.1 m peat layer, (iii) stiff grey sandy gravelly clay 1.0 m thick, (iv) 0.8 m of dense clayey gravel, and (v) broken fragments of rock. Three pistons were recovered at the peat layer and bag samples at all strata but the fill. Water was found at 5.5 m on the 10th March and by the end of the day it was located at 4.0 m, and on the 11th it was measured at 5.9 m.

Standard Penetration Test was conducted on both boreholes in the clay and gravel layers.

3.3.3 Field shear vane tests

Three field vane tests were performed on the 24th of February 2010 by the technical staff of the Soil Mechanics Laboratory, and its location is shown in Figure 3.9. The equipment utilized was a Geonor H-10 Penetration Field Shear Vane with a vane size of 55 mm x 110 mm (Figure 3.12).

Figure 3.13 shows the shear strength distribution with depth, related to S.D. The surface level for the three vane tests is also presented. The horizontal distance between the three vane tests is calculated by drawing an axis that crosses the three tests in Figure 3.9, and assuming origin in Vane 1.
As it can be seen from Figure 3.13, the shear strength with depth is very variable between the three tests. However, for vanes 1 and 2, at approximately level 105.2 m (roughly 3.5 m depth) the shear strength of the peat increases and advancement became harder. In vane test 3, there is a decrease in the shear strength at level 105.4 m, but at this point, when further advancement was attempted the soil became very hard and the test was stopped.
3.3.4 Rainfall monitoring

An ARG100/DT2-R tipping bucket rain gauge (Environmental Measurements Limited, Undated) was installed on site on 14th August 2009, and rainfall was recorded hourly until 12th April 2011. The rain gauge was placed approximately 200 m to the east of the test area, on an open area following the manual instructions as shown in Figure 3.14.

![Rain gauge positioning: (a) north view and (b) south view](image)

Figure 3.14: Rain gauge positioning: (a) north view and (b) south view

![Daily rainfall measurement before vacuum pumping](image)

Figure 3.15: Daily rainfall measurement before vacuum pumping.
Figure 3.15 shows the daily rainfall measured at Ballydermot bog. The data marked as Ballydermot (red) was recorded directly by the rain gauge installed on site. The data marked as Edenderry (blue), which is the closest weather station from the site, was provided by Met Éireann. As can be seen from the period where there is data from both sources (i.e. August – September 2009), there is good agreement between readings, and the weather station at Edenderry can be consider as representative for Ballydermot bog. The rainfall records from Mullingar and Oak Park stations were also provided by Met Éireann, and the full set is shown in Appendix A.

3.3.5 Water table monitoring

Due to the importance of the water table level to maintain an adequate seal in the treated area during vacuum consolidation (§2.2.3.1), 24 stand pipes where installed, on the 4th March 2009, in the testing area prior to the construction works of the TCD/NRA pilot test, to monitor the water table variation and select the depth of the peripheral trench.

The stand pipes were placed in a rectangular arrangement with 4.0 m spacing, in six lines (A, B, C, D, E and F) and four rows (1, 2, 3 and 4) as shown in Figure 3.16. The stand pipes were made in-house, at the Soils Mechanic Laboratory in Trinity College, from 1.5 m long and 36 mm diameter PVC pipes. Slots were cut along the lower 0.4 m length to create an effective length for the water to penetrate (Figure 3.17(a)). The effective length was then covered with a strong lycra “tight”, which will allow the water infiltration but prevent the peat from clogging the pipe. The “tight” was secured using duct tape. For the installation, an Edelman auger was used to open an initial hole in the ground and then the stand pipe was pushed down to position, leaving 0.2 m above the surface (Figure 3.17(b)).

The stand pipes were regularly monitored using a dip-meter (Figure 3.17(c)). At first no site datum reference was established, hence the initial water table depth measurements were used as rough guide. As will be explained in Chapter 4, the site datum was set up on late June 2009, and from that moment onwards water table readings are tied to this point.

Figure 3.18(a) shows the water table readings for all stand pipes up to 4th November 2009, before pumping started. In this figure, the readings are not related to the site datum. The
related measurements are presented in Chapter 4. It is important to note that in June 2009, after the exact location of the 10 x 10 m test area was selected, the surface was excavated and 10 pipes were lost (A1, A2, A3, B1, B2, C1, C2, D1, D2, E1). The location of the 12 m x 12 m excavated area in relation to the stand pipes is shown in Figure 3.16(a).

From Figure 3.18(a) it can be seen that there is a seasonal variation of the water table in the period before the field test started. The level dropped in March and reached its lower point during the summer time, around the mid-June. This can be related with the rainfall records in Figure 3.15, which show some dry periods and lower intensity in rainfall during the spring and summer, until early-June. From this point onwards, the stand pipes show an

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**Figure 3.16:** Stand pipe initial arrangement: (a) schematic and (b) site view.
increase in the water table levels, coinciding with an increase in the frequency of the rainfall. The highest levels are reached around mid-July, early-September and late-October, coinciding with rainy periods with some high intensities; and the drops in levels at early-August and early-October are concurrent with a short dry period in early-August and a virtually dry September.

Also in Figure 3.18(a), it is shown that the lower water table levels are obtained in the stand pipes located in row four towards the ditch (Figure 3.16(a)), indicating a dip in the water table around this area. This effect can be better seen in Figure 3.18(b), in which the water table profile along line F is presented for different times before pumping. An imaginary origin is taken at pipe F1, and the distance increases towards the ditch, located at 13.0 m. It is shown that for most dates, the profile dips in pipe F4 near the ditch. However, on October 21st to 23rd, the water table level is at its highest in pipe F4. This could be due to a combined effect, of intense rain during those dates and the construction of a granular bed on the test area using a peat digger around the pipes, as will be explained in Chapter 4.
Figure 3.18: Water table depth before vacuum pumping: (a) all stand pipes vs time and (b) column F profile.

3.3.6 Piezometer installation and hydraulic conductivity of clay

Four piezometers were installed at the gravel layer of the two boreholes (Figure 3.19), two in each, at different depths. In BH-1, the first one was placed at 6.0 m in the clay, while the
second one was located at 4.2 m in the peat-clay interface. In BH-2, the first one was placed at 7.0 m in the gravel, and the second one at 4.5 m close to the peat-clay interface. The piezometers installed were 24 mm in diameter and with an effective length of 240 mm; and the standpipes used were 17.3 mm diameter. A cross-section detail of the installation is shown in Figure 3.20.

Figure 3.19: Piezometer installed at Ballydermot bog.

Figure 3.20: Cross-section detail of piezometer installations.
Due to the heavy traffic around the test area of peat harvesting machinery from Bord na Móna, the piezometers from BH-2 were damaged when the author arrived to site on 19th April 2010. The piezometer located at 7.0 m was recovered and kept working through the project; however, the one located at 4.5 m was lost and only one month of readings were taken.

Falling head tests were conducted on the three remaining piezometers on the 25th May 2010, in accordance with BS 5930: 1999 (BSI, 2007), and the results are shown in Table 3.1. The time lag from Method 1(b) of the standard was used for estimating the hydraulic conductivity:

\[ k = \frac{A}{FT} \]  

where:
\( k \) is the permeability of the soil
\( A \) is the cross-sectional area of the standpipe
\( F \) is the intake factor as in equation (3.2)
\( T \) is the basic time factor as proposed by Hvorslev (1951)

\[ \frac{F}{D} = \frac{2.32\pi(\frac{L}{D})}{\ln e\left(1.1(\frac{L}{D}) + \sqrt{1 + 1.1(\frac{L}{D})^2}\right)} \]  

were for cylindrical piezometers \( L \) is the length and \( D \) the diameter.

<table>
<thead>
<tr>
<th>Piezometer</th>
<th>Hydraulic conductivity, ( k ) (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-1 at 4.2 m</td>
<td>2.59x10(^{-5})</td>
</tr>
<tr>
<td>BH-1 at 6.3 m</td>
<td>1.96x10(^{-6})</td>
</tr>
<tr>
<td>BH-2 at 7.0 m</td>
<td>1.15x10(^{-5})</td>
</tr>
</tbody>
</table>
3.4 LABORATORY TESTS

Classification tests and nine oedometer tests were conducted by the author on the samples collected during the field investigation at Ballydermot bog.

3.4.1 Classification of peat

3.4.1.1 Degree of humification, $H_N$

The degree of humification was determined using the squeeze test as described by Head (2006). The von Post values for Ballydermot peat ranged between $H_4$ and $H_7$, for the entire soil profile, classifying the peat as pseudo-fibrous (Figure 3.21(a)).

For the same peat, Hanrahan (1964) found humification values from $H_5$ to $H_{6.5}$ (pseudo-fibrous to amorphous). Hebib (2001) reported von Post values ranging between $H_5$ and $H_0$ (pseudo-fibrous to amorphous) for samples collected at depths between 0.5 to 2.5 m. Meanwhile, Walsh (2009) reported a von Post rating of $H_3$ (fibrous) for peat samples collected approximately 5.0 m to the west of the test area shown in Figure 3.9. The sampling depth varied between 0.65 and 0.9 m, after removing the peaty fill.

3.4.1.2 Water content, $w$

For mineral soils, the water content is usually estimated by comparing the mass of soil before and after oven drying the sample at 105°C for 24 h. However, some authors such as Landva et al. (1983) consider that drying peat at 105°C will result in charring of the organic constituents, which would lead to measured water contents in excess of the true value. MacFarlanen and Allen (1964) conducted a study in the determination of the peat water content, with a range of temperatures from 75 to 120°C increasing in increments of 5°C, and concluded that the temperature of drying should never exceed 95°C and that preferably the temperature should not exceed 85°C. In a study using different temperatures (60, 85, 100, 105 and 110°C) to obtain the water content of different organic soils and peats, Skempton and Petley (1970) concluded that little advantage is gained by adopting temperatures lower than 105°C.
Figure 3.21: Profile of soil properties for Ballydermot bog (a) degree of humification, (b) water content, and (c) organic content.
Figure 3.21 (cont.): Profile of soil properties for Ballydermot bog (d) fibre content, (e) specific gravity, and (f) acidity.
O'Kelly (2005b) tested nine different mineral and organic soils and peats, at temperature ranges between 60 and 150°C, and found that for peats there could be a dry mass reduction of up to 4.4% when using an oven temperature of 105°C. In a separate study, O'Kelly (2005a) found that for organic soils (70% organic content) a larger error arises when the water content of the tested soil was determined using the specimen dry mass recorded at 60°C rather than the standard drying temperature of 110 ± 5°C, and that the true value of the water content corresponded to an oven drying temperature of 86°C.

Figure 3.21(b) presents the results of the water content, on the samples recovered from the mechanical and manual boreholes, dried at a temperature of 105°C for 24 h, or until constant weight was achieved. Nonetheless, paired samples from the same bags, for BH-1 and BH-2, were also oven dried at 80°C for the same 24 h period, or until constant weight was achieved, in order to observe if there was any significant difference in the results (Figure 3.22).
A two-sample T-Test, with a 95% confidence interval, was carried on the paired samples results for each borehole, in order to assess if the difference in the results was statistically significant or not (i.e. if the results are different or not). The T-Tests were carried using the statistical software Minitabs 15, and the results are shown in Table 3.2 and Table 3.3.

**Table 3.2:** Two-Sample T-Test and CI for BH-1 for two different temperatures.

<table>
<thead>
<tr>
<th></th>
<th>N</th>
<th>Mean</th>
<th>SE Mean</th>
</tr>
</thead>
<tbody>
<tr>
<td>80°C(BH-1)</td>
<td>8</td>
<td>724</td>
<td>148</td>
</tr>
<tr>
<td>150°C(BH-1)</td>
<td>8</td>
<td>762</td>
<td>137</td>
</tr>
</tbody>
</table>

Difference = \( \mu (80°C(BH-1)) - \mu (150°C(BH-1)) \)

Estimate for difference: -38

95% CI for difference: (-473.397)

T-Test of difference = 0 (vs not =): T-Value = -0.19 P-Value = 0.855 DF = 13

**Table 3.3:** Two-Sample T-Test and CI for BH-2 for two different temperatures.

<table>
<thead>
<tr>
<th></th>
<th>N</th>
<th>Mean</th>
<th>SE Mean</th>
</tr>
</thead>
<tbody>
<tr>
<td>80°C(BH-2)</td>
<td>10</td>
<td>825</td>
<td>113</td>
</tr>
<tr>
<td>150°C(BH-2)</td>
<td>10</td>
<td>826</td>
<td>114</td>
</tr>
</tbody>
</table>

Difference = \( \mu (80°C(BH-2)) - \mu (150°C(BH-2)) \)

Estimate for difference: -1

95% CI for difference: (-340.338)

T-Test of difference = 0 (vs not =): T-Value = -0.00 P-Value = 0.997 DF = 17

Table 3.2 and Table 3.3, show that the P-Values for both tests are higher than 0.05, indicating that the water contents obtained under both temperatures are statically the same; hence, it can be concluded that for engineering purposes a temperature of 105°C is acceptable.

The water content for the studied peat varies 650 and 1350%, as shown in Figure 3.21(b). It is notable that the upper layer close to the peaty fill, there is a reduction in the water
content to around 600%; while the water content reduces drastically for the samples in the peat-gravel interface, to values as low as 180%, probably due to the materials mixture. Hanrahan (1954) found water contents in excess of 1400% for undrained peat at Ballydermot, and ranging from 1000% for shallow drainage to 700% for deep drains. Hebib (2001) reported water content values between 750 and 950%, while Walsh (2009) obtained values from 1000 to 1100%. Cuddy (1988) reported a mean value of 979%. All authors used a temperature of 105°C.

3.4.1.3 Organic content

The organic content of peat was determined by the mass loss on ignition at a temperature of 450°C, with a 5 h combustion period. Before ignition, the samples were prepared according to BS 1377:1990 part 3 (BSI, 1990), except that the samples were oven dried at 105°C. Good experience has been obtained at TCD using this procedure, originally recommended by Arman (1971). Furthermore, the same temperature for drying the samples before charring is established in ASTM 2974 – 07a (2007a).

In general, organic contents between 93 and 98% were obtained (Figure 3.21(c)), except for one result of 87% in BH-1. For the samples gather from the peat-clay interface zone, the organic contents reduce significantly, indicating a higher presence of mineral soils. Hebib (2001) observed values ranging from 94 to 98%, while Walsh found an organic content of 97% for the Ballydermot peat.

It is to note that the ASTM D 4427 -07 (2007b) classification system requires the use of the ash content, rather than the organic content. The ash content can be obtained by the following difference: Ash content (%) = 100 – Organic content (%).

3.4.1.4 Fibres content

Three fibre content tests were conducted on samples from the upper three pistons from BH-1. The tests were carried in accordance with ASTM 1997 – 91 (2008). According to ASTM D 4427 – 07 (2007b) depending on the fibre content obtained from the test the peat can be classified as: Fibric (FC > 67%), Hemic (33% < FC < 67%) and Sapric (FC < 33%).
For Ballydermot bog, fibre contents of between 40 and 64% were recorded (Figure 3.21(d)).

Considering the fibres presented in Figure 3.23, it can be said that a moderately content of both, fine (smaller than 1 mm) and coarse fibres (longer than 1 mm), is present. A small amount of wood remnant is also present.

![Figure 3.23: Fibres in Ballydermot peat (Walsh, 2009).](image)

### 3.4.1.5 Specific gravity, $G_s$

The specific gravity of an organic soil depends upon the amount of mineral and organic constituents present on it (Hobbs, 1986). Thus, this property is more variable in peat and other organic soils than in mineral soils. The specific gravity was determined in accordance with BS 1377: 1990 part 2 (BSI, 1990), using white spirit instead of water. The density of the white spirit is 0.778 Mg/m$^3$.

Figure 3.21(e) presents the results of the specific gravity, on the samples recovered from the mechanical and manual boreholes. The average $G_s$ value of the peat was 1.47, while for the gravel was 2.70.
Skempton and Petley (1970), proposed equation (3.3) as a convenient method to calculate the specific gravity of organic soils from the proportion of pure organic matter to mineral matter, using the loss on ignition test to obtain the organic content.

\[
G_s = \frac{G_{s(\text{org})} \cdot G_{s(\text{min})}}{(G_{s(\text{org})} - G_{s(\text{min})})(1 - C(1 - N)) + G_{s(\text{min})}}
\]  

(3.3)

where:

- \(G_s\) is the specific gravity of the soil
- \(G_{s(\text{org})}\) is the specific gravity of the organic matter
- \(G_{s(\text{min})}\) is the specific gravity of the mineral matter
- \(N\) is the loss on ignition expressed as a proportion
- \(C\) is a correction factor

It has been accepted that the specific gravity of mineral soils is about 2.7 (Skempton and Petley, 1970), and about 1.4 to 1.5 for peat based on the specific gravity of cellulose and lignin (Hobbs, 1986). Considering this, Skempton and Petley (1970) proposed to use 1.4 and 2.7 as the values for the specific gravity of the organic and mineral matters, respectively, in equation (3.3). A number of authors have adopted these values since then (Hobbs, 1986; Nichol and Farmer, 1998; Farrell, in press).

The correction factor, \(C\), compensates for the loss of mineral matter for furnace temperatures exceeding 450°C. Skempton and Petley (1970) suggested a \(C\) value of 1.04 for a temperature of 550°C. O'Loughlin (2001) found that \(C\) may be taken as 1.0 for peat with organic contents greater than 30 – 40% when following the recommendations of Arman (1971).

Figure 3.24 shows the correlation of the loss on ignition versus the specific gravity values given in Figure 3.21(c) and (e) for Ballydermot peat. The specific gravity values calculated using equation (3.3) are also plotted. For the calculation, \(C\) was taken as 1.0 since a temperature of 450°C was used to estimate the loss on ignition (see §3.4.1.3). The \(G_{s(\text{min})}\) used was 2.7 as this is the average for the boulder clay and the recommended value in the
litterature. Two values were used for $G_{s(\text{org})}$, 1.40 as recommended and 1.47 as the average obtained for Ballydermot peat.

As can be seen from Figure 3.24, good agreement is achieved using equation (3.3) to estimate the specific gravity of organic soils.

![Figure 3.24: Correlation of specific gravity with loss on ignition for Ballydermot bog.](image)

Hanrahan (1953) reported specific gravity values to be between 1.20 to 1.70. Hebib (2001) estimated that the specific gravity for Ballydermot peat is 1.20, while Walsh (2009) obtained a value of 1.49. Hobbs (1986) reported that values as low as 1.10 have been measured, but cautioned about these values, since the specific values of its constituents are in the range of 1.40 to 1.50, as mentioned above.

### 3.4.1.6 Acidity, $pH$

According to Hobbs (1986) there is a correlation between the type of vegetation, the chemistry of the water, and the acidity or alkalinity of the peat and the peat water. As a rough general rule, this author stated that the $pH$ of fen peat tends to be greater than about
5, the higher the pH the richer the fen; acid fen peat will have a pH less than 5; bog peat has a pH in the range from 3.3 to 4.5; and transitional peats fall in the range from 4 to 6.

Figure 3.21(f) presents the variation of the pH of the peat with depth at Ballydermot bog. The pH values were obtained using ASTM 2976 – 71 (2004). The value of pH for Ballydermot peat ranges between 4.6 and 5.6, as expected; however, at greater depths, closer to the gravel, the pH starts to increase reaching values of 7.6 to 8.0 for the gravel layer. These alkaline values in the gravel can be explained due to the presence of calcium minerals, as described by Hammond (1981). A pH value of 4.9 was measured by Hebib (2001) and of 5.0 to 5.4 by Walsh (2009), for Ballydermot peat.

3.4.1.7 Botanical composition

Sir Harry Godwin described the botanical composition of the peat at Ballydermot bog (Cuddy, 1988) as follows: (i) a 4.6 m upper layer of bog peat mainly sphagnum and calluna species, (ii) a 3.2 m middle layer composed by a mixture of fen and bog peat mainly sedge (carex and cladium) and sphagnum species, (iii) underlain by a 0.9 m thick layer of fen peat composed by alnus and betula wood peat and phragmited reed peat.

Hanrahan (1964) also described three main layers of peat at Ballydermot bog, namely: (i) 3.3 m of sphagnum moss and other species, (ii) 1.5 m of fen peat composed by some wood peat (birch of the betula family) followed by sedge species (carex and cladium), and (iii) 2.4 m of cladium and phragmites fen peat, and reed swamp (reed peat) slightly woody at the bottom.

Hammond (1969) found a similar stratigraphy of alnus and betula fen peat underlying a layer of sphagnum bog peat with the presence of some localised calluna species. Hebib (2001) described the stratigraphy as old sphagnum peat underlain by reed swamp peat.

The stratigraphy and botanical composition of the peat described above, with fen and reed peat underlying bog peat, is in accordance with the geological formation process of raised bogs in the Irish mid-lands described by Hammond (1981) and by Mitchell (1986).
3.4.1.8 Classification

Based on the description given above, the peat from Ballydermot bog was classified using two different systems.

In general, Ballydermot peat properties can be summarised as follows: peat with a degree of humification of $H_4$ to $H_7$, a water content of 65% and 135%, and organic content of 93 to 98% (or ash content of 2 to 7%), a fibre content of 40 to 64%, a specific gravity of 1.47, a pH of 4.6 to 5.6, and a main botanical composition of Sphagnum – Carex – Cladium – Alnus – Betula – Phragmites peat.

According to the ASTM 4427 – 07 (2007b), Ballydermot peat can be then classified as; Fibric to Hemic, Low to Medium Ash, Moderately Acidic, Sphagnum – Carex – Cladium – Alnus – Betula – Phragmites peat.

According to the extended von Post system by Hobbs (1986):

\[ SCWPh \; H_{4-7} \; B_{3-4} \; F_2 \; R_2 \; W_1 \; N_5 \; A_0 \; pH_1 \]

3.4.2 Classification of mineral soils

The classification of the mineral soils underlying the peat, at Ballydermot bog, was done according to the British Soil Classification System (BSCS) described in BS 5930: 1999 (BSI, 2007), and it is presented in Table 3.4. The particle size distribution curves and plasticity charts, for all samples from both boreholes, can be found in Appendix A.

In Ireland, the glacial soils formed during the glacial period are usually called ‘boulder clay’. Farrell (2010) suggested that, for Irish conditions, this term can be applied to soils with a fine content equal or greater than 35%, as this would be sufficient for the soil to support a shallow excavation (approximately 3.0 m) in a temporary condition, provided it has the required undrained shear strength.
### Table 3.4: Mineral soils classification according to the BSCS.

<table>
<thead>
<tr>
<th>Sample</th>
<th>% Gravel</th>
<th>% Sand</th>
<th>% Fines</th>
<th>LL (%)</th>
<th>PI (%)</th>
<th>Organic content (%)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-1 / 4.0-4.2 m</td>
<td>11.2</td>
<td>56.7</td>
<td>32.1</td>
<td>58</td>
<td>13</td>
<td>10</td>
<td>Organic very silty gravelly SAND</td>
</tr>
<tr>
<td>BH-1 / 4.2-4.4 m</td>
<td>34.3</td>
<td>27.1</td>
<td>38.7</td>
<td>22</td>
<td>15</td>
<td>N/A</td>
<td>Slightly sandy and gravelly CLAY</td>
</tr>
<tr>
<td>BH-1 / 4.5-4.9 m</td>
<td>49.0</td>
<td>21.2</td>
<td>29.8</td>
<td>20</td>
<td>14</td>
<td>N/A</td>
<td>Very clayey sandy GRAVEL</td>
</tr>
<tr>
<td>BH-1 / 5.2-5.7 m</td>
<td>79.8</td>
<td>7.9</td>
<td>12.3</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>Silty or clayey slightly sandy GRAVEL</td>
</tr>
<tr>
<td>BH-1 / 5.8-6.2 m</td>
<td>74.9</td>
<td>20.9</td>
<td>4.2</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>Slightly silty or clayey very sandy GRAVEL</td>
</tr>
<tr>
<td>BH-1 / 6.2-6.3 m</td>
<td>72.2</td>
<td>16.7</td>
<td>11.1</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>Silty or clayey sandy GRAVEL</td>
</tr>
<tr>
<td>BH-2 / 4.0-4.5 m</td>
<td>39.2</td>
<td>28.0</td>
<td>32.8</td>
<td>48</td>
<td>11</td>
<td>16</td>
<td>Organic very sandy very silty GRAVEL</td>
</tr>
<tr>
<td>BH-2 / 5.0-5.3 m</td>
<td>51.67</td>
<td>20.18</td>
<td>28.15</td>
<td>20</td>
<td>6</td>
<td>N/A</td>
<td>Very sandy very silty GRAVEL</td>
</tr>
<tr>
<td>BH-2 / 5.6-5.8 m</td>
<td>91.33</td>
<td>4.40</td>
<td>4.27</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>Slightly silty or clayey slightly sandy GRAVEL</td>
</tr>
<tr>
<td>BH-2 / 6.0-6.1 m</td>
<td>89.63</td>
<td>4.93</td>
<td>5.43</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>Slightly silty or clayey slightly sandy GRAVEL</td>
</tr>
</tbody>
</table>
If this criterion is applied to the soils in Table 3.4, only a small layer of 0.2 m in BH-1 would fall into this category. However, as it was stated in §3.2.3, Hammond (1969) and Hebib (2001) had classified previously these soils as boulder clay.

In the author’s view, the mineral soil found at Ballydermot can be described in general manner as a: GRAVEL with very high contents of fines and sand at the top, reducing with depth, with some organic materials present at the peat-gravel interface.

### 3.4.3 One-dimensional compressibility of Ballydermot peat

A series of nine standard oedometer tests were carried on samples extracted from three pistons obtained from BH-1. Particular care was taken during the preparation of the sample due to the compressible nature of the peat. The pistons were placed in a clamp, and the samples were pushed out using a manual extruder. A piece of peat, larger than the specimen, was then cut using a large, sharp, serrated knife. This knife was used in order to avoid any undesirable compression induced by pushing a flat bladed knife. The peat was then placed on a flat wooden table. The oedometer ring was placed on top of the sample and used as a guide to trim the specimen to the appropriate diameter. Afterwards, the ring was move downwards, very slowly, and any excess material was cut using the serrated knife. Subsequently, the top and bottom of the specimen were trimmed at the level of the ring. Any voids on the surface were filled with the excess trimmings, and the remaining material was used for index properties testing as recommended by Head (1994). After the specimen was ready, the ring was placed in the consolidation cell, and the cell was filled with distilled water. All oedometer frames used were lever arm.

The oedometer tests carried out are listed on Table 3.5. All tests were conducted using the procedures set out in BS 1377: 1990 part 5 (BSI, 1990), but with the loading steps given on Table 3.6.

For each piston, two incremental tests and a long term single increment test were conducted (Table 3.6). Generally the loads were maintained for 24 h, with at least one increment being left on for a greater period. In the three single increment tests, the 50 kPa load steps were maintained for periods of 10, 14 and 62 days.
Table 3.5: Oedometer tests description.

<table>
<thead>
<tr>
<th>PISTON</th>
<th>TEST</th>
<th>Diameter (mm)</th>
<th>Height (mm)</th>
<th>$e_v$</th>
<th>$\gamma_s$ (kN/m³)</th>
<th>$\gamma_s$ (kN/m³)</th>
<th>$G_s$</th>
<th>$w$ (%)</th>
<th>$S_r$ (%)</th>
<th>LOI (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-1 0.90 – 1.55 m</td>
<td>Oedometer 1</td>
<td>75.0</td>
<td>19.05</td>
<td>8.69</td>
<td>10.32</td>
<td>1.59</td>
<td>1.57</td>
<td>549</td>
<td>99</td>
<td>79</td>
</tr>
<tr>
<td></td>
<td>Oedometer 2</td>
<td>76.2</td>
<td>18.50</td>
<td>6.78</td>
<td>10.56</td>
<td>2.01</td>
<td>1.59</td>
<td>427</td>
<td>100</td>
<td>78</td>
</tr>
<tr>
<td></td>
<td>Oedometer 4</td>
<td>76.2</td>
<td>17.80</td>
<td>6.81</td>
<td>10.48</td>
<td>1.98</td>
<td>1.58</td>
<td>428</td>
<td>99</td>
<td>81</td>
</tr>
<tr>
<td>BH-1 1.50 – 2.30 m</td>
<td>Oedometer 1</td>
<td>75.0</td>
<td>19.05</td>
<td>14.50</td>
<td>9.96</td>
<td>0.96</td>
<td>1.51</td>
<td>942</td>
<td>98</td>
<td>98</td>
</tr>
<tr>
<td></td>
<td>Oedometer 2</td>
<td>76.2</td>
<td>18.42</td>
<td>13.50</td>
<td>10.12</td>
<td>0.97</td>
<td>1.44</td>
<td>939</td>
<td>101</td>
<td>98</td>
</tr>
<tr>
<td></td>
<td>Oedometer 4</td>
<td>76.2</td>
<td>18.64</td>
<td>13.43</td>
<td>10.24</td>
<td>0.99</td>
<td>1.46</td>
<td>932</td>
<td>100</td>
<td>98</td>
</tr>
<tr>
<td>BH-1 2.40 – 3.20 m</td>
<td>Oedometer 1</td>
<td>75.0</td>
<td>19.05</td>
<td>12.50</td>
<td>9.57</td>
<td>1.05</td>
<td>1.45</td>
<td>809</td>
<td>94</td>
<td>98</td>
</tr>
<tr>
<td></td>
<td>Oedometer 2</td>
<td>76.2</td>
<td>18.40</td>
<td>11.51</td>
<td>10.17</td>
<td>1.08</td>
<td>1.38</td>
<td>839</td>
<td>101</td>
<td>98</td>
</tr>
<tr>
<td></td>
<td>Oedometer 4</td>
<td>76.2</td>
<td>18.60</td>
<td>12.15</td>
<td>10.46</td>
<td>1.08</td>
<td>1.45</td>
<td>867</td>
<td>103</td>
<td>98</td>
</tr>
</tbody>
</table>

Table 3.6: Loading / unloading steps duration.

<table>
<thead>
<tr>
<th>Load Increment (kPa)</th>
<th>BH-1 0.90 – 1.55 m</th>
<th>BH-1 1.50 – 2.30 m</th>
<th>BH-1 2.40 – 3.20 m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Oedometer 1</td>
<td>Oedometer 2</td>
<td>Oedometer 4</td>
</tr>
<tr>
<td>6.25</td>
<td>---</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>12.5</td>
<td>---</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>25</td>
<td>---</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>50</td>
<td>14</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>100</td>
<td>---</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>200</td>
<td>---</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>100</td>
<td>---</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>50</td>
<td>---</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>25</td>
<td>---</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>12.5</td>
<td>---</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

The results of the nine consolidation tests carried out on Ballydermot peat, plotted in terms of void ratio at the end of the load increment, $e_v$, versus the logarithm of the vertical effective stress, $\sigma'_{v}$, are shown in Figure 3.25.
In the following sections, the laboratory results gathered from the oedometer tests will be analysed using the models described in §2.3.

### 3.4.3.1 The end of primary approach

Equations (2.15) and (2.16) were used to model the consolidation behaviour observed during the six incremental oedometer tests, and to calculate the values of the compression index ($C_c$), swell index ($C_s$) and preconsolidation pressure ($\sigma'_p$). As an example, Figure 3.26 shows the model for specimen BH-1 Oedometer 2 D = 0.90 – 1.55 m. The rest of the models can be found in Appendix A, Figure A.7 to Figure A.12. Table 3.7 presents a summary of the parameters calculated.

The compression – time behaviour for Ballydermot peat, using the EOP approach, was studied using the Casagrande and Taylor graphical constructions, as described by Head (1994). From these constructions, the time for end of primary consolidation was obtained as $t_{100-c}$ and $t_{100-t}$, where the subscripts $c$ and $t$ differentiate between Casagrande and Taylor
respectively. Moreover, in order to evaluate the variation of the permeability \(k\) with the reduction in void ratio \(e\), the coefficient of volume compressibility \(m_v\) and the coefficient of consolidation \(c_s\) were also evaluated using both methods, and can be readily identified by the aforementioned subscripts. The permeability behaviour will be discussed in detailed in §3.4.3.3. The secondary compression index, \(C_a\), was determined using the linear portion of the \(e - \log t\) curve as described in §2.3.1, and the values are given in Figure 3.27. The rest of the parameters determined are presented in Table A.1 to Table A.9 in Appendix A, while the Casagrande and Taylor constructions are shown in Figure A.13 to Figure A.21 of the same Appendix.

From Table 3.7 it can be seen that the preconsolidation pressure for Ballydermot peat ranges between 8 and 11 kPa for all depths. Hebib (2001), determined a value of 15 kPa for the same peat, while O’Loughlin (2001) estimated it as 17 kPa. Hanrahan (1954) presented results of consolidation tests on remoulded samples from Ballydermot bog; however, the results of one oedometer tests on an undisturbed sample was shown and a preconsolidation pressure of 21 kPa was estimated. The preconsolidation pressure of Ballydermot bog is quite high when compared with the values determined for other bogs.
such as 3 kPa for Clara bog peat (O’Loughlin, 2001), or 5 kPa for Raheenmore peat (Hebib, 2001). However, this high value can be explained as Ballydermot bog has been drained and used as a cut-away bog for the last 60 years (see §3.2.3).

Table 3.7: Compressibility parameter for Ballydermot peat using the EOP approach.

<table>
<thead>
<tr>
<th>PISTON</th>
<th>TEST</th>
<th>( \sigma'_p ) (kPa)</th>
<th>( C_C )</th>
<th>( C_S )</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-I</td>
<td>Oedometer 1</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>0.90 - 1.55 m</td>
<td>Oedometer 2</td>
<td>10.0</td>
<td>2.05</td>
<td>0.37</td>
</tr>
<tr>
<td></td>
<td>Oedometer 4</td>
<td>10.5</td>
<td>2.25</td>
<td>0.46</td>
</tr>
<tr>
<td>BH-I</td>
<td>Oedometer 1</td>
<td>8.0</td>
<td>5.05</td>
<td>0.96</td>
</tr>
<tr>
<td>1.50 - 2.30 m</td>
<td>Oedometer 2</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td></td>
<td>Oedometer 4</td>
<td>8.0</td>
<td>5.51</td>
<td>0.93</td>
</tr>
<tr>
<td>BH-I</td>
<td>Oedometer 1</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>2.40 - 3.20 m</td>
<td>Oedometer 2</td>
<td>9.0</td>
<td>4.79</td>
<td>0.62</td>
</tr>
<tr>
<td></td>
<td>Oedometer 4</td>
<td>11.0</td>
<td>6.13</td>
<td>0.78</td>
</tr>
</tbody>
</table>

Figure 3.27: Variation of the coefficient of secondary consolidation, \( C_{in} \), with the vertical effective stress.

A striking characteristics of the peat tested, is the marked difference between the surficial and the deeper specimens. The void ratio of the samples from piston BH-1 \( D = 0.90 - 1.55 \)
m are approximately half of the void ratios found on the samples taken from pistons BH-1 D = 1.50 – 2.30 m and BH-1 D = 2.40 – 3.20 m. This difference can also be seen in the compression index, while for the deeper peat $C_C$ ranges between 4.79 and 6.13, for the samples closer to the surface is 2.15 in average (Table 3.7). With respect to the long term creep behaviour the difference is also present, though to a lesser extent. The secondary compression index value of the surface samples ranges from 0.039 to 0.140, and for the deeper ones is between 0.059 and 0.390.

Hebib (2001) found an average value for $C_C = 6.12$ while $C_r$ ranged between 0.19 and 0.47 for Ballydermot peat. O’Loughlin (2001) measured a wider range for the secondary compression index, varying from 0.05 to 0.6. Comparing these two sets of results, with the ones presented above, it seem that the peat from these studies is more similar to the peat found in the deeper deposits.

3.4.3.2 Janbu’s resistance concept

The stress – strain behaviour and the strain – time behaviour of Ballydermot peat were also analysed using the resistance concept (Janbu, 1969). The tangent modulus $(M)$, the modulus number $(m)$, the time resistance of the soil $(R)$ and the creep resistance number $(r_s)$, were determined using the procedure described in §2.3.2.

As an example, Figure 3.28 shows the stress – strain resistance concept applied to sample BH-1 Oedometer 4 D = 2.40 to 3.20 m. Also, Figure 3.29 shows the time – strain behaviour concept applied to the 50 kPa load increment for the same oedometer test. The rest of the strain – stress figures can be found in Appendix A, Figure A.22 to Figure A.27. The time – strain figures are shown from Figure A.28 to Figure A.65, and the creep resistance number values, $r_s$, are given in Table A.1 to Table A.9, of the same appendix.

Table 3.8 presents the preconsolidation pressure calculated using the stress – strain resistance concept $(\sigma_{rc,J M})$, along with the same parameter determined using the strain – time concept $(\sigma_{rc,J R})$. As can be seen, the former method yielded results closer to the ones obtained using the EOP approach. The values of the modulus number, $m$, are also
presented in the table. Figure 3.30 shows the variation of the creep resistance number, $r_s$, with the vertical effective stress.

**Figure 3.28**: Stress–strain resistance concept applied to BH-1 Oed 4 D = 2.40 to 3.20 m.
Figure 3.29: Strain – time resistance concept applied to the 50 kPa load increment of BH-1 Oed 4 D = 2.40 to 3.20 m.
Table 3.8: Compressibility parameter for Ballydermot peat using the resistance concept.

<table>
<thead>
<tr>
<th>PISTON</th>
<th>TEST</th>
<th>$\sigma_{p-1-M}$ (kPa)</th>
<th>$\sigma_{p-1-N}$ (kPa)</th>
<th>$m$</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-1</td>
<td>Oedometer 1</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>0.90 - 1.55 m</td>
<td>Oedometer 2</td>
<td>12.0</td>
<td>28</td>
<td>5.95</td>
</tr>
<tr>
<td></td>
<td>Oedometer 4</td>
<td>12.0</td>
<td>28</td>
<td>5.86</td>
</tr>
<tr>
<td>BH-1</td>
<td>Oedometer 1</td>
<td>N/A</td>
<td>25</td>
<td>5.30</td>
</tr>
<tr>
<td>1.50 - 2.30 m</td>
<td>Oedometer 2</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td></td>
<td>Oedometer 4</td>
<td>12.0</td>
<td>12</td>
<td>4.52</td>
</tr>
<tr>
<td>BH-1</td>
<td>Oedometer 1</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>2.40 - 3.20 m</td>
<td>Oedometer 2</td>
<td>8.0</td>
<td>25</td>
<td>4.51</td>
</tr>
<tr>
<td></td>
<td>Oedometer 4</td>
<td>13.0</td>
<td>25</td>
<td>4.63</td>
</tr>
</tbody>
</table>

Figure 3.30: Variation of the creep resistance number, $r_c$, with the vertical effective stress.

Unlike the $C_c$ values, the modulus number values ($m$) do not differ greatly between for the surficial and deeper deposits, though a slight reduction is present in the latter ones. For the surficial deposits $m \approx 5.90$, while for the deeper peat it ranged between 4.02 and 5.30. The
difference is reflected, however, on the maximum strain ($\varepsilon$) reached by the specimens. While the surficial samples reached an average $\varepsilon \approx 41\%$, the strain for the deeper samples ranged between 57 and 65%.

Lefebvre et al. (1984) reported values for $m$ between 2.56 and 3.57, for a Quebec fibrous peat (see §2.3.2.1). However, these values were determined using the natural strain rather than the linear strains, which reduced the tangent modulus values greatly, and in turn reduced the values of the modulus number.

As has been shown for clay (Janbu, 1969; 1985; Havel, 2004), in peat the creep resistance number, $r_c$, also seems to be dependent on the level of stress applied (Figure 3.30).

The strain–time figures adequately describe the secondary compression of peat for the 24 h, 36 h, 48 h and 72 h tests; showing the expected linear behaviour after an initial parabolic period. Furthermore, in some cases the initial parabolic behaviour is very small or does not even appear. This is due to the rapid dissipation of excess pore water pressure typically observed in peat, particularly fibrous and pseudo-fibrous peat. However, for the long term tests (10 days, 14 days and 62 days) there is an initial tendency of the specimen to behave linearly, but after certain time the linearity of the curve is distorted showing scatter behaviour (Figure A.28, Figure A.46 and Figure A.53). This behaviour may be explained by the tertiary compression observed in peat soil, due to the decay of the organic matter (see §2.3), however further research is needed.

Despite the strain–time figures correctly depicted the linear secondary compression of most samples, there is a difference on the preconsolidation pressure values estimated when compared with the values determined using the stress–strain figures. For the BH-1 Oedometer 2 D = 2.40 – 3.20 m, the value found was over three times higher when using the strain–time figure. However, due to the low preconsolidation pressures found, it seems that the $r_s$ – time curve and the $M - \sigma'_v$ curve were not sufficiently defined at the low stress levels by the load increments used, reducing the accuracy in the determination of the preconsolidation pressure.
3.4.3.3 Permeability characteristics

The permeability of Ballydermot peat was computed from the oedometer tests results, as described by Head (1994), using the Casagrande and Taylor graphical constructions. The results obtained are shown in Figure 3.31, in an $e - \log k_e$ plot.

As can be seen from Figure 3.31, the coefficient of permeability of Ballydermot peat follows the linear semi-logarithmic relationship suggested by Tavenas et al. (1983b), as previously described in §2.3.4. Two permeability change index, $C_k$, were identified for Ballydermot peat, one for the surficial more consolidated deposits, and one for the deeper deposits with higher void ratios. As has been reported (Tavenas et al., 1983a; Mesri et al., 1997), the $C_k$ values dependent on the initial void ratio value of the soil.

For the deeper deposits, the $C_k = 3.44$ using Casagrande construction and $C_k = 3.71$ with Taylor construction. For the surface ones, $C_k = 1.80$ with the first method and $C_k = 1.79$ with the second. Mesri et al. (1997) suggested that $C_k = 0.25e_{ov}$, which is in good agreement with the results presented here (Table 3.9). The full set of values determined can be found in Table A.1 to Table A.9, from Appendix A.

Table 3.9: $C_k / e_{ov}$ values for Ballydermot peat.

<table>
<thead>
<tr>
<th>PISTON</th>
<th>TEST</th>
<th>$e_{ov}$</th>
<th>$C_k$</th>
<th>$C_k / e_{ov}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-I</td>
<td>Oedometer 1</td>
<td>8.69</td>
<td>1.80</td>
<td>0.21</td>
</tr>
<tr>
<td>0.90 – 1.55 m</td>
<td>Oedometer 2</td>
<td>6.78</td>
<td>0.27</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Oedometer 4</td>
<td>6.81</td>
<td></td>
<td>0.26</td>
</tr>
<tr>
<td>BH-I</td>
<td>Oedometer 1</td>
<td>14.50</td>
<td>3.44</td>
<td>0.23</td>
</tr>
<tr>
<td>1.50 – 2.30 m</td>
<td>Oedometer 2</td>
<td>13.50</td>
<td></td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>Oedometer 4</td>
<td>13.43</td>
<td></td>
<td>0.26</td>
</tr>
<tr>
<td>BH-I</td>
<td>Oedometer 1</td>
<td>12.50</td>
<td>3.44</td>
<td>0.28</td>
</tr>
<tr>
<td>2.40 – 3.20 m</td>
<td>Oedometer 2</td>
<td>11.51</td>
<td></td>
<td>0.30</td>
</tr>
<tr>
<td></td>
<td>Oedometer 4</td>
<td>12.15</td>
<td></td>
<td>0.28</td>
</tr>
</tbody>
</table>

An interesting characteristic that can be noticed in Figure 3.31 is that the surficial materials have a slightly higher initial value of permeability, in spite of having half the initial void ratio of the materials found at greater depths.
Figure 3.31: $e - \log k_c$ variation for Ballydermot peat: (a) Casagrande construction, (b) Taylor construction.

Hebib (2001) measured the permeability of Ballydermot peat using the Rowe cell apparatus and also found good agreement with the linear semi-logarithmic relationship
used here, and a $C_k = 5.0$. The results obtained by Hanrahan (1954; 1964), and processed by Hobbs (1986) in a $\log e - \log k_c$ plot (Figure 2.59), also shows a linear agreement for Ballydermot peat.

3.4.3.4 Numerical models: The Soft Soil and Soft Soil Creep models

All the oedometer tests were modelled using finite element package PLAXIS version 2010, with both, the Soft Soil and the Soft Soil Creep models, as described in §2.3.3. Because of the symmetry of the problem, an axisymmetric model of one half of the oedometer was used, with 15-node triangular elements. Figure 3.32 shows the typical model used, including mesh, load, fixities, boundaries and initial water conditions. At the vertical mesh boundaries horizontal movement is prevented, while the bottom boundaries are fixed for horizontal and vertical displacement. The upper horizontal boundary is free to move in the vertical direction, and a linear distributed load is applied (Figure 3.32(a)). The material is free to drain at the top and bottom boundaries, and the water initial level is set at upper boundary (Figure 3.32(b)).

**Table 3.10:** Estimated values of $\lambda^*$ and $k^*$ from $C_r$ and $C_s$.

<table>
<thead>
<tr>
<th>PISTON</th>
<th>TEST</th>
<th>$\lambda^*$</th>
<th>$k^*$</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-1</td>
<td>Oedometer 1</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>0.90 - 1.55 m</td>
<td>Oedometer 2</td>
<td>0.114</td>
<td>0.042</td>
</tr>
<tr>
<td></td>
<td>Oedometer 4</td>
<td>0.125</td>
<td>0.051</td>
</tr>
<tr>
<td>BH-1</td>
<td>Oedometer 1</td>
<td>0.142</td>
<td>0.054</td>
</tr>
<tr>
<td>1.50 - 2.30 m</td>
<td>Oedometer 2</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td></td>
<td>Oedometer 4</td>
<td>0.166</td>
<td>0.056</td>
</tr>
<tr>
<td>BH-1</td>
<td>Oedometer 1</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>2.40 - 3.20 m</td>
<td>Oedometer 2</td>
<td>0.166</td>
<td>0.043</td>
</tr>
<tr>
<td></td>
<td>Oedometer 4</td>
<td>0.202</td>
<td>0.051</td>
</tr>
</tbody>
</table>

The design parameters ($\lambda^*$, $k^*$, $\mu^*$) were initially estimated using the EOP approach parameters (Table 3.10), as described in §2.3.3, and later refined by a trial and error process, varying each parameter until good agreement was reached, between the laboratory data and the model results.
In Plaxis, the preconsolidation conditions can be defined by either the Over-Consolidation Ratio, OCR, or the Pre-Overburden Pressure, POP (Brinkgreve et al., 2010). For this thesis, the preconsolidation conditions were incorporated using the POP method. The POP value was initially defined equal to the preconsolidation pressure and later refined in the
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model. According to Waterman and Broere (2004), the modified compression index and the modified swelling index ($\lambda^*$, $\kappa^*$) can also be determined from $\varepsilon - \ln \sigma_v'$ curves, and the modified creep index ($\mu^*$) can be measured from the $\varepsilon - \ln t$ curves. For Ballydermot peat, the $\varepsilon - \ln \sigma_v'$ curves are presented in Figure 3.33. The $\varepsilon - \ln t$ curves for all the oedometer tests are shown in Appendix A, Figure A.66 to A.74.

The modified creep index values ($\mu^*$) estimated using the EOP approach parameters are shown in in Table A.1 to Table A.9, along with the values determined from the $\varepsilon - \ln t$ curves. The latter values are plotted in Figure 3.34 versus the vertical effective stress to show the variation of $\mu^*$ against $\sigma_v'$. When Figure 3.34 is compared with Figure 3.27 ($C_\alpha$ vs $\sigma_v'$), the resemblance between the figures can be noticed, as it is expected since both are creep parameters.

The initial permeability values were calculated based on equation (2.36), using the $C_k$ and initial void ratio determined in the oedometer tests. The horizontal and vertical permeability were assumed to be equal for all models. The shear strength of Ballydermot peat was extensively investigated by Hebib (2001), who reported effective angles of friction of $\phi'=55^\circ$ when determined in the drained triaxial compression test, and $\phi'=21^\circ$ when determined in the ring shear test. Due to this difference, it was decided to use shear strength parameters on the low side as $c'=1$ kPa and $\phi'=30^\circ$. These values were used for all the models presented here. The at rest lateral earth pressure coefficient, $k_s$, was allowed to be calculated automatically by PLAXIS, according the method described in the Reference manual (Brinkgreve et al., 2010), which is based in the POP value. The initial void ratio, $e_o$, and the unit weight, $\gamma_h$, were taken from the laboratory results (Table 3.5).

The refined model parameters, for both SS and SSC models, are shown in Table 3.11 and Table 3.12, respectively. The comparison figures for all tests are shown in Appendix A, Figure A.75 to Figure A.83. As can be seen from the final value of design parameters ($\lambda^*$, $\kappa^*$ and $\mu^*$), a good initial approximation is provided by equations (2.32), (2.33) and (2.35) using the $C_l$, $C_N$ and $C_{\varphi}$. 
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Figure 3.33: $\varepsilon - \log \sigma'$ curves for Ballydermot peat at different depths.

Figure 3.34: Variation of the modified creep index, $\mu^*$, with the vertical effective stress.
### Table 3.11: Soft Soil model parameters.

<table>
<thead>
<tr>
<th>PISTON</th>
<th>TEST</th>
<th>POP</th>
<th>$A^*$</th>
<th>$k^*$</th>
<th>$\lambda^<em>\mu^</em>$</th>
<th>$\gamma_s$ (kN/m$^3$)</th>
<th>$c_v$</th>
<th>$k_s$ (m/day)</th>
<th>$k_a$ (m/day)</th>
<th>$C_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-1</td>
<td>Oedometer 1</td>
<td>10</td>
<td>0.125</td>
<td>0.051</td>
<td>2.45</td>
<td>10.32</td>
<td>8.69</td>
<td>0.128</td>
<td>0.128</td>
<td>1.8</td>
</tr>
<tr>
<td></td>
<td>Oedometer 2</td>
<td>10</td>
<td>0.11</td>
<td>0.04</td>
<td>2.75</td>
<td>10.56</td>
<td>6.78</td>
<td>0.011</td>
<td>0.011</td>
<td>1.8</td>
</tr>
<tr>
<td>0.90 – 1.55 m</td>
<td>Oedometer 4</td>
<td>10</td>
<td>0.12</td>
<td>0.048</td>
<td>2.5</td>
<td>10.48</td>
<td>6.81</td>
<td>0.012</td>
<td>0.012</td>
<td>1.8</td>
</tr>
<tr>
<td></td>
<td>Oedometer 1</td>
<td>6.5</td>
<td>0.142</td>
<td>0.055</td>
<td>2.58</td>
<td>9.96</td>
<td>14.50</td>
<td>0.011</td>
<td>0.011</td>
<td>3.44</td>
</tr>
<tr>
<td></td>
<td>Oedometer 2</td>
<td>6.3</td>
<td>0.166</td>
<td>0.056</td>
<td>2.96</td>
<td>10.12</td>
<td>13.50</td>
<td>0.0056</td>
<td>0.0056</td>
<td>3.44</td>
</tr>
<tr>
<td></td>
<td>Oedometer 4</td>
<td>6.1</td>
<td>0.155</td>
<td>0.055</td>
<td>2.82</td>
<td>10.24</td>
<td>13.43</td>
<td>0.0053</td>
<td>0.0053</td>
<td>3.44</td>
</tr>
<tr>
<td></td>
<td>Oedometer 1</td>
<td>6.5</td>
<td>0.125</td>
<td>0.051</td>
<td>2.45</td>
<td>10.32</td>
<td>8.69</td>
<td>0.128</td>
<td>0.128</td>
<td>1.8</td>
</tr>
<tr>
<td></td>
<td>Oedometer 2</td>
<td>6.3</td>
<td>0.166</td>
<td>0.055</td>
<td>2.5</td>
<td>10.56</td>
<td>6.78</td>
<td>0.011</td>
<td>0.011</td>
<td>1.8</td>
</tr>
<tr>
<td></td>
<td>Oedometer 4</td>
<td>6.1</td>
<td>0.155</td>
<td>0.055</td>
<td>2.82</td>
<td>10.24</td>
<td>13.43</td>
<td>0.0053</td>
<td>0.0053</td>
<td>3.44</td>
</tr>
<tr>
<td></td>
<td>Oedometer 1</td>
<td>11</td>
<td>0.2</td>
<td>0.051</td>
<td>3.92</td>
<td>9.57</td>
<td>12.50</td>
<td>0.0029</td>
<td>0.0029</td>
<td>3.44</td>
</tr>
<tr>
<td></td>
<td>Oedometer 2</td>
<td>7</td>
<td>0.166</td>
<td>0.043</td>
<td>3.86</td>
<td>10.17</td>
<td>11.50</td>
<td>0.0015</td>
<td>0.0015</td>
<td>3.44</td>
</tr>
<tr>
<td></td>
<td>Oedometer 4</td>
<td>9.5</td>
<td>0.19</td>
<td>0.04</td>
<td>4.75</td>
<td>10.46</td>
<td>12.15</td>
<td>0.0023</td>
<td>0.0023</td>
<td>3.44</td>
</tr>
</tbody>
</table>

### Table 3.12: Soft Soil Creep model parameters.

<table>
<thead>
<tr>
<th>PISTON</th>
<th>TEST</th>
<th>POP</th>
<th>$A^*$</th>
<th>$k^*$</th>
<th>$\lambda^<em>\mu^</em>$</th>
<th>$(\lambda^<em>\mu^</em>)/\mu^*$</th>
<th>$\gamma_s$ (kN/m$^3$)</th>
<th>$c_v$</th>
<th>$k_s$ (m/day)</th>
<th>$k_a$ (m/day)</th>
<th>$C_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-1</td>
<td>Oedometer 1</td>
<td>10</td>
<td>0.11</td>
<td>0.022</td>
<td>0.008</td>
<td>5</td>
<td>11</td>
<td>10.32</td>
<td>8.69</td>
<td>0.128</td>
<td>1.8</td>
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<tr>
<td></td>
<td>Oedometer 2</td>
<td>10</td>
<td>0.115</td>
<td>0.042</td>
<td>0.0065</td>
<td>2.74</td>
<td>11.23</td>
<td>10.56</td>
<td>6.78</td>
<td>0.011</td>
<td>3.44</td>
</tr>
<tr>
<td>0.90 – 1.55 m</td>
<td>Oedometer 4</td>
<td>10</td>
<td>0.125</td>
<td>0.05</td>
<td>0.0065</td>
<td>2.5</td>
<td>11.54</td>
<td>10.48</td>
<td>6.81</td>
<td>0.012</td>
<td>3.44</td>
</tr>
<tr>
<td></td>
<td>Oedometer 1</td>
<td>5</td>
<td>0.142</td>
<td>0.055</td>
<td>0.0078</td>
<td>2.58</td>
<td>11.15</td>
<td>9.96</td>
<td>14.50</td>
<td>0.011</td>
<td>3.44</td>
</tr>
<tr>
<td></td>
<td>Oedometer 2</td>
<td>6.3</td>
<td>0.14</td>
<td>0.03</td>
<td>0.01</td>
<td>4.67</td>
<td>11</td>
<td>10.12</td>
<td>13.50</td>
<td>0.0056</td>
<td>3.44</td>
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<tr>
<td></td>
<td>Oedometer 4</td>
<td>5</td>
<td>0.16</td>
<td>0.055</td>
<td>0.0078</td>
<td>2.91</td>
<td>13.46</td>
<td>10.24</td>
<td>13.43</td>
<td>0.0053</td>
<td>3.44</td>
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<tr>
<td></td>
<td>Oedometer 1</td>
<td>11</td>
<td>0.16</td>
<td>0.033</td>
<td>0.009</td>
<td>4.85</td>
<td>14.11</td>
<td>9.57</td>
<td>12.50</td>
<td>0.0029</td>
<td>3.44</td>
</tr>
<tr>
<td></td>
<td>Oedometer 2</td>
<td>6</td>
<td>0.166</td>
<td>0.043</td>
<td>0.0065</td>
<td>3.86</td>
<td>18.92</td>
<td>10.17</td>
<td>11.50</td>
<td>0.0015</td>
<td>3.44</td>
</tr>
<tr>
<td></td>
<td>Oedometer 4</td>
<td>9</td>
<td>0.2</td>
<td>0.04</td>
<td>0.009</td>
<td>5</td>
<td>17.78</td>
<td>10.46</td>
<td>12.15</td>
<td>0.0023</td>
<td>3.44</td>
</tr>
</tbody>
</table>
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From the figures presented in Appendix A, it can be seen that both models adequately described the stress – strain behaviour of Ballydermot peat. The modified compression index values ($A^*$) for both models reflect the difference between the compressibility of the upper and lower deposits. For the surficial peat, $A^*$ ranged between 0.11 and 0.125 in both models, while for the deeper deposit it varied from 0.14 and 0.20. The refined value of the POP yielded very similar results as those of the preconsolidation pressure determined with the EOP approach (Table 3.7).

With respect to the creep behaviour, the SSC model correctly described the secondary compression of the peat observed during entire duration of each oedeometer tests. In contrast, the SS model was able to determine the final vertical strain value for each increment, but without following the strain –time behaviour shown by the peat.

3.5 SUMMARY AND DISCUSSION

Ballydermot bog is a cutaway bog were extensive peat extraction has taken place for the last 60 year. As part of the process of peat harvesting, Bord na Móna built drainage channels along the bog in order to reduce the water table.

Due to the extensive drainage, the peat layer at Ballydermot bog has undergone a continuous settlement process over the last 60 to 70 year. The peat depth in 1935 was measured to be 8.7 m, and to date approximately only 3.2 to 4.0 m remains.

Based on rainfall and water table measurement, a seasonal variation of the water table level was observed in the studied area; with the lowest points found during the summer time, around the mid-June, and the highest levels reached around mid-July, early-September and late-October, coinciding with particularly rainy periods with some high intensities. Next to the selected test area, the presence of a ditch induces a variation of the water table levels, with the lowest levels found closer to the ditch.

Based on the field and the laboratory investigation, the soil profile of the test area at Ballydemort bog is mainly composed of three layers: (i) a 0.8 m thick man-made fill, (ii) a
3.2 m pseudo-fibrous peat layer, and (iii) a 1.9 m gravel layer with very high contents of fines and sand, reducing with depth.

The vane shear tests results were very variable. However, in general, the shear strength of the peat increased towards the bottom of the deposit and advancement become very hard in the last 0.5 to 0.6 m. This is in good agreement with what was observed during the manual boreholes, where at the bottom of one of the boreholes even small amounts of mineral soils were recovered mixed with the peat. According to Hobbs (1986), this mixture can be seen in fen peat (particularly reed peat), where the peat contains varying amounts of clays and silt.

Based on the laboratory investigation, Ballydermot peat can be described as: pseudo-fibrous peat with a degree of humification of H₄ to H₇, a water content of 650 and 1350%, and organic content of 93 to 98%, a fibre content of 40 to 64%, a specific gravity of 1.47, and a pH of 4.6 to 5.6, and a main botanical composition of Sphagnum – Carex – Cladium – Alnus – Betula – Phragmited peat.

The water content determination in peat and organic soils have been studied by several workers (§3.4.1.2). Different concepts in regards of the optimum temperature have been given. Some authors such as Skempton and Petley (1970) have concluded that little advantage is gained by adopting temperatures lower than 105°C, while others like MacFarlanen and Allen (1964) have stated that the temperature of drying should never exceed 95°C and that preferably the temperature should not exceed 85°C.

A statistical study conducted by the author, of the water contents results from Ballydermot peat samples, dried at 80°C and 105°C, has shown that there is no significant difference on the water contents obtained under either temperature, and that the differences can be attributed to chance variation. From these results can then be concluded that for engineering purposes a temperature of 105°C is acceptable.

Based on the LOI tests, Ballydermot peat is highly organic with values up to 98% along the entire deposit, with the organic content reducing drastically in the peat-gravel interface. These results are in good agreement with the water content, specific gravity and pH values.
Chapter 3 DESCRIPTION OF TEST AREA

The specific gravity average value of 1.47 is in accordance with the 1.4 to 1.5 specific gravity of the cellulose and lignin components of the peat (Hobbs, 1986). The pH average value around 5.0 indicates the acidic nature of the raised bog, and the water contents between 650 and 1350% are typical of cutaway drained bogs in Ireland.

Furthermore, the variation of the water content, LOI, Gs and pH values is consistent between all the properties along the entire profile (Figure 3.21). The water content and LOI are slightly reduced close to the top near the man-made fill, while the pH increases to close to 7, with no variation for Gs. Along the peat layer, all the values remained around the average ranges given above. When the peat-gravel interface is reached, the water content and LOI values are significantly reduced, and the specific gravity and pH are drastically increased.

The compressibility of Ballydermot peat was investigated by mean of nine standard oedometer tests, six of which were incremental tests and three were long term single increment tests. These tests were modelled using four different theories, namely: the End of Primary approach, Janbu’s resistance concept and the numerical models Soft Soil and Soft Soil Creep, included in the PLAXIS finite element package.

From the tests results, two distinct layers were identified. In the surficial layer, down to a depth of approximately 1.50 m, the initial void ratio was found to be between 6.8 and 8.7. For the deeper deposits, the initial void ratio ranged from 11.5 to 14.5. Despite the initial void ratio value, most specimens tended to converge to a final void ratio value between 3.5 and 4.5, at an effective stress of 200 kPa.

The EOP approach adequately modelled the stress – strain behaviour of Ballydermot peat. Using this method, the preconsolidation pressure was estimated between 8 and 11 kPa for all depths. The preconsolidation pressure of Ballydermot bog is quite high when compared with the preconsolidation pressure of other bogs in the Irish midlands (3 kPa for Clara bog and 5 kPa for Raheenmore bog). These high values can be explained due to the consolidation process induced by the drainage of the bog for harvesting purposes for the last 60 years. According to Hobbs (1986), drainage of bogs will increase the preconsolidation pressure, the extent depending on the draw down level.
Chapter 3 DESCRIPTION OF TEST AREA

The difference found in the void ratio value was also reflected in the compression index, while for the deeper peat $C_c$ ranges between 4.79 and 6.13, for the samples closer to the surface is 2.15 in average. Furthermore, the secondary compression index also shows this difference; the $C_{ca}$ of the surface samples ranges from 0.039 to 0.140, and for the deeper ones is between 0.059 and 0.390.

The fact that the selected test area serves as a transit zone for the heavy machinery from Bord na Móna, to enter and exit the bog, could explain the difference between the surficial and deeper deposits. The heavy traffic could have induced a higher void ratio reduction in the layers closer to the surface, while the man-made fill could have acted as a semi-permeable layer in the process.

The permeability of Ballydermot peat was computed from the oedometer tests results. The coefficient of permeability was found to follow a linear relationship in the semi-logarithmic space $e - \log k$. This behaviour has also been observed by Tavenas et al. (1983b) and Mesri et al. (1997). The $C_k$ values determined using both Casagrande and Taylor graphical constructions were very similar between them. For the deeper deposits, the $C_k = 3.44$ using Casagrande construction and $C_k = 3.71$ with the Taylor construction. For the surface ones, $C_k = 1.80$ with the first method and $C_k = 1.79$ with the second. From these results, the difference between the surface and the deep deposits is evidenced again. Mesri et al. (1997) suggested that $C_k = 0.25e_o$ for peat, which is in good agreement with the results presented here. Even though the surface deposits have a lower initial void ratio than the deeper ones, the permeability of the former materials is slightly higher than that of the latter.

Janbu’s resistance concept adequately described the stress – strain behaviour of all tests, and the strain – time behaviour of most load increments. However, the long term tests (10 days, 14 days and 62 days) did not behave linearly as expected, but after certain time the linearity of the curve was distorted showing scatter behaviour. This behaviour may be explained by the tertiary compression observed in peat soil, due to the decay of the organic matter, however further research is needed.
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The preconsolidation pressure was calculated using the stress – strain resistance concept and the strain – time concept. The first method yielded results closer to the ones obtained using the EOP approach, between 8 and 13 kPa; while for the second method it varied from 12 to 28 kPa. The difference in the value between the two methods could be because of the load increments selected. As the preconsolidation pressure is very low, the curves where not detailed enough around \( \sigma_p \) making it hard to defined.

The creep resistance number, \( r_* \), of peat seems to be dependent on the stress level as it does in clay.

The Soft Soil and the Soft Soil Creep models correctly depicted the stress – strain behaviour shown by Ballydermot peat on the oedometer tests. The strain – time behaviour was adequately described by the SSC model, but not by the SS model. However this last one was able to determine the final vertical strain value.

The modified compression index values (\( \lambda^* \)) of both models also showed the difference between the surficial and deeper materials. For the upper peat, \( \lambda^* \) ranged between 0.11 and 0.125, while for the deeper deposit it varied from 0.14 and 0.20. The POP values determined were very similar to the preconsolidation pressure determined with the EOP approach, and with the Janbu’s stress – strain resistance concept. The POP varied from 6.1 to 11 kPa for both methods.
4. TCD/NRA VACUUM CONSOLIDATION FIELD TEST

4.1 INTRODUCTION

Vacuum consolidation has become increasingly popular as a ground improvement technique since the 1980’s, particularly in Asia, with an estimated treated area of over 6 million m² worldwide (Yee and Ooi, 2010). However, most of the research and practical applications have been conducted in the improvement of highly compressible clayey soils and hydraulic fills for land reclamation projects. Table 2.1 presented a compilation of several vacuum consolidation projects found in the literature, from which it can be seen that vacuum consolidation has not been used extensively in peat soils.

This chapter initially describes the design, implementation and monitoring of the field test at Ballydermot bog. Subsequently, the field results are discussed, and a finite element model is used to back-analyse the test, using the parameters determined in the laboratory. The field and model results are compared, and the ability of the model to predict the field behaviour is assessed. Finally, the technical difficulties encountered while implementing the test are described and an analysis of its effect in the project is given.

4.2 TEST DESCRIPTION

The TCD/NRA vacuum consolidation field test was conducted in a 10 x 10 m area at Ballydermot bog. In short, the field test can be divided into three main stages: (i) monitoring of initial conditions, test construction and baseline setup, starting on the 12th March 2009; (ii) vacuum pumping stage, staring on the 30th November 2009; and (iii) rebound stage, starting on the 29th October 2010 and finishing on the 12th April 2011. The chronology for the 769 days of field works is shown in Figure 4.1.
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TCD/NRA VACUUM CONSOLIDATION FIELD TEST

Initial monitoring

Manual boreholes

Construction and instrumentation

Vacuum pumping (period 1)

Field vane tests

Mechanical boreholes

Change of pumping system

Vacuum pumping (period 2)

Rebound

Figure 4.1: Chronology of field works.
4.2.1 Test construction

(i) Preparation of the surface: Since the upper 0.8 m of the ground comprised a man–made fill or reworked peat (see §3.2.4), 0.4 m of this fill was removed from a 12 x 12 m, on the 18th June 2009, using a peat digger and the area was subsequently levelled (Figure 4.2).

Figure 4.2: Surface preparation: (a) fill removal, (b) levelled area.
(ii) Prefabricated vertical drain and subsurface instrumentation installation: Ninety-eight prefabricated vertical drains were pushed down vertically into the peat to a depth of 2.65 m with the assistance of the bucket of a bog digger, using an aluminium box section as the lance (Figure 4.3), on the 29th and 30th of June 2009. This left approximately 1.35 m of peat between the PVD bottom end and the peat–gravel interface to prevent the escape of the vacuum pressure (see §2.2.3.1). A cross-section of the test setup is shown in Figure 4.4(a).

Figure 4.3: Prefabricated vertical drain installation: (a) pushing down process, and (b) final arrangement.
According to Hayashi et al. (2002), when using prefabricated vertical drains, no improvement effects is achieved unless the drain spacing is 0.9 m or smaller (see §2.2.4.4). In order to evaluate how the difference in the spacing of the PVDs affects the improvement method, the test area was subdivided in two, one in which the spacing of the drains was 0.85 m and a second one with spacing of 1.2 m, both in a square grid (Figure 4.4(b)). The subsurface instrumentation system, which was pushed down on the 9th of July 2009 after the PVDs, will be discussed in detail in §4.2.2.

Figure 4.4: TCD/NRA vacuum consolidation field test: (a) cross-section, and (b) plan view.
(iii) Granular bed and horizontal pipe system: A granular bed was placed on the ground surface, between the 21\textsuperscript{st} and 22\textsuperscript{nd} of October 2009, comprised by a 0.15 m layer of sand overlaid by a 0.30 m layer of gravel. In total, 90 tons of sand and gravel were used, equivalent to a pressure on the ground surface of 8.83 kPa.

\textbf{Figure 4.5:} Granular bed installation: (a) sand, and (b) gravel.
A system of perforated flexible pipes, 76.2 mm in diameter, was embedded horizontally within the gravel layer (Figure 4.6). The granular bed and the horizontal drains have the functions of transmitting the vacuum to the soil as well as discharging water and air out of the treated area (see §2.2.3.1).
(iv) Trench excavation and airtight plastic liner installation: From the monitoring performed before the pumping stage, a maximum ground water table depth of 0.90 m was recorded on the pipes around the test area and of approximately 1.0 m on the stand pipes next to the ditch (Figure 3.18(a)). Based on this, between the 4th and 5th of November 2009, a trench, 0.3 m wide and 1.0 m deep, was dug around the 10 x 10 m test area and an airtight 1200 gauge polythene membrane was laid over the entire area. The membrane was keyed at the bottom of the trench by backfilling the trench and covering the top with peat in order to help maintain the seal and protect the membrane from weathering and animal attacks (Figure 4.7).

Figure 4.7: (a) Trench around test area, (b) airtight polythene membrane.
Before the peat cover was placed, the membrane was carefully inspected for punctures or tears that might affect the vacuum generation and when found these were covered with butyl sealant tape. However, two holes needed to be cut on the membrane; one for the vacuum line connection and a second one for a pipe through which all the subsurface instruments cables were brought out (Figure 4.7(b)). After the pipes were installed, these were carefully attached to the membrane using butyl tape, and the cables pipe was filled using expanding sealant foam to prevent any vacuum loss.

4.2.2 Instrumentation

The instrumentation system was designed to measure settlement at different depths, positive and negative pore water pressure at different depths, barometric pressure, surface and ground temperatures, water flow, water table and rainfall. The system can be divided into surface and subsurface systems. The surface instruments include settlement plates, stand pipes, a barometer/thermometer and a rain gauge; while the subsurface instruments include vibrating wire settlement cells and vibrating wire piezometers.

4.2.2.1 Subsurface instrumentation

The subsurface instrumentation includes 6 push-in Vibrating Wire Settlement Cells (Figure 4.8(a)) and 10 push-in Vibrating Wire Piezometers (Figure 4.8(b)), calibrated for both positive and negative pressure. Since the treated area was subdivided in two different sections, three settlement cells and four piezometers were pushed in at the centre of each subarea at different depths, to perform independent monitoring of settlement and pore pressures of each section. One of the remaining piezometers was located at the inner edge of the treated area, in the 0.85 m spacing section, to investigate the boundary effects; while the second one was located 5.0 m outside the test area to investigate the effect of the vacuum outside the studied area.

The settlement cells were pushed down to 0.90, 1.50 and 2.65 m in both tests areas. The first three piezometers in both areas were pushed down to the same depths; however, the bottom ones reached different depths due to hardening of the peat–gravel interface at the
bottom of the deposit (see §3.5). For the 0.85 m spacing area, the bottom piezometer was located at a depth of 4.05 m, while for the 1.20 m spacing the piezometer reached 3.72 m.

Due to the soft nature of the peat, the subsurface instruments were pushed into the ground by hand (Figure 4.9(a)). Verticality was guaranteed by using a spirit level (Figure 4.9(b)). All instruments installations were carried according the recommendations provided by the instruction manuals (MGS Geosense, 2009a; 2009b), and assisted by personnel from MGS Geosense and NVM Ltd. Ireland.

The piezometers were located in the centre of the square area formed by four PVDs, in order to measure the average pore water pressure variation induced. Each settlement cell was located next to a piezometer with the purpose of pairing the instruments according to depth. The location and depth of the subsurface instruments is shown in Figure 4.4(b).
The VW settlement cells monitor the settlement or heave, by measuring any changes in the elevation between a reservoir (Figure 4.10(a)) and the cell. The reservoir was mounted on a weather proof instrument box, at a higher elevation relative to the cells. As the cells settle, the liquid pressure at the settlement cell increases and the increase is measured by the vibrating wire sensor. This system required that the instrument box, where the reservoir was mounted, did not settle during the entire test. To achieve this, two steel bars, six meters long each, were driven down to the gravel strata using the same system as for the PVDs (Figure 4.10(b)). Once the bars were fixed in the ground, a wood board was attached to them and two weather proof boxes were attached to either side (Figure 4.10(c)). One of the boxed held the reservoir (Figure 4.10(d)), while the second box held the instruments board (Figure 4.10(e)). Afterwards, the instrument boxes were covered by building a small shed around them (Figure 4.10(f)). The barometer/thermometer was fixed to the frame outside the boxes (Figure 4.11(a)).

Readings from each instrument were taken by connecting a vibrating wire readout unit (Figure 4.11(b)) to the corresponding plug in the instruments board. Ambient temperature, barometric pressure, and settlement and ground temperature at a depth of 0.90 m, in the 0.85 m spacing area, were recorder hourly by connecting the instruments to a data logger. This data logger was used until 2nd July 2010. From this date onwards, a new data logger (Figure 4.12) with additional channels was introduced; thus the pore water pressure at a
depth of 0.90 m for both spacing areas, the ground temperature at the 1.20 m spacing area and the vacuum pressure at the bog surface, where also recorded hourly in addition to what was being recorded before.

Figure 4.10: Instrument boxes construction process.
4.2.2.2 Surface instrumentation

Due to the construction of the granular bed, two different arrangements of surface settlement monitoring points were used (Figure 4.13). The first one was located prior to the granular bed construction, which was lost during the building process, and was used to monitor ground movements up to the 21st October 2009, before the vacuum was applied beneath the membrane. The second array was placed after the granular bed was built, and it was used to monitor during the pumping process.
Figure 4.13: Surface instrumentation systems: (a) before pumping, (b) after pumping.
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The first surface instrumentation system (Figure 4.13(a)) comprises 23 surface settlement pegs, nine of which were located inside the test area, 14 stand pipes to monitor the ground water table level, one of each located next to one of the pegs outside the test area; one surface barometer/thermometer (Figure 4.11(a)), and a rain gauge placed to record precipitation and to correlate it with the variations on the water table. The rain gauge and stand pipes are the same as described in §3.3. Although the pegs in the test area were covered by the embankment, the settlement of their original location was estimated by taking surface levels at close spacing and interpolating the surface level, to Site Datum, at the location of each point and subtracting 0.44 m, which was the average thickness of the granular bed.

The second surface instrumentation system (Figure 4.13(b)) includes 26 surface settlement plates, nine of which were located inside the test area, 17 stand pipes located next to the remaining settlement plates outside the treated area; a surface barometer/thermometer, a rain gauge and a water meter placed at the end of the water discharge pipe to measure the amount of water expelled from the peat deposit and correlate it with the settlement measures (see §4.2.3.1).

Two Bench Marks were set for the project in order to reference all measurement to them. Bench Mark A (BMA) was used as Site Datum with coordinates (0, 0, 100); while Bench Mark B (BMB) was used as a reference for the calculations with the purposes of aligning all the surveys in the X–Y plane. The Bench Marks were set outside the bog, on a concrete pavement, in Bord na Móna grounds (Figure 4.14). The coordinates of all the points shown in Figure 4.13 are referenced to Site Datum. Figure 4.15 shows the location of the subsurface instrumentation and of the settlement plates inside the test area, related to S.D.

All surveys on site were carried out using a Total Station NTS–302B (Figure 4.16(a)). In the first surface array, all pegs had a screw attached on the top, and a prism target (Figure 4.14) was placed on it for each reading. This system required the assistance of a second person on site. In order to make the data collection process a one person operation, the settlement plates were built with a concrete base, a metallic bar and a stick-on minitarget compatible with the total station, and were then buried just under the bog surface (Figure
4.16(b)). A minitarget was also attached to the instruments box with the purpose of monitoring if the box underwent any settlement (Figure 4.11(a)).

Figure 4.14: Bench Marks set for the project.

Figure 4.15: Surface and subsurface instruments location, related to S.D.
4.2.3 Vacuum generating systems

Two different vacuum generating systems were used during the project in order to evaluate which produced better results in peat deposits.

4.2.3.1 Period 1: 30th November 2009 to 29th June 2010

The first vacuum generating system (Figure 4.17) is comprised by a 38 mm jet pump with a 1.5 kW centrifugal water pump, which can generate a vacuum pressure up to 95 kPa. This system was used due to the versatility provided by the jet pump to handle the air-water mixture pumped out of the peat deposit (see §2.2.3.1).

The system works with the venturi principle using the centrifugal pump to shoot water under pressure through the jet pump. The jet pump is in turn connected to the vacuum line, which is connected to the pipework in the granular bed beneath the membrane, thus transmitting the vacuum directly to the granular bed at the bog surface (Figure 4.17(a)). The water used is stored in the main tank and recirculated through the system, reducing water waste to a minimum. The water pumped out the peat is also deposited in the main tank and once this is filled, the excess water passes to the overflow tank from where it is discharged to the municipal sewage system.

Figure 4.16: Survey equipment: (a) Total station, (b) Settlement plate with stick-on minitarget.
The vacuum generating system was also instrumented. Two vacuum gauges were installed, one at the jet pump (Figure 4.18(a)) and another at the end of the vacuum line before going into the granular bed (Figure 4.18(b)), with the purpose of assessing if there was any vacuum loss in the vacuum line. At the discharge line in the overflow tank, a water meter was installed (Figure 4.19), in order to register the amount of water extracted from the peat deposit, and to correlate this measure with the settlement achieved by the technique.

4.2.3.2 Period 2: 29th July 2010 to 29th October 2010

After Period 1, the vacuum generating system was modified by introducing a liquid ring vacuum pump as the vacuum generating device, with a vacuum level up to 94 kPa.
Different pump sizes were tried (0.75 kW, 1.5 kW and 2.2 kW) for a month, and finally the 2.2 kW was chosen. The pump used was a Busch Dolphin LX 0110 A.

![Image](image-url)

**Figure 4.18:** (a) Jet pump and vacuum gauge, (b) Vacuum gauge in the vacuum line.

![Image](image-url)

**Figure 4.19:** Water meter at the discharge line.

The principle of operation of liquid ring pumps is presented in Figure 4.20. Liquid ring vacuum pumps normally function with water as the operating medium. An eccentrically installed impeller (2) rotates in the casing (1) partly filled with liquid. By the rotational movement of the impeller and the resulting centrifugal force the liquid within the cylinder forms the so called liquid ring (3). Gas is conveyed in the spaces between the fixed vanes and the liquid ring. Due to the eccentric installation of the impeller the liquid ring moves out and the process gas is sucked in through the suction slot (4). Further rotation causes the liquid ring to move in so that the gas is compressed and discharged through the discharge slot (5).
Even though liquid ring pump uses water as a lubricating medium, the volume required is fixed and should be provided constantly. If the volume is decreased or increased, the efficiency of the pump is reduced, and its integrity might be compromised. When the volume is increased significantly, the forces in the impeller and the electrical load are also increased, thus forcing the circuit breaker to trip.

Taking this into consideration, along with the amount of water being extracted from the bog, the author designed a vacuum chamber for separating the air–water mixture being pumped. The vacuum chamber was built at the Mechanical Workshop of the Department of Civil, Structural & Environmental Engineering, in Trinity College Dublin, from a 500 x 300 mm PVC pipe sealed at the top and bottom (Figure 4.21(a)). Three lines were placed at the top of the chamber; one for connecting the vacuum line to the bog surface reaching just under the top cover of the chamber, one for the liquid ring pump reaching the centre of the chamber, and the last one for the jet pump reaching the bottom of the vacuum chamber (Figure 4.21(b)).
Figure 4.22 shows the setup adopted for the vacuum generating system in period 2. The setup works with the liquid ring as the main vacuum generating device. The liquid ring is connected to the vacuum chamber, creating a vacuum in it. The vacuum is in turn transmitted to the pipework in the granular bed beneath the membrane by the vacuum line. The water that is extracted from the bog is collected in the vacuum chamber. The different lengths of the pipes in the vacuum chamber are used for the air–water mixture separation. The water is deposited at the bottom of the chamber by the vacuum line and is evacuated by the centrifugal pump – jet pump system. The liquid ring pump line in the chamber was left at the centre of the chamber to prevent any significant amount of water from being sucked into the pump (Figure 4.22(a)).

The water used to feed both, the centrifugal and liquid ring pumps, is stored in the main tank and any excess water pumped out of the bog goes into the overflow tank, in the same manner as described for the previous system.

The instrumentation used for this vacuum generation system is the same as the one used for the system described above, but a VW pressure meter was added at the end of the vacuum line before going into the granular bed, providing hourly readings of the vacuum pressure at the bog surface (see §4.2.2.1).
Figure 4.22: Vacuum generating system, period 2: (a) schematic, (b) site setup.

4.3 TEST MONITORING AND RESULTS

The test monitoring at Ballydermot bog started on the 10th June 2009 after the installation of the subsurface instrumentation, and was conducted until the 12th of April 2011. Initially,
the instruments were monitored in order to review their behaviour, while at the same time surveys were being conducted. After analysing the initial data, and comparing the settlement readings with the results from the surveys, it was decided to set the 18th August 2009 as zero for all settlement readings in the project. The settlement at different depths for the entire test duration is shown in Figure 4.23, along with the readings for the instruments box and BMB. It is important to note that the settlement values are indicated by the negative numbers, while heave is given in positive numbers.

In parallel, the rainfall and water table variations on site were also monitored. The rainfall readings before the vacuum pumping started were shown in Figure 3.15, in the previous chapter. Figure 4.24 presents the daily rainfall measured at Ballydermot bog during and after pumping. The data marked as Ballydermot (red) was recorded directly by the rain gauge installed on site, and the data marked as Edenderry (blue) was provided by Met Éireann, from the records at the closest weather observing station from the site.

In spite of having changed the surface instrumentation array after the construction of the granular bed (see §4.2.2.2 and Figure 4.13), some stand pipes were undamaged during
construction, and were used through the entire project. This allowed continuous monitoring of the water table variation at some points. Figure 4.25 presents the water table variation at these points, along with the corresponding surface levels, with all measures related to S.D.

Figure 4.24: Daily rainfall measurement during and after vacuum pumping.

Figure 4.25: Water table variation during the entire project.
Figure 4.26 shows the surface and ground temperatures, at a depth of 0.90 m for both PVD spacing section, for Ballydermot bog during and after vacuum pumping.

The water table variation for the entire project shows that there is a seasonal variation, as it was described in §3.3.5, with the levels dropping around mid-March and reaching the lowest point in the summer around mid-June to early July. The water table variation is also dependent on the rainfall, as expected. As an example, the most intensive rainfall during the project occurred on the 16th June 2010 with a total of 46 mm for the day (Figure 4.24), followed by an immediate increase in the water table level at all stand pipes (Figure 4.25).

As described in the previous chapter, the water table level is at the lowest point near the ditch and higher closer to the test area. During the summer of 2010, the water table near the test area dropped to approximately 0.9 m for plate 21 reaching the same value as during the summer of 2009; while near the ditch (plates 13 and 14) the level drop to 1.2 m, approximately 0.2 m below the level of 2009. From this results, it can be initially said that the vacuum had no effect in water table levels outside the test are, however a more detailed analysis is given in §4.3.2.
For the following analysis, the results are separated into two different phases. The first phase covers the period prior to the start of the vacuum pumping, from 10th June 2009 to 30th November 2009. The second phase covers the pumping and rebound monitoring, from 30th November 2009 to 12th April 2011.

4.3.1 Monitoring before pumping

Figure 4.27 shows the variation on the water table around the test area, along with the corresponding surface level, related to S.D, from 23rd September 2009. The ground vertical deformations at different depths for both spacing subareas is presented in Figure 4.28, while Figure 4.29 presents the pore water pressures recorded at different depths in both subareas.

![Diagram of water table levels before pumping around the test area](image)

Figure 4.27: Water table levels before pumping around the test area.

Figure 4.28 shows a heave on the ground at all depths (0.14 m at the surface) from 18th August 2009 to 12th September 2009. The heave can be directly related to the increase on the water table levels and rainfall on the same period (see §3.3.5). Furthermore, Figure
4.29 also presents an increase on the pore water pressures at all depths inside the testing area, for both spacing sections. The piezometer located outside the testing area, however, depicts a slower response on the increase of the pore water pressure. The faster response inside the testing area can be explained by the change in the hydrogeological behaviour introduced by driving the PVDs in the peat deposit.

Since part of the fill was removed from the surface of the test area, and there was a significant increase in the water table, the level of water rose above surface level, flooding the test area as is shown in Figure 4.30(a) (picture from 3rd September 2009). The water level reduced slowly for a period of approximately three weeks, and on the 23rd of September 2009, the surface was almost dry as can be seen in Figure 4.30(b). The reduction is also reflected by a slight reduction in the PWP readings for both sections in the test area (Figure 4.29).
Figure 4.29: Pore water pressure variation with time, before the pumping stage: (a) for 0.85 m PVD spacing, (b) for 1.20 m PVD spacing
On 21st October 2009 and 22nd October 2009 the granular bed, 0.44 m thick, was built on the surface of the testing area (see §4.2.1). The granular bed introduced a load of 8.8 kPa, inducing a settlement of 0.14 m (Figure 4.28). Piezometers also recorded an increase in the pore water pressures at all depths after the granular bed was constructed (Figure 4.29). Moreover, due to some heavy rainfalls in the days prior to the construction (Figure 3.15), the water table was raised (Figure 3.18(a) and Figure 4.27) and the test area was flooded.
again, though not to the level observed in September (Figure 4.5(a) and Figure 4.31(a)). This situation made the construction of the sand layer difficult, thus a drainage channel had to be open to evacuate the water from the work area (Figure 4.31(b)).

Figure 4.31: Test area: (a) flooded on 21st October 2009, (b) Drained during sand layer construction.
Later, on 4th November 2009 and 5th November 2009, the trench was excavated, the impervious membrane was installed and the peat coverage was placed on top. The instrumentation also shows a reaction to these processes. Figure 4.28 shows a sudden increase in the settlement, for all depths. The pore water pressures registered an instant drop, followed by a fast recovery of the previous level (Figure 4.29).

Figure 4.32 and Figure 4.33 present the PWP and effective stress variation with depth for the 0.85 and 1.20 m PVD spacing subareas, respectively. Figure 4.32(a) and Figure 4.33(a) show the variation of the pore water pressure for selected dates; while Figure 4.32(b) and Figure 4.33(b) depict the variation of effective stress for the same dates, along with the total stress for before, during and after the granular bed construction. To calculate the total and effective stresses, the unit weights used were the average values of the ones obtained from the oedometer tests as follows: (i) for D=0.9–1.55 m, $\gamma_h=10.45$ kN/m$^3$; (ii) for D=1.50–2.30 m, $\gamma_h=10.03$ kN/m$^3$; and (iii) for D=1.50–2.30 m, $\gamma_h=10.07$ kN/m$^3$. The unit weight used for the granular bed was $\gamma_h=19.93$ kN/m$^3$.

As can be seen Figure 4.32(a) and Figure 4.33(a), the pore water pressure increases linearly for the peat deposit; however, at greater depths when reaching the gravel layer, the PWP values reduce. This reduction is due the lower piezometric head at the silty clayey sandy gravel, which induces a flow of water downwards. The difference in the piezometric heads could be explained by the higher permeability found in gravel when compared to the permeability of the peat (between one and two orders higher, §3.3.6 and §3.4.3.3) creating a drainage layer beneath the peat. A more detailed analysis of the piezometric heads is given in §4.3.2.2. It is important to note that the piezometer in the 0.85 m area is located deeper than the piezometer in the 1.20 m spacing area (see §4.2.2.1). Because of this, and according to the approximated soil profiles shown in Figure 3.8, the piezometer from the former is located in the gravel layer, while the piezometer from the latter is at the peat–gravel interface, which explains why the PWP reduction appears to be higher for the 0.85 m. When the PWP values for the peat are compared for the two subareas, no significant difference is noticed.

It is important to note that the PWP value given in Figure 4.32(a), is taken from the piezometer located at the centre of the 0.85 m PVD spacing subarea, and not from the one
located at the edge. However, Figure 4.29(a) shows that there is no difference in the values recorded by both piezometers.

Figure 4.32: Ground pressures variation with depth, before the pumping stage, for selected dates for 0.85 m spacing section: (a) Pore water pressure, (b) Effective and total stresses.
Figure 4.33: Ground pressures variation with depth, before the pumping stage, for selected dates for 1.20 m spacing section: (a) Pore water pressure, (b) Effective and total stresses.
As mentioned in §4.2, the TCD/NRA vacuum consolidation field test commenced on the 30th November 2009. Figure 4.24 shows the daily rainfall and Figure 4.34 the variation of the water table surface levels outside the test area at different points. Figure 4.35(a) presents the vacuum pressure at the bog surface taken by reading a vacuum dial gauge at each site visit, while Figure 4.35(b) shows hourly readings taken by the vibrating wire pressure meter and recorded with a data logger introduced on 2nd July 2010, as described in §4.2.2.1. Some technical difficulties arose during the pumping stage which reduced the pumping pressure, sometimes even fully stopping the process (Figure 4.35). A detailed description of these difficulties, with an analysis of its effect in the project is given in §4.5.

Figure 4.34: Water table levels during and after pumping around the test area.

Figure 4.35(a) shows that vacuum pressure values up to 80 kPa were reached while using the first vacuum generating system (jet pump), however due to the interruptions produced by water freezing in the pipe work these values were not able to be maintained for long periods. However, an average vacuum pressure of 55 kPa was reached during the first month.
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Figure 4.35: Vacuum level at bog surface: (a) Vacuum gauge readings, (b) data logger readings.
Figure 4.35(b) shows that more stable pressures were achieved when using the second vacuum generating system (liquid ring pump), with vacuum pressure ranging between 52 and 71 kPa.

By comparing the water table and surface levels outside the test area (Figure 4.34) with the vacuum pressure, it can be seen that neither was affected by the vacuum pumping as there was no appreciable settlement of the surface and the water table varied with the rainfall rather than with the vacuum variations. Furthermore, Figure 4.35(a) shows a continuous drop in the vacuum pressure after 1st April 2010, after the water table started to fall around 28th March 2010 (Figure 4.34). The fall in the water table is related to a seasonal effect and not to the pumping, as it was also observed in 2009 and it is described in §3.3.5. This shows the importance of the water table outside the test area acting as a seal. A detailed analysis of the vacuum pressure variation due to the water table is given in §4.5.3.

4.3.2.1 Settlement

Figure 4.36 shows the surface settlement for the test area, for selected plates around it, and the vertical displacement of the instruments box. Figure 4.37 presents the ground settlement at different depth. Settlement plates 5 and 2 were selected for indicating the average surface settlement of the 0.85 and 1.20 m PVD spacing section respectively, as these plates are the closest to the settlement cells installed (Figure 4.15).

Figure 4.37 shows that the settlement for both spacing subareas followed each other closely during the entire testing period at all depths. The maximum surface settlement was recorded on 27th October 2010 for both sections. For the 0.85 m PVDs spacing subarea the surface settlement was 1.11 m, while it was 1.09 m for the 1.20 m spacing subarea. However, the maximum surface settlement for the entire project was equal to 1.23 m, measured at plate 4 (Figure 4.36), which with plate 3, are the closest to the point where the vacuum line connects with the pipework inside the granular bed in the centre of the test area. The maximum vertical displacements measured for the instrument box ranged between -0.009 and 0.008 m throughout the test, indicating that the box did not suffer any settlement during the project.
Figure 4.36: Surface settlement for both spacing subareas during and after the pumping stage.

Figure 4.37: Ground settlement at different depths for both spacing subareas during and after the pumping stage.
Figure 4.38 and Figure 4.39 show the vertical ground movement and surface settlement for profile A – A’ and profile B – B’, as shown in Figure 4.13(b), at selected dates. From these figures, it can be seen that the settlement is higher at the centre of the test area and reduces towards the edges. In regards with the ground displacement outside the test area, despite some settlement occurring on the plates just outside it (plates 10, 21 and 25), no significant settlement occurred, with the maximum value reached being 0.1 m in plate 10 (Figure 4.39(b)).

From Figure 4.38 and Figure 4.39 it can be seen that approximately half the settlement of the entire project was induced on the first two weeks, from 30th November 2009 until 15th December 2009, and that 0.8 m of the maximum 1.2 m occurred during the first month; although, the vacuum pressure afterwards was intermittent due to different reasons until the second vacuum generating system was introduced on 29th July 2010.

After the second system was in place, the settlement rate increased again, though not reaching the level of the first month. The settlement from 31st August 2010 to 29th September 2010 is approximately equal to 0.15 m.

It is important to note that, due to the change of the surface instrumentation after building the granular bed, not all the settlement plates correspond to a peg from the previous system, thus only for constructing the surface settlement profiles in Figure 4.38(b) and Figure 4.39(b), zero settlement was assumed on the 30th November 2009. The rest of the thesis uses the 18th August 2009 as described in §4.3.

Figure 4.40 presents the settlement versus depth for both PVD spacing subareas, at selected dates. As it can be seen, before construction a heave of 0.14 m occurred around the 12th September 2009, which remained constant until 22nd October 2009, when it was reverted due to the granular bed construction (see §4.3.1). An interesting characteristic is how approximately 40% of the settlement occurs between 0.9 and 1.9 m, indicating that the most compressible layers of peat are around this area. This agrees well with what was found in the field and laboratory investigation (Chapter 3), where it was found that the most surficial deposits where less compressible, and that the deeper deposits where harder to sample.
Figure 4.38: Surface settlement profile $A - A'$: (a) ground movement relative to S.D., (b) settlement.
Figure 4.39: Surface settlement profile B – B': (a) ground movement relative to S.D., (b) settlement.
Figure 4.40: Settlement vs depth: (a) spacing of 0.85 m, (b) spacing of 1.20 m.

Considering the maximum settlement values given in Figure 4.37 and the depth of each settlement plate and cell (see §4.2.2), the maximum strains achieved during vacuum
consolidation are presented in Table 4.1. These strains were achieved under maximum pore water pressure reductions of between 45.3 kPa and 56.9 kPa for the 0.85 m spacing area, and between 35.6 kPa and 49.7 kPa for the 1.20 m spacing area. When these values are compared with the strains achieved during the oedometer tests (Figure 3.33), under similar loads (between 25 kPa and 50 kPa), it can be seen that similar results were obtained. For the samples from depths of 0.90 – 1.55 m the strains were between 14% and 24%, while for the samples from depths of 1.50 – 2.3 m and 2.4 – 3.2 m the strains ranged from 24% to 38%.

Table 4.1: Maximum strains measured during pumping.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Spacing 0.85 m</th>
<th>Spacing 1.20 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface</td>
<td>30.9</td>
<td>30.1</td>
</tr>
<tr>
<td>0.90</td>
<td>32.3</td>
<td>34.0</td>
</tr>
<tr>
<td>1.90</td>
<td>28.5</td>
<td>27.2</td>
</tr>
<tr>
<td>2.65</td>
<td>17.8</td>
<td>17.2</td>
</tr>
</tbody>
</table>

The volume of water pumped out of the bog during the test is plotted in Figure 4.41, along with the one-dimensional volumetric compression, considering only vertical movement occurred during testing. The one-dimensional volumetric compression is calculated by averaging the settlement between plates 2 and 5, and multiplying it by the 10 x 10 m of the area. As it can be seen, the volume of water is almost 3 times higher than the volumetric compression, indicating that water was not only pumped out from the peat under the membrane, but also from the gravel beneath the peat. Some water may have also come from the peat outside the membrane; however, as no decline was observed on the standpipes installed around the area, and considering the higher permeability in the gravel layer, the water is more likely to have come from it.

The settlement induced in test area was observable the next day after pumping started. Figure 4.42 shows a view of this settlement on 29th January 2010, using a white measuring tape to show the original level of the granular bed before pumping. Figure 4.43 show the settlement on 1st June 2010.
Figure 4.41: Volume of water extracted during vacuum pumping.

Figure 4.42: Visual of settlement at the centre of the test area on 29th January 2010.
4.3.2.2 Pore water pressure

Figure 4.44 shows the pore water pressures versus time recorded at different depths, for both spacing subareas. The change in pore water pressure (ΔPWP) with respect to 30th November 2009 is presented in Figure 4.45. In Figure 4.46 and Figure 4.47, the PWP and effective stress variation with depth for both spacing subareas are presented for selected dates; while Figure 4.48 shows the ΔPWP variation with depth for the same dates.

From Figure 4.44(a), Figure 4.45(a), Figure 4.46(a) and Figure 4.48(a) it can be seen that for the 0.85 m spacing subarea there is a similar reduction in the PWP at depths between 0.90 m and 2.65 m within the peat layer, being the maximum reduction between 45.3 kPa and 56.9 kPa, on 5th October 2010; however, the piezometer located at 2.65 m on the edge had a slightly lower reduction of 38.9 kPa on the same date. Nonetheless, for the period between 6th September 2010 and 11th October 2010, the average vacuum level was 63 kPa (Figure 4.35(b)), showing that the full vacuum pressure was not transmitted to the monitored points in the peat layer. The piezometer located at 4.05 m depth in the peat–gravel interface presented a maximum reduction of only 13.9 kPa.
Figure 4.44: Pore water pressure variation with time, during and after the pumping stage: (a) for 0.85 m PVD spacing, (b) for 1.20 m PVD spacing.
Figure 4.45: ΔPWP versus time: (a) for 0.85 m PVD spacing, (b) for 1.20 m PVD spacing.
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Figure 4.46: Ground pressures variation with depth, during the pumping stage, for selected dates for 0.85 m spacing section: (a) Pore water pressure, (b) Effective and total stresses.
Figure 4.47: Ground pressures variation with depth, during the pumping stage, for selected dates for 1.20 m spacing section: (a) Pore water pressure, (b) Effective and total stresses.
Figure 4.48: $\Delta$PWP versus depth: (a) for 0.85 m PVD spacing, (b) for 1.20 m PVD spacing
It is important to remember that the PVDs were only installed to a depth of 2.65 m. The pore water pressures measured at 4.05 m depth followed the same trend of all other depth, with lower pressure reductions though, until the 31st December 2010, remaining relatively constants afterwards, with small drops in pressure on 1st June 2010 and 11th September 2010 (Figure 4.44(a)). The smaller response and later unresponsiveness of the pressures at this depth is explained due to the higher permeability of the gravel, allowing the vacuum pressure to escape through this layer.

For the 1.20 m spacing subarea, Figure 4.44(b), Figure 4.45(b), Figure 4.47(a) and Figure 4.48(b) present very similar results that the ones for the 0.85 m spacing subarea, with a maximum pore pressure reduction between 35.6 kPa and 49.7 kPa, at depths from 0.90 m and 3.72 m within the peat layer, occurring on the period between 5th October 2010 and 13th October 2010. Note that for this subarea the pore pressure reductions were slightly lower than for the 0.85 m spacing section. In the 1.20 m spacing section, the deepest piezometer is located at a depth of 3.72 m, 1.05 m under the bottom of the PVDs and 0.30 m above the peat–gravel interface. Despite being located closer to the interface than to the PVDs, the PWP reduction is at about the same level, and follows the same trend, than the other piezometers.

The pore water pressures measured by the piezometer located 5.0 m outside the treated area did not follow the same trend as the pressures inside the test area. The pressures kept increasing until the 29th January 2010, following a trend it had from before the start of the test (Figure 4.44). Afterwards, the pressures dropped more or less continually until 1st April 2010, where they remained approximately constant until the end of the test.

From the previous figures it can be seen that the value of the absolute pore water pressures recorded is only slightly lower, for both testing sections, for depths of 0.90 m when compared with the PWP at depths of 1.90 m during the pumping, indicating that the pore water pressure reduction is approximately the same, for both depths, as shown in Figure 4.45. For the spacing of 0.85 m the maximum pore water pressure reduction is 56.8 kPa at 0.90 m depth while is 57.0 kPa at 1.90 m depth. At the 1.20 m spacing section the maximum pore water pressure reduction is 46.1 kPa at 0.90 m depth and 49.7 kPa at 1.90 m depth. Figure 4.44(b) also shows that the absolute pore water pressures at 2.65 m and
3.72 m depth, in the peat layer, tend to equilibrate during the pumping periods, though to a lower value than at 0.90 m and 1.90 m depth.

In spite of the difference on the pore pressure reduction, Figure 4.48 shows that this reduction is somewhat uniform along the entire peat deposit, for both PVD spacing areas, with a significantly lower reduction when the gravel layer is reached.

With respect to the effective stress (Figure 4.46(b) and Figure 4.47(b)), a significant increase is observed for the peat deposit. As mentioned in §2.2.2.1, the effective stress increment is equal to the pore water pressure reduction.

Figure 4.49 and Figure 4.50 show the piezometric head for both the peat and the gravel, in relation to S.D. For the peat, the values are obtained by converting the pore pressure measurement in kPa into mH\textsubscript{2}O, and then relating this values position of the piezometer with respect to S.D. All the values for the gravel are directly related to S.D., as the values are taken in meters of water head.

Figure 4.49 shows how before pumping the piezometric head for the peat at all depths was at about the same level as the water table, which is shown in Figure 4.50(b). However, the piezometric head of the gravel was approximately 1.7 m below. After the construction of the granular bed, the increase in the piezometric head was higher for the gravel than for the peat. After starting the pumping process, a drop on all the heads is observed, indicating that, as discussed above in §4.3.2.1, there is a flow of water from the gravel into the peat due to the vacuum pressure.

On the 10\textsuperscript{th} March 2010, the piezometer for boreholes 1 and 2 were installed (see §3.3.6). The piezometric head in this piezometer agree well with the head from the VW piezometer in the gravel layer in the 0.85 m PVD spacing subarea, particularly the ones in BH-1. After the pumping process is terminated, the piezometric head in the peat recovers very quickly to the original value, for both spacing subareas. Furthermore, the value of the piezometric head for the gravel is also slightly increased at all points, indicating that the vacuum pressure slightly affects the gravel layer.
Figure 4.49: Piezometric head variation with time: (a) for 0.85 m PVD spacing, (b) for 1.20 m PVD spacing.
Figure 4.50: (a) Piezometric head in gravel layer, (b) water table levels outside test area.
4.3.2.3 Degree of Consolidation (DOC)

Based on the settlement and pore water pressure readings (see §4.3.2.1 and §4.3.2.2), the degree of consolidation achieved during vacuum consolidation in the treated area was estimated using the procedures detailed in §2.2.2.3.

4.3.2.3.1 Degree of Consolidation by the Asaoka method

Figure 4.51 shows the Asaoka plot for the centre of the test area, for the entire duration of the project, with a time interval $\Delta t = 18$ days. As it can be seen, the use of two different pumping systems, achieving two different average vacuum pressures, acted as a two stage loading system, deviating the original straight line found for the first pumping system (see §2.2.2.3.1). Table 4.2 shows the average vacuum pressure achieved during both pumping stages, as well as the ultimate settlement estimated using Asaoka's method.

Considering the maximum settlement was achieved on the 27th Oct 2010 (see §4.3.2.1), and using the ultimate settlement estimated for the second stage, the average degrees of consolidation, at the centre of both spacing areas, are shown in Table 4.3.
Table 4.2: Average vacuum pressure and ultimate settlement for the two pumping stages.

<table>
<thead>
<tr>
<th>Pumping stage</th>
<th>Average vacuum pressure (kPa)</th>
<th>Ultimate settlement (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>First</td>
<td>45.1</td>
<td>1.08</td>
</tr>
<tr>
<td>Second</td>
<td>54.4</td>
<td>1.27</td>
</tr>
</tbody>
</table>

Table 4.3: Average degree of consolidation for both spacing areas by Asaoka’s method.

<table>
<thead>
<tr>
<th>PVD spacing areas</th>
<th>Average DOC</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.85 m</td>
<td>87%</td>
</tr>
<tr>
<td>1.20 m</td>
<td>84%</td>
</tr>
</tbody>
</table>

4.3.2.3.2 Degree of Consolidation by the PWP monitoring method

In order to obtain the degree of consolidation using the pore water pressure procedure, it is necessary to calculate the suction line, which depends on the vacuum pressure applied (see §2.2.2.3.2 and equation (2.16)). As it was explained in § 4.3.2, the vacuum pressure varied significantly due to different technical difficulties. However, considering the maximum vacuum pressure applied was 80 kPa, the DOC was calculated using this value to estimate the suction line. Table 4.4 and Table 4.5 show the degree of consolidation at different depth in the peat layer, as well as the average DOC, for both spacing areas.

Table 4.4: Degree of consolidation for PVD spacing of 0.85 m by PWP method.

<table>
<thead>
<tr>
<th>Spacing = 0.85 m</th>
<th>Depth (m)</th>
<th>DOC</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.9</td>
<td>68%</td>
</tr>
<tr>
<td></td>
<td>1.9</td>
<td>71%</td>
</tr>
<tr>
<td></td>
<td>2.65</td>
<td>56%</td>
</tr>
<tr>
<td></td>
<td>2.65T</td>
<td>49%</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>65%</td>
</tr>
</tbody>
</table>

As it can be observed, the average degrees of consolidation estimated by the settlement method are higher compared to the values obtained by the PWP method. Similar results have been pointed by others such as Hansbo (1997), Yan and Chu (2003), Arulrajah and Bo (2008); in both vacuum loading and embankment loading projects.
Table 4.5: Degree of consolidation for PVD spacing of 1.20 m by PWP method.

<table>
<thead>
<tr>
<th>Spacing = 1.20 m</th>
<th>Depth (m)</th>
<th>DOC</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.9</td>
<td>55%</td>
</tr>
<tr>
<td></td>
<td>1.9</td>
<td>60%</td>
</tr>
<tr>
<td></td>
<td>2.65</td>
<td>44%</td>
</tr>
<tr>
<td></td>
<td>3.72</td>
<td>49%</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td>52%</td>
</tr>
</tbody>
</table>

According to Hansbo (1997), the difference could be attributed to the following factors:

- Both the settlements and the pore water pressures were measured at specific points only. Thus the data may not be representative of the average values for the whole layer.
- There are uncertainties involved in the prediction of the ultimate settlement.
- It was a large strain consolidation problem.

Arulrajah and Bo (2008) found that, for Asaoka's method the magnitude of ultimate settlement decreases and the degree of consolidation subsequently increases as a longer period of assessment is used in the prediction. According to the authors, as the time interval increases, a cut-off time interval is obtained after which increasing time intervals would converge to the same magnitude of ultimate settlement. However, the use of increasing time intervals would be restricted by the number of data points available to assess the best-fit line.

Furthermore, Arulrajah and Bo (2008) also found that the degree of consolidation predicted by the piezometers is found to be in good agreement with the Asaoka's method for the early period of assessment. However as the assessment period increases, the piezometer indicates lower degree of consolidation as compared to field settlement predictions. Mikasa (1995) attributed this to the non-linearity of the stress-strain behaviour of soil. In the non-linearity theory, the effective stress gain is slower in initial stage whereas settlement rate is faster in such stage. Therefore degree of consolidation worked out from settlement ratio is much more greater than that worked out from pore pressure (Arulrajah and Bo, 2008).
However, another factor to consider when using Asaoka’s method, is that the field settlement observations do not discriminate the amount of settlement produced by the consolidation process (excess pore water pressure dissipation) and by the creep compression (see §2.3, §2.3.2.2 and §2.3.3.2), thus over-estimating the degree of consolidation. On the other hand, the pore water pressure monitoring method allows the determination of the degree of consolidation only due to the pore water pressure dissipation of the soil layer.

Comparing average degree of consolidation values calculated using Asaoka’s method (Table 4.3) with the values obtained from the PWP method (Table 4.4 and Table 4.5), it can be concluded that, in vacuum consolidation, the secondary compression is a major contributor of the observed long term settlement of peat, in the same manner as it is in embankment loading (see §2.4.2).

### 4.3.2.4 Horizontal coefficient of consolidation ($c_h$) and horizontal permeability ($K_h$)

The values of the horizontal coefficient of consolidation and the horizontal permeability were back-calculated following the procedure described in §2.2.2.4. Considering the variation of the vacuum pressures applied during the project (§4.3.2 and Figure 4.35), only the initial 24 days of testing were consider in the back-analysis of these parameters, as the vacuum pressure during this period was somewhat constant with an average value of 50.5 kPa. The geometrical properties of the PVDs are shown in Table 4.6 and Table 4.7, while the $m_v$ values were taken as the average of the initial values determined from the oedometer tests (Table 4.8).

### Table 4.6: Equivalent drain diamater.

<table>
<thead>
<tr>
<th>Width (mm)</th>
<th>Thickness (mm)</th>
<th>$d_v$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>4</td>
<td>0.052</td>
</tr>
</tbody>
</table>

### Table 4.7: Equivalent diameter of the unit cell.

<table>
<thead>
<tr>
<th>Spacing (m)</th>
<th>Area (m$^2$)</th>
<th>Re (m)</th>
<th>$d_v$ (m)</th>
<th>n</th>
<th>F(n)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.85</td>
<td>0.7225</td>
<td>0.48</td>
<td>0.96</td>
<td>18.44</td>
<td>2.174</td>
</tr>
<tr>
<td>1.2</td>
<td>1.44</td>
<td>0.68</td>
<td>1.35</td>
<td>26.04</td>
<td>2.5148</td>
</tr>
</tbody>
</table>
Table 4.8: Average coefficient of volume compressibility

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Oed 1</th>
<th>Oed 2</th>
<th>Oed 4</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.9-1.55</td>
<td>5.92</td>
<td>6.39</td>
<td>5.50</td>
<td>5.94</td>
</tr>
<tr>
<td>1.5-2.3</td>
<td>14.43</td>
<td>9.76</td>
<td>13.63</td>
<td>12.61</td>
</tr>
<tr>
<td>2.4-3.2</td>
<td>8.22</td>
<td>13.33</td>
<td>5.25</td>
<td>8.93</td>
</tr>
</tbody>
</table>

Table 4.9 and Table 4.10 present the estimation of the degree of consolidation for the first 24 days of testing, and the calculation of \( c_h \) and \( K_h \), based on this DOC, for both PVD spacing areas.

Table 4.9: DOC, horizontal coefficient of consolidation and horizontal permeability for 0.85 m spacing area.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>( U_h )</th>
<th>( T_h )</th>
<th>( c_h ) (m²/day)</th>
<th>( c_s ) (m²/year)</th>
<th>( K_h ) (m/day)</th>
<th>( K_s ) (m/year)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.9</td>
<td>60%</td>
<td>0.25</td>
<td>0.00957</td>
<td>3.49</td>
<td>0.00056</td>
<td>0.203</td>
</tr>
<tr>
<td>1.9</td>
<td>68%</td>
<td>0.31</td>
<td>0.01180</td>
<td>4.31</td>
<td>0.00146</td>
<td>0.533</td>
</tr>
<tr>
<td>2.65</td>
<td>63%</td>
<td>0.27</td>
<td>0.01022</td>
<td>3.73</td>
<td>0.00090</td>
<td>0.327</td>
</tr>
<tr>
<td>2.65</td>
<td>53%</td>
<td>0.21</td>
<td>0.00791</td>
<td>2.89</td>
<td>0.00069</td>
<td>0.253</td>
</tr>
</tbody>
</table>

Table 4.10: DOC, horizontal coefficient of consolidation and horizontal permeability for 1.20 m spacing area.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>( U_h )</th>
<th>( T_h )</th>
<th>( c_h ) (m²/day)</th>
<th>( c_s ) (m²/year)</th>
<th>( K_h ) (m/day)</th>
<th>( K_s ) (m/year)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.9</td>
<td>43%</td>
<td>0.18</td>
<td>0.01353</td>
<td>4.94</td>
<td>0.00079</td>
<td>0.288</td>
</tr>
<tr>
<td>1.9</td>
<td>53%</td>
<td>0.24</td>
<td>0.01816</td>
<td>6.63</td>
<td>0.00225</td>
<td>0.820</td>
</tr>
<tr>
<td>2.65</td>
<td>46%</td>
<td>0.19</td>
<td>0.01481</td>
<td>5.41</td>
<td>0.00130</td>
<td>0.474</td>
</tr>
<tr>
<td>3.72</td>
<td>48%</td>
<td>0.20</td>
<td>0.01559</td>
<td>5.70</td>
<td>0.00137</td>
<td>0.499</td>
</tr>
</tbody>
</table>

Table 4.11, the permeability values obtained from the oedometer tests are given. As it can be seen, the values from the samples at depths of 1.50-2.30 m and 2.40-3.20 m agree well with the back calculated permeability values from the field trial, with ranges between 0.16-1.5 m/year for the oedometer tests and between 0.25-0.82 m/year for the field trial for both spacing subareas. However, the permeability values back calculated from the field data for the surficial peat (0.9 m deep) are in the same range of the other strata, but the
permeability values calculated from the oedometer tests are higher by approximately a factor of 10, as mentioned in §3.4.3.3.

Table 4.11: $K_0$, from oedometer tests.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Test</th>
<th>$K_0$ (m/s)</th>
<th>$K_0$ (m/day)</th>
<th>$K_0$ (m/year)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.9-1.55</td>
<td>Oed 1</td>
<td>1.051E-07</td>
<td>0.00908</td>
<td>3.31621</td>
</tr>
<tr>
<td></td>
<td>Oed 2</td>
<td>7.713E-08</td>
<td>0.00666</td>
<td>2.43391</td>
</tr>
<tr>
<td></td>
<td>Oed 4</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>1.50-2.30</td>
<td>Oed 1</td>
<td>1.947E-08</td>
<td>0.00168</td>
<td>0.61435</td>
</tr>
<tr>
<td></td>
<td>Oed 2</td>
<td>4.935E-09</td>
<td>0.00043</td>
<td>0.15574</td>
</tr>
<tr>
<td></td>
<td>Oed 4</td>
<td>1.949E-08</td>
<td>0.00168</td>
<td>0.61510</td>
</tr>
<tr>
<td>2.40-3.20</td>
<td>Oed 1</td>
<td>4.911E-08</td>
<td>0.00424</td>
<td>1.54968</td>
</tr>
<tr>
<td></td>
<td>Oed 2</td>
<td>1.861E-08</td>
<td>0.00161</td>
<td>0.58734</td>
</tr>
<tr>
<td></td>
<td>Oed 4</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Nonetheless, Galvin and Hanrahan (1967) reported peat permeability values, determined from field pumping tests, ranging from 1.072 to 3.585 m/years. The site was located at the Peatland Experimental Station of the Agricultural Institute, County Mayo, with an average peat depth of 4.3 m and an average water content of 1400%. Rycroft et al. (1975) reported hydraulic conductivity values, determined from field falling and constant head tests, varying between 1.58 and 1200 m/year. According to the authors, the site was located at a partly intervened bog, with moderately humified peat (von Post ranging between $H_3$ – $H_5$).

Based on the literature, Hobbs (1986) presented a summary of different values of field permeability for peat, depending on the botanical composition and humification level (Table 4.12).

Table 4.12: Variation of field permeability of peat (Hobbs, 1986).

<table>
<thead>
<tr>
<th>Peat type</th>
<th>$K$ (m/s)</th>
<th>$K$ (m/year)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Highly humified blanket peat</td>
<td>$6\times10^{-10}$</td>
<td>$1.89\times10^{-7}$</td>
</tr>
<tr>
<td>Slightly humified fen peat</td>
<td>$5\times10^{-9}$</td>
<td>$157784.76$</td>
</tr>
<tr>
<td>Sphagnum peat, $H_6$ to $H_{10}$</td>
<td>$6\times10^{-8}$</td>
<td>1.89</td>
</tr>
<tr>
<td>Sphagnum peat, $H_2$</td>
<td>$1\times10^{-5}$</td>
<td>315.57</td>
</tr>
<tr>
<td>Sedge peat, $H_3$ to $H_5$</td>
<td>$1\times10^{-5}$</td>
<td>315.57</td>
</tr>
<tr>
<td>Brushwood peat, $H_4$ to $H_5$</td>
<td>$1\times10^{-5}$</td>
<td>315.57</td>
</tr>
<tr>
<td>Sphagnum, cotton sedge and heather peat, $H_1$ to $H_6$</td>
<td>$1\times10^{-7}$</td>
<td>3.16</td>
</tr>
<tr>
<td>$1\times10^{-6}$</td>
<td>31.56</td>
<td></td>
</tr>
</tbody>
</table>
As it can be observed, the permeability values determined from the field vacuum consolidation test and from the oedometer tests, are in the low range, or even lower, than the values reported in the literature. This may be explained by the compression process suffered induced to the bog by 60 years of intensive peat harvesting and drainage (see §3.2.3).

However, it is important to consider that the permeability values determined from the vacuum consolidation test and from the oedometer tests were back-calculated using Terzaghi's theory, which is based on the assumption that the soil compressibility and permeability are constant during the consolidation process. Thus, the values of $c_v$, $c_h$, $m_v$, $K_v$ and $K_h$ are intermediate between the actual maximum and minimum in each load step (Tavenas et al., 1983a). This assumption is a major weakness in the determination of the parameters required to model the behaviour of peat under a consolidation process, when the variation of the compressibility and permeability are considered, as the values estimated from this methodology are intermediate and not the real initial values.

As it will be shown later in §4.4.1, the permeability values used to adequately model the field behaviour in PLAXIS (Table 4.13), for the three peat layers considered, were between 2.5 and 100 times higher than the values back-calculated from the oedometer tests. The values used in PLAXIS, however, are in good agreement with the field permeability values reported in the literature for moderately to highly humified Sphagnum and Sedge peats (Table 4.12), as this is the case for Ballydermot bog (see §3.4.1.7).

**Table 4.13:** K values for PLAXIS model.

<table>
<thead>
<tr>
<th>Layer</th>
<th>K (m/day)</th>
<th>K (m/year)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper peat</td>
<td>0.3</td>
<td>109.57275</td>
</tr>
<tr>
<td>Middle peat</td>
<td>0.2</td>
<td>73.0485</td>
</tr>
<tr>
<td>Lower peat</td>
<td>0.01</td>
<td>3.652425</td>
</tr>
</tbody>
</table>

Furthermore, comparing the vertical and horizontal permeability values determined from the oedometer tests and the field results, respectively, it can be seen that Ballydermot peat does not show a highly hydraulic anisotropic behaviour, with $K_h/K_v$ values ranging between 0.1 and 1.16. As it will be shown later (§4.4.1), no hydraulic anisotropy was
consider \( K_h/K_v = 1 \) for the numerical models, rendering good results when compared with the field behaviour.

4.3.2.5 Cracks around the test area

Cracking around the test area was initially observed on 15\textsuperscript{th} April 2010, while the first vacuum generating system was still in place. The cracks first appeared on the east side, just outside the test area towards the 1.20 m PVDs spacing subarea (Figure 4.52), and on the east side of the shed, to the south of the test area.

![Figure 4.52: Crack on the east side of the test area on 15\textsuperscript{th} April 2010.](image)

The cracks appeared after a period of continuous pumping after mid-march, with vacuum pressures ranging from 37 to 80 kPa (Figure 4.35(a)). After 19\textsuperscript{th} April 2010, the vacuum pressure level varied between 10 and 38 kPa, up to the end of June when the pumping system was changed, and no further widen of the cracks was observed.

On 26\textsuperscript{th} August 2010, after one month of pumping with the second vacuum generating system, the cracks appeared to be widening and more cracks appeared on the west and south sides of the test area. The crack on the east side of the test area was 8.2 m long, 0.1
m wide and 0.82 m deep (Figure 4.53). Figure 4.54 shows the progression of the crack next to the shed, from when it was originally observed in April to after the end on the project in early November 2010.

![Image](image_url)

**Figure 4.53:** Crack on the east side of the test area on 26th August 2010.

![Image](image_url)

**Figure 4.54:** Crack next to the shed, south of the test area (a) 26th August 2010, (b) 7th September 2010, and (c) 3rd November 2010.

Cracks were also observed to the west of the test area (Figure 4.55), next to the 0.85 m PVD spacing subarea, during the second period of pumping; however, these were not as wide and deep as on the east side. The bigger crack was 3.6 m long, 0.02 m wide and 0.065 m deep on 26th August 2010. The final state of the cracks, after pumping stopped, on all three sides of the test is shown in Figure 4.56.
Chapter 4  TCD/NRA VACUUM CONSOLIDATION FIELD TEST

Figure 4.55: Crack on the west side of the test area on 7th September 2010.

Figure 4.56: Final state of cracks after pumping on 3rd November 2010: (a) east side, (b) south side, and (c) west side.

It is important to note that no cracks appeared to the north of the test area. This is probably due to fact that this side was used by the Bord na Móna vehicles to circle around the test area and go into the peat harvesting fields.

By comparing the dates when cracks appeared or widened with the vacuum levels (Figure 4.35), it can be observed that the cracks appeared during periods where the vacuum pressure were at the highest levels (65 kPa to 80 kPa). These high vacuum pressures also
induced the highest pore water pressure reductions on the ground (Figure 4.45), with \( \Delta PWP \) up to 45.5 kPa for the 0.85 m PVD spacing area and up to 40.9 kPa for the 1.20 m area, at a depth of 0.90 m. However, at some stages, the \( \Delta PWP \) is slightly higher at a depth of 1.90 m with values of up to 50 kPa for the 0.85 m PVD spacing area and up to 43.0 kPa for the 1.20 m area. Nevertheless, as it was mentioned above, the maximum cracking depth was 0.82 m. The cracking depth is closely related with the lateral stresses on the ground.

As it will be explained in the Appendix B, the vacuum consolidation process follows a one-dimensional consolidation stress path. However, around the test area, due to the consolidation movements, the lateral stress of the soil elements is gradually reduced until tension stresses are reached, and thus vertical tension cracks are formed. Once the tension cracks appear, there is no further lateral restrain on the ground and inwards lateral deformation occurs due to the suction pressure inside the treated area.

The formation of tension cracks around the treated area may have significant effects in the consolidation process such as, the reduction of the vacuum pressure due to the breakage of the seal from the atmosphere, lateral deformations on the ground as a result of the reduction of the lateral restrain of the soil elements, the introduction of a significant safety hazard during the ground improvement works. In order to minimize the effects that the tension cracks may have, these should be filled using an impervious material, such as bentonite, reintroducing lateral restrain in the ground and, at the same time, improving the seal of the ground from the atmospheric pressure.

### 4.4 NUMERICAL MODELLING

#### 4.4.1 Field test back-analysis

A back-analysis of the field test was performed using finite element package PLAXIS version 2010, with both, the Soft Soil and the Soft Soil Creep models, as described in §2.3.3. The back-analysis was carried out on two single cell elements, one for each spacing subarea. The model used was axisymmetric with 15-node triangular elements. The equivalent radius, \( R \), of the cross-section area was calculated using equation (4.1), as described by Craig (2004), where \( S \) is the spacing between the PVDs in a square grid. Table 4.14 shows the equivalent radius used for both single cell models.
Table 4.14: Equivalent radius for single cell simulation.

<table>
<thead>
<tr>
<th>Spacing (m)</th>
<th>R (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.85</td>
<td>0.48</td>
</tr>
<tr>
<td>1.20</td>
<td>0.68</td>
</tr>
</tbody>
</table>

Figure 4.57 and Figure 4.58 show the single cell models used, including mesh, load, fixities, boundaries and water conditions. At the vertical boundaries horizontal movement is prevented, while the bottom boundaries are fixed for horizontal and vertical displacement. The upper horizontal boundary is free to move in the vertical direction, and a linear distributed load is applied on top (Figure 4.57(a) and Figure 4.58(a)). Four main soil layers were set for the model. The upper layer is the granular bed, modelled with the Mohr–Coulomb material model and 0.4 m thick. The other three layers represent the peat deposit. The upper peat layer is 1.0 m thick, the middle layer is 1.4 m thick and the lower layer is 1.2 m thick. For these three layers, both the SS and SSC material models were used. The soil is free to drain at the top, and a vertical drain with pore pressure equal to zero at all time was set at the left vertical boundary, to a depth of 2.65 m. Drainage was prevented in the other two sides. The water level was set above the upper peat layer (Figure 4.57(b) and Figure 4.58(b)).

The initial design parameters ($\lambda^*, k^*, \mu^*$) used, were taken as the average values of the parameters obtained from the models of oedometer tests in chapter 3 (Table 3.11 and Table 3.12). These parameters were later refined when running the model, and the final results are presented in Table 4.15 for the SS model and in Table 4.16 for the SSC model. The final refined parameters are close to the values estimated from the oedometer tests (Table 3.11 and Table 3.12), indicating that these parameters can be obtained from the conventional oedometer tests, normally used in the practice. The parameters for the Mohr–Coulomb model are given in Table 4.17.
Figure 4.57: Single cell PLAXIS model for 0.85 m PVDs spacing: (a) fixities, load and mesh; and (b) boundaries and water conditions.
Figure 4.58: Single cell PLAXIS model for 1.20 m PVDs spacing: (a) fixities, load and mesh; and (b) boundaries and water conditions.
Table 4.15: Soft Soil model parameters

<table>
<thead>
<tr>
<th></th>
<th>Upper peat</th>
<th>Middle peat</th>
<th>Lower peat</th>
</tr>
</thead>
<tbody>
<tr>
<td>POP</td>
<td>10</td>
<td>5</td>
<td>11</td>
</tr>
<tr>
<td>( \lambda^* )</td>
<td>0.125</td>
<td>0.16</td>
<td>0.16</td>
</tr>
<tr>
<td>( k^* )</td>
<td>0.05</td>
<td>0.055</td>
<td>0.04</td>
</tr>
<tr>
<td>( \lambda^* / k^* )</td>
<td>2.5</td>
<td>2.91</td>
<td>4</td>
</tr>
<tr>
<td>( y_0 (kN/m^3) )</td>
<td>10.45</td>
<td>10.1</td>
<td>10.06</td>
</tr>
<tr>
<td>( c_0 )</td>
<td>7.42</td>
<td>13.81</td>
<td>12.05</td>
</tr>
<tr>
<td>( k_c (m/day) )</td>
<td>0.3</td>
<td>0.2</td>
<td>0.01</td>
</tr>
<tr>
<td>( k_h (m/day) )</td>
<td>0.3</td>
<td>0.2</td>
<td>0.01</td>
</tr>
<tr>
<td>( c_k )</td>
<td>1.8</td>
<td>3.44</td>
<td>3.44</td>
</tr>
</tbody>
</table>

Table 4.16: Soft Soil Creep model parameters.

<table>
<thead>
<tr>
<th></th>
<th>Upper peat</th>
<th>Middle peat</th>
<th>Lower peat</th>
</tr>
</thead>
<tbody>
<tr>
<td>POP</td>
<td>10</td>
<td>5</td>
<td>11</td>
</tr>
<tr>
<td>( \lambda^* )</td>
<td>0.125</td>
<td>0.16</td>
<td>0.16</td>
</tr>
<tr>
<td>( k^* )</td>
<td>0.05</td>
<td>0.055</td>
<td>0.04</td>
</tr>
<tr>
<td>( \mu^* )</td>
<td>0.0065</td>
<td>0.0078</td>
<td>0.009</td>
</tr>
<tr>
<td>( \lambda^* / k^* )</td>
<td>2.5</td>
<td>2.91</td>
<td>4</td>
</tr>
<tr>
<td>( \lambda^* / \mu^* )</td>
<td>13.54</td>
<td>13.46</td>
<td>13.33</td>
</tr>
<tr>
<td>( \lambda^* / \mu^* )</td>
<td>19.23</td>
<td>20.51</td>
<td>17.77</td>
</tr>
<tr>
<td>( y_0 (kN/m^3) )</td>
<td>10.45</td>
<td>10.1</td>
<td>10.06</td>
</tr>
<tr>
<td>( c_0 )</td>
<td>7.42</td>
<td>13.81</td>
<td>12.05</td>
</tr>
<tr>
<td>( k_c (m/day) )</td>
<td>0.3</td>
<td>0.2</td>
<td>0.01</td>
</tr>
<tr>
<td>( k_h (m/day) )</td>
<td>0.3</td>
<td>0.2</td>
<td>0.01</td>
</tr>
<tr>
<td>( c_k )</td>
<td>1.8</td>
<td>3.44</td>
<td>3.44</td>
</tr>
</tbody>
</table>

Table 4.17: Mohr–Coulomb model parameters

<table>
<thead>
<tr>
<th>Granular bed</th>
<th>( y_0 (kN/m^3) )</th>
<th>( E' (kPa) )</th>
<th>( v' )</th>
<th>( c' (kPa) )</th>
<th>( \phi' (')</th>
<th>( k_c (m/day) )</th>
<th>( k_h (m/day) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>150 x 10^3</td>
<td>0.35</td>
<td>0.1</td>
<td>30</td>
<td>864</td>
<td>864</td>
<td></td>
</tr>
</tbody>
</table>

Since PLAXIS does not allow to assign negative pressure in the models, it was decided to artificially increase the pore water pressure in the soil by introducing a surcharge load applied instantaneously on the surface, equivalent to the vacuum pressure, and then allow the consolidation of the ground. This is the traditional method for modelling vacuum consolidation as it was mentioned on §2.2.6, and as described by Park et al. (1997).

As the vacuum applied throughout the project varied considerably due to several reasons (Figure 4.35), the finite element models were loaded and unloaded accordingly in different stages. Figure 4.59 shows the vacuum pressure readings taken during the project using the vacuum gauge (blue) and the data logger (red), and the vacuum loading stages as introduced in PLAXIS (black).
Figure 4.59: Vacuum pressure stages for PLAXIS models.

Figure 4.60 shows the comparison between the measured and estimated settlement versus time, using the SS and the SSC model, for both PVDs spacing subareas. From this figure it can be seen that both models give a relative good agreement, particularly for the 0.9 and 1.9 m depths, for both spacing subareas. The SSC model tends to produce higher values of compression due to the creep effect. Finally, both models seem to overestimate the heave.

Figure 4.61 to Figure 4.67 show the comparison between the measured and calculated PWP versus time, using the SS and SSC models, for both PVDs spacing subareas at all depths. As expected, the calculated PWP did not model the behaviour shown by the measured values. This is because of the variation of the total stress introduced by the surcharge load applied at the surface, to artificially increase the PWP to let it dissipate afterwards (see §2.2.6).
Figure 4.60: Comparison between measured and calculated settlement versus time: (a) for 0.85 m PVD spacing, (b) for 1.20 m PVD spacing.
Figure 4.61: Comparison between measured and calculated PWP versus time at 0.90 m depth for 0.85 m PVD spacing subarea.

Figure 4.62: Comparison between measured and calculated PWP versus time at 1.90 m depth for 0.85 m PVD spacing subarea.
Figure 4.63: Comparison between measured and calculated PWP versus time at 2.65 m depth for 0.85 m PVD spacing subarea.

Figure 4.64: Comparison between measured and calculated PWP versus time at 0.90 m depth for 1.20 m PVD spacing subarea.
Figure 4.65: Comparison between measured and calculated PWP versus time at 1.90 m depth for 1.20 m PVD spacing subarea.

Figure 4.66: Comparison between measured and calculated PWP versus time at 2.65 m depth for 1.20 m PVD spacing subarea.
**Figure 4.67:** Comparison between measured and calculated PWP versus time at 3.72 m depth for 1.20 m PVD spacing subarea.

### 4.4.2 Numerical model using a single vacuum load

Figure 4.68 shows the settlement results for the same single cell model used in the previous section, but considering a single vacuum load of 80 kPa during the entire improvement process. As it can be observed, for this case, the end of primary consolidation is reached faster in all the peat layers than when compared against the results obtained in the previous section. In all cases, the end of primary consolidation is reached, approximately, at around 40 days after the beginning of pumping.

A significant difference can be observed between the results from the SS and from the SSC models. The final settlements obtained with the latter method are considerable higher as these include the effect of creep. Furthermore, when the results from both models are compared with the field results, it can be seen that the final settlement estimated using the SSC model renders a better approximation to the final settlement achieved in the field, particularly for the surface and the upper peat layer.
Figure 4.68: Comparison between measured and calculated settlement, using a single vacuum load of 80 kPa, versus time: (a) for 0.85 m PVD spacing, (b) for 1.20 m PVD spacing.
In order to assess the influences of the mesh refinement in the settlement results, a simple single cell Soft Soil Creep model was set up in PLAXIS, with one peat layer only (Figure 4.69). The model used was axisymmetric with 15–node triangular elements. The radius of the cross-section area was 0.5 m and the height 2.0 m. At the vertical boundaries horizontal movement is prevented, while the bottom boundaries are fixed for horizontal and vertical displacement. The upper horizontal boundary is free to move in the vertical direction, and a linear distributed load is applied on top. The soil element is free to drain in all directions. The water level was set at the surface of the peat layer. The model parameters \( (\lambda^*, k^*, \mu^*) \) used, were the same as the Middle peat layer from the field test back-analysis (Error! Reference source not found.).

The model was studied under four different mesh conditions: (a) Fine mesh – No Updated mesh (Small strains), (b) Very fine mesh – No Updated mesh (Small strains), (c) Fine mesh – Updated mesh (Large strains), and (d) Very fine mesh – Updated mesh (Large strains). In all cases, the model was allowed to consolidate for 1000 days under a load of 80 kPa. Figure 4.70 shows the comparison between the settlement calculations for the four different analyses.

Table 4.18: Soft Soil Creep model parameters for mesh refinement.

<table>
<thead>
<tr>
<th>POP</th>
<th>Peat</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \lambda^* )</td>
<td></td>
<td>0.16</td>
</tr>
<tr>
<td>( k^* )</td>
<td></td>
<td>0.055</td>
</tr>
<tr>
<td>( \mu^* )</td>
<td></td>
<td>0.0078</td>
</tr>
<tr>
<td>( \lambda^<em>/k^</em> )</td>
<td></td>
<td>2.91</td>
</tr>
<tr>
<td>( (\lambda^<em>/k^</em>)/\mu^* )</td>
<td></td>
<td>13.46</td>
</tr>
<tr>
<td>( \lambda^<em>/\mu^</em> )</td>
<td></td>
<td>20.51</td>
</tr>
<tr>
<td>( \gamma_0 ) (kN/m²)</td>
<td></td>
<td>10.1</td>
</tr>
<tr>
<td>( c_u )</td>
<td></td>
<td>13.81</td>
</tr>
<tr>
<td>( k_e ) (m/day)</td>
<td></td>
<td>0.2</td>
</tr>
<tr>
<td>( k_n ) (m/day)</td>
<td></td>
<td>0.2</td>
</tr>
<tr>
<td>( c_n )</td>
<td></td>
<td>3.44</td>
</tr>
</tbody>
</table>

As it can be seen, there is no difference in the settlement estimated when the mesh coarseness is set to Fine or Very fine, employing a small strain analysis (No updated mesh). However, when the updated mesh option is used (large strain analysis), there is a slight difference between the results using different mesh coarseness. Furthermore, the
ultimate settlement calculated using the large strain analysis is considerably smaller than the one obtained with the small strain analysis.

Figure 4.69: Simple single cell PLAXIS SSC model: (a) fixities and load; and (b) boundaries and water conditions.
According to the Plaxis 2D 2010 - Reference Manual (Brinkgreve et al., 2010): "It should be noted than an updated mesh analysis takes much more time and is less robust than a normal calculation. Hence this option should only be used in special cases". Moreover, during a personal communication with Plaxis bv staff, the author was informed that a small strain analysis was adequate for type of analysis required for this project.

Considering the good agreement achieved between the laboratory and field settlement results and the back-analyses conducted (see §3.4.3.4 and §4.3.2.1), the aforementioned mesh refinement study, in addition to what is stated in the Plaxis 2D 2010 - Reference Manual and the recommendation from Plaxis bv staff, it can be concluded that the small strain finite element analysis, with a mesh coarseness set to Fine, is adequate for both the oedometer and the field tests.

4.5 TECHNICAL DIFFICULTIES

During the eleven months pumping stage, some technical difficulties arose reducing the pumping levels or even sometimes fully stopping the pumping process (Figure 4.35). The
difficulties included pipe leakage or bursting, water freezing in the vacuum line during winter time, airtight membrane piercing or tearing, ground water table variations, electrical cuts at site and chemical incrustation on the pipes and pumps.

4.5.1 Pipe burst and leak

During the first week of the pumping stage, the pumping process was stopped due to two faults that occurred on a reinforced plastic hose used to transport the water under pressure from the centrifugal pump to the jet pump (Figure 4.71). On 1\textsuperscript{st} December 2009 the hose was pulled out from the copper pipe at the centrifugal pump due to the high pressure of the water expelled from this pump. The fault was fixed on the same day. On 6\textsuperscript{th} December 2009, after arriving on site, it was noticed that the main tank that recirculated the water into the pumps was empty. This was due to the hose connecting the centrifugal pump to the jet pump split leaking the water from the system.

In order to avoid subsequent faults, the hose was replaced by a copper pipe line as shown earlier in Figure 4.17 and the test was restarted on 8\textsuperscript{th} December 2010.
4.5.2 Water freezing in the vacuum line

The low temperatures between mid-December 2009 and mid-March 2010 (Figure 4.26) caused the freezing of the ground surface (Figure 4.72) and, at times, of the water flowing through the vacuum line (Figure 4.73) connecting the granular bed and the jet pump. As has been explained (see §2.2.1), the vacuum line had the double function of transmitting the vacuum from the jet pump into the granular bed and to collect the water-air mixture from the peat deposit and to bring it to the tanks.

Figure 4.72: Frozen bog surface on 24th December 2009.

Figure 4.73: Frozen water in vacuum line on 13th January 2010.

Figure 4.35(a) shows the effect that the frozen water had on the vacuum level. The initial vacuum pressures recorded ranged between 52 kPa and 60 kPa. Around the 24th December
2009 the water started to freeze in the vacuum line, reducing the vacuum pressure to 14 kPa on 28th December 2009. On 31st December 2009 the water melted and the vacuum rose again to 60 kPa. During the first weeks of January 2010 the temperature dropped again, fully freezing the water in the vacuum line, impeding the vacuum transmission to the bog and reducing the vacuum pressure to zero on 13th January 2010. The melting–freezing cycle continued until mid-March 2010, generating variations in the vacuum pressure ranging between 0 kPa and 80 kPa.

4.5.3 Piercing of the airtight membrane and ground water table variations

Due to the low temperatures between December 2009 and March 2010, the peat coverage placed over the airtight membrane froze (Figure 4.72). The icy peat layer formed on top of the membrane punctured it in different places, reducing the seal and allowing air to come in the system, hence reducing the vacuum. This effect is particularly noticeable after the peat started to melt. The water in the vacuum line melted after 9th March 2010 with the respective increase in the vacuum pressure. However, on 11th March 2010 the vacuum pressure started to drop. On 22nd March 2010 a puncture was found on a corner at the surface of the bog and was repaired immediately.

Furthermore, variations on the ground water table depth also influence the vacuum level as the water acts as a seal around the treated area, preventing air from coming into the peat deposit. Figure 4.34 shows an increase in the ground water table depth until the 1st April 2010, which is related with continues rain fall at the end of March 2010 (Figure 4.24). In Figure 4.35(a) a rapid increase in the vacuum pressure can been seen after the 22nd March 2010, following the rise in the ground water table and after the puncture in the membrane was fixed.

Afterwards, a relatively dry period occurred from early April 2010 until mid-July 2010 (Figure 4.24), where the ground water table drop considerably (Figure 20), even deeper than 1.0 m at some standpipes. The drop in the ground water table allowed air to enter the treated zone, reducing the vacuum level achieved (Figure 4.35(a)).
A tear was found on the airtight membrane at the bottom of a side trench, just under the cables pipes, which was fixed on 8th April 2010, by adding a second plastic membrane to the area, filling the bottom with bentonite pellets and filling the rest with reworked peat (Figure 4.74).

From 29th April 2010 to 12th May 2010, the surface of the treated area was flooded with water in order to investigate if more surface punctures were present affecting the vacuum
levels (Figure 4.75). During this period, the vacuum level increased from 17 kPa to 38 kPa (Figure 4.35(a)). After flooding of the surface stopped, the vacuum levels started to drop again. However, the bog surface remained flooded for a period of over 30 days after the flooding process stopped, indicating that if holes were present, they were small. It is important to note, that no more holes were found.

4.5.4 Electrical cuts at site

During the pumping stage, two electrical cuts occurred on site due to works that were being conducted at the Bord na Móna workshops. The first cut occurred on 29th June 2010 (Figure 4.35(a)), during the period where the first pumping arrangement was used. This cut was used to change the pumping arrangement. The second electrical cut happened on 12th August 2010 and lasted until 26th Aug 2010 (Figure 4.35(b)).

4.5.5 Settlement cells breakage

As shown in Figure 4.37, the settlement cells reservoir was found empty on two occasions (18th February 2010 and 16th July 2010). The first time, this was because a nut connecting one of the cells tubes into the reservoir sheared due to the low temperatures during the winter (Figure 4.76), and allowed the antifreeze liquid to drain. The second time, a seal from one of the connectors also failed allowing the liquid to drain again.

![Figure 4.76: Nut after shear failure on 18th February 2010.](image-url)
4.5.6 Chemical incrustation on the pipes and pumps

Chemical incrustation usually results from the precipitation of dissolved mineral and gases in the water being pumped. The major forms of chemical incrustation include: calcium and magnesium carbonates incrustations, and iron and manganese incrustations (US Army Corps of Engineers, 1992). Incrustation may be of chemical or microbiological origin (Misstear et al., 2006) and, in pumping systems, can cause production losses by reduced heat transfer, more frequent turnarounds, emergency shutdowns, and many other factors. Equipment life can be shortened by corrosion and the carrying capacity of pipelines and other vessels can be drastically reduced. Costs can also be increased by the need for more frequent cleaning and added pumping requirements (Cowan and Weinritt, 1976).

Chemical incrustation appeared during the second period of pumping. On 7th August 2010 a reddish crust was noticed on the hose connected to the outlet of the liquid ring pump. The crust then extended to several other hoses in the system. Figure 4.77 shows the incrustation at the end of the vacuum line on 18th August 2010. The intense red colour on the pipe indicated the possible presence of iron or manganese minerals.

According to Misstear et al. (2006), iron and manganese minerals increase the solubility appreciably when the oxygen in the ground water becomes depleted as it happens in peat bogs. When the water is pumped and oxygen comes in contact with the ground water,
possibly combined with an increase in pH, if there is sufficient iron dissolved in the water, it can lead to the formation of a ferric incrustation in the pumps and pipe lines.

When the test was restarted on 26th August 2010, the jet pump was not able to reach a vacuum level over 34 kPa (Figure 4.35(b)). On the end of August and early September 2010, high levels of rain fell rising the ground water table (Figure 4.24 and Figure 4.34). The reduced capacity of the jet pump prevented the adequate evacuation of water from the vacuum chamber, allowing water to go into the liquid ring pump overloading the circuit and activating the breaker on 16th September 2010. When the pumping system was turned on again, the centrifugal pump and the liquid ring pump were stuck and unable to turn. The three pumps were opened and found to be covered with scale. The pumps were cleaned and reconnected.

Figure 4.78 shows the impeller and front cover of the liquid ring pump partly clean, where some of the chemical incrustation can still be observed. The pumping was restarted on 18th September 2010. The issue was repeated on 13th October 2010 and pumps restarted on 14th October 2010, on 26th October 2010 being restarted on the same date, and finally on 29th October 2010 when it was decided to stop the test.

Afterwards, the pumps, pipes, vacuum chamber and water samples from the main tank were brought to the Soils Mechanics Laboratory at Trinity College Dublin, and the incrustation from the pumps and pipes was removed (Figure 4.79). The pipework inside the vacuum chamber was found to have rusted significantly in three months of the second period of vacuum pumping (Figure 4.80). Chemical analysis was carried out on the solid and water samples retrieved from the pumps, pipes and main tank, in the Environmental Engineering Laboratory of Trinity College Dublin, on a Varian Liberty AX Series II ICP-AES. A certified multi-element reference standard solution (AccuTrace) was utilised. The ICP machine was calibrated with a maximum percentage error of 10%.

From the results in Table 4.19 and Table 4.20, it can be seen that calcium was the element with higher content in both the solid and water samples. Calcium carbonate incrustation is very commonly found and it has a characteristic white colour (Figure 4.79(f)), as found in this case in the jet pump (Figure 4.79(c)), centrifugal pump (Figure 4.79(d)) and vacuum
line (Figure 4.79(e)). Relatively high contents of manganese and iron were also found. Manganese was particularly high in the solid samples from the vacuum line, which may explain the intense reddish colour observed in Figure 4.77, and the colouring of the incrustation in the liquid ring pump (Figure 4.79(a) and Figure 4.79(b)).

Figure 4.78: Chemical incrustation inside the liquid ring pump on 17th September 2010: (a) impeller, (b) front cover.
The high concentrations of calcium found, can be explained as water was not only being extracted from the peat, but also from the gravel layer (see §4.3.2) which contains a substantial concentration of calcium minerals (Hammond, 1981). This also explains the high pH values determined for the water samples taken from the main tank (Table 4.19), despite the low pH values obtained for the peat (Figure 3.21(f)).
Figure 4.80: Rust found inside vacuum chamber after vacuum test.

Table 4.19: pH and chemical analysis result for water from the main tank.

<table>
<thead>
<tr>
<th>Test</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
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<tr>
<td>pH</td>
<td>8.32</td>
<td>8.30</td>
<td>8.30</td>
</tr>
<tr>
<td>Fe (mg/l)</td>
<td>0.003514</td>
<td>0.004243</td>
<td></td>
</tr>
<tr>
<td>Mg (mg/l)</td>
<td>12.524</td>
<td>12.652</td>
<td></td>
</tr>
<tr>
<td>Mn (mg/l)</td>
<td>0.07566</td>
<td>0.077785</td>
<td></td>
</tr>
<tr>
<td>Cl (mg/l)</td>
<td>0.0</td>
<td>0.0</td>
<td></td>
</tr>
<tr>
<td>K (mg/l)</td>
<td>2.7094</td>
<td>2.6909</td>
<td></td>
</tr>
<tr>
<td>Na (mg/l)</td>
<td>14.424</td>
<td>14.291</td>
<td></td>
</tr>
<tr>
<td>Ca (mg/l)</td>
<td>74.734</td>
<td>74.79</td>
<td></td>
</tr>
</tbody>
</table>
Table 4.20: Chemical analysis results for solid from pumps and pipes.

<table>
<thead>
<tr>
<th>TEST</th>
<th>Label</th>
<th>Fe (mg/kg)</th>
<th>K (mg/kg)</th>
<th>Mg (mg/kg)</th>
<th>Mn (mg/kg)</th>
<th>Ca (mg/kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Liquid ring old flakes A</td>
<td>380.94</td>
<td>51.5928</td>
<td>604.268</td>
<td>1758.96</td>
<td>&gt; 8400</td>
</tr>
<tr>
<td>2</td>
<td>Liquid ring old flakes B</td>
<td>340.508</td>
<td>42.7014</td>
<td>514.472</td>
<td>1525.3</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Liquid ring new flakes A</td>
<td>926.156</td>
<td>52.2088</td>
<td>630.518</td>
<td>1855.56</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Liquid ring new flakes B</td>
<td>870.996</td>
<td>45.5154</td>
<td>613.55</td>
<td>1818.32</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Plastic Pink pipe A</td>
<td>429.786</td>
<td>728.602</td>
<td>252.042</td>
<td>3538.26</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Eductor A</td>
<td>273.21</td>
<td>56.448</td>
<td>563.136</td>
<td>1782.48</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Eductor B</td>
<td>245.714</td>
<td>51.0832</td>
<td>541.884</td>
<td>1735.86</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>Centrifugal pump A</td>
<td>288.932</td>
<td>68.4992</td>
<td>549.906</td>
<td>1724.52</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Centrifugal pump B</td>
<td>261.433</td>
<td>65.8798</td>
<td>485.884</td>
<td>1547.14</td>
<td></td>
</tr>
</tbody>
</table>

4.6 IMPROVEMENT OF GROUND PROPERTIES

In vacuum consolidation, as in embankment surcharging projects, the improvement of the ground properties is normally evaluated using two criteria: the variation of the normalized shear strength \( \frac{S_v}{\sigma'_v} \) and the reduction of the post-construction settlements due to both, primary consolidation and secondary compression. These criteria are used for both inorganic and organic soils.

4.6.1 Variation of the normalized shear strength \( \frac{S_v}{\sigma'_v} \)

The shear strength of mineral and organic soils is related to the effective stress, thus any change in the effective stress changes the shear strength of the soil. The relationship between shear strength and effective stress is mainly controlled by the consolidation history of the soil (Stamatopoulos and Kotzias, 1985).

It has been established that, for normally consolidated soils, the shear strength is proportional to the effective stress \( \sigma'_v \), under which it has consolidated. Under hydrostatic conditions, the effective stress of normally consolidated soils varies linearly with depth, and the shear strength \( S_v \) must also vary linearly with depth: thus the ratio \( \frac{S_v}{\sigma'_v} \) is constant (Stamatopoulos and Kotzias, 1985). Furthermore, it has also been shown
that for both, normally consolidated deposits (YOUNG) and overconsolidated deposits (AGED), $S_u/\sigma'_v$ depends on the plasticity index (Figure 4.81). The $S_u/\sigma'_v$ ratio is higher for overconsolidated soils, by amounts varying with the overconsolidation ratio (Stamatopoulos and Kotzias, 1985; Lechowicz, 1996).

![Figure 4.81](image)

Figure 4.81: Dependence of $s_u/\sigma'_v$ on the plasticity index (Stamatopoulos and Kotzias, 1985).

In the case of organic soils, a similar dependence of $S_u/\sigma'_v$ with the organic content has been established (Figure 4.82). According to Lechowicz (1996), experience from organic soils indicates that the normalized undrained shear strength changes with the change in effective stresses for both, normally and overconsolidated soils.

![Figure 4.82](image)

Figure 4.82: Normalized undrained shear strength as a function of the organic content (Lechowicz, 1996).

The variation of the normalized shear strength ($S_u/\sigma'_v$) was used by Hayashi et al. (2002) to evaluate the increase of the shear strength of a peat soil, in a combined vacuum-surcharge consolidation project (see §2.2.4.4). Before the vacuum pressure was applied to the
ground, it was found that $S_t/\sigma_v = 0.4$ for the peat and clayey-peat layers, with organic contents of more than 20%. Afterwards, the $S_t/\sigma_v$ of the peaty layers to which the vacuum consolidation method was applied, showed a higher value than that of the peaty layers on sections where no vacuum improvement was applied, although no average value was mentioned.

Shang et al. (1998), also showed that the shear strength increments, in peat and peaty clay layers, achieved using vacuum consolidation are analogous as the ones obtained using embankment loading.

Based on the aforementioned researches, and considering that Ballydermot peat is nearly normally consolidated (see §3.4.3), the low shear strength values gather from the field vane tests (see §3.3.3) and the variation to which the peat was subjected during the vacuum consolidation field test (see Figure 4.46 and Figure 4.47), it can be inferred that there was an increase on the normalized shear strength ($S_t/\sigma_v$) of Ballydermot peat.

4.6.2 Reduction of the post-construction settlements

When postconstruction settlements due to both, primary consolidation and secondary compression, are to be minimised, surcharging is one of the most widely used methods for soil improvement. This method is frequently used in conjunction with the installation prefabricated of vertical drains in order to accelerate the consolidation process and to reduce the improvement period. Surcharging is considered a very useful technique of construction on most organic soils. The use of a heavy surcharge can be assumed to reduce the postconstruction settlements due to both primary consolidation and secondary compression (Wolski, 1996).

The most commonly used surcharge loads used are earth fills; however, any type of load that induces a reduction in the pore water pressures with the subsequent increase in the effective stresses, such as a vacuum load, can be used as a surcharge load (Mitchell, 1981).

Figure 4.83 shows, schematically, the use of a surcharge load to reduce future postconstruction settlements. As it can be seen, when a surcharge load is applied to the soil
deposit for a time $t_{cr}$, the layer will settle by an amount $S_{cr}$, which is higher than the expected settlement under the permanent load alone, thus reaching the expected average degree of consolidation faster (Mitchell, 1981; Wolski, 1996).

Figure 4.83: Surcharging to reduce primary consolidation and secondary compression (Wolski, 1996).

The effect of secondary compression is particularly important on the postconstruction settlement on organic material. Surcharging has proven effective for minimizing the effect of subsequent secondary compression under permanent loads on peat soil (Samson, 1985). Schematically, the soil is allowed to settle under the permanent load plus the surcharge (Figure 4.83); then, after the surcharge is removed, there is a rebound effect followed by a slow rate settlement due to secondary compression. Subsequently, after the permanent plus surcharge loading curve (continuous line) reaches the permanent loading curve (dashed line), the settlements return to the normal rate of creep (Wolski, 1996; Farrell, in press). This effect, delays the normal rate of long-term settlements on the soil. Samson (1985) reported that, for an expressway built on peat using the surcharge method, the effect on the postconstruction settlements and on the reduce rate of long-term settlement was felt for a period or 5.5 to 8 years, depending on the section studied.
Authors such as Cognon et al. (1994) and Shiono et al. (2001), have showed that the effect on the settlements and effective stresses attained by surcharging peat deposits using vacuum pressures, is nearly equal to that attained by surcharging using embankment fills.

Considering the settlements achieved (due to primary consolidation and secondary compression) and the degree of consolidation obtained during the field test (see §4.3.2), it can be inferred that if an embankment is built over the treated area, the postconstruction settlement would be significantly reduced. However, it is important to note, that the reduction of this future settlements would be significant if the load to be applied by the embankment is equal or lower than the average vacuum load attained during treatment.

4.6.3 Benefits for highway engineering

This effect would be highly beneficial to highway engineers, since an increase on the shear strength of the soil will translate in an improvement of the stability of the embankments required for the bog roads. Moreover, as it was shown from different field applications (see §2.2.4) that there was not only an increase in the stability of the road embankments, but also an increase in the height and construction speed on different cases.

4.7 SUMMARY AND DISCUSSION

The TCD/NRA vacuum consolidation field test was successfully implemented in a 10 m area at Ballydermot bog, for a period of 11 months, from 30th November 2009 to 29th October 2010. In order to evaluate how the difference in the spacing of the PVDs affects the improvement method, the test area was subdivided in two, one in which the spacing of the drains was 0.85 m and a second one with spacing of 1.20 m, both in a square grid. A permeable granular bed with a system of horizontal perforated flexible pipes embedded within was placed on the ground surface. A 1.0 m deep trench was dug around the area, and an airtight membrane was used to cover the granular bed and the trenches.

An instrumentation system was designed to measure the settlement at different depths, positive and negative pore water pressures at different depths, barometric pressure, surface and ground temperatures, water flow, ground water table and rainfall. The instrumentation
system, for the vacuum pumping stage, included six push-in vibrating wire settlement
cells, ten push-in vibrating wire piezometers (calibrated for both positive and negative
pressures), a surface barometer/thermometer, 26 surface settlement plates, 17 stand pipes
to monitor the ground water table level, a water meter and a rain gauge.

Two different vacuum generating systems were used during the project. The first system
used a jet pump as the vacuum creating device, while the second system used a liquid ring
pump for this purpose.

Two separate stages of monitoring were conducted throughout the project. The first stage
covers the period prior to the start of the pumping stage, including test construction and
setting the baseline of the instruments. The second stage covers the eleven months
pumping period.

From the initial monitoring it was observed that the water table levels and the ground
vertical displacements show a seasonal variation, and are particularly dependant on the
rainfall intensity.

The pore water pressure readings showed that the clayey silty sandy gravel underlying the
peat, acts as a drainage permeable layer, inducing a downwards flow reducing the pore
water pressures and increasing the effective stress of the deeper peat layers. This increased
effective stress also increases the shear strength, thus explaining the higher values
observed in the field shear vane results at the bottom of the peat (see §3.3.3), and the
problems when trying to sample during the manual boreholes from this layer (see §3.3.1).

During the pumping stage, it was observed that higher vacuum pressure values, up to 80
kPa, were obtained when using the jet pump; however, a more stable pressure was
maintained when using the liquid ring pump, with vacuum pressures up to 71 kPa.

The settlement for both spacing subareas followed each other closely during the entire
testing period at all depths. A maximum surface settlement of 1.23 m was reached at the
end of the project, with most of the settlement occurring during the first month of
pumping. The surface settlement profiles showed that higher values of settlement occurred
in the centre of the test area and that towards the edges smaller settlements are achieved, reaching a value of 0.91 m at the plate with the smallest recorded settlement. It is important to note that the pore water pressure reduction on the edge of the test area was slightly smaller than the reduction achieved at the centre of the area. In some cases the difference was as low as 2 kPa, however in some cases it was up to 8.5 kPa. The bigger differences occurred during the summer months when the water table levels around the test area were lower, probably compromising the airtight seal provided by the water.

The pore water pressure measurements showed that the ΔPWP is somewhat uniform along the entire peat deposit, for both PVD spacing areas, with a significantly lower reduction when the gravel layer is reached.

From the piezometric head measurements, it was concluded that the water was not only being extracted from the peat deposit, but also from the gravel layer. This is further confirmed by the volume of water measured by the water meter at the end of the vacuum generating system; and indirectly confirmed by the calcium incrustation observed in the pumps and pipes, and the basic pH values measured in the main tank water samples.

In general, the settlement and PWP behaviour in the two different PVD spacing areas was very similar through the entire project, indicating that the vacuum pressure is adequately transmitted for both, 0.85 and 1.20 m spacing of the PVD.

Cracks appeared around the test area during phases where the vacuum pressures measured at the bog surface were over 50 kPa continuously for periods of time exceeding two weeks. The cracking indicates that lateral inward displacement was occurring in the test area.

Some technical difficulties appeared during the pumping stage, reducing the vacuum levels or even sometimes fully stopping the pumping process. The difficulties included pipe leakage or bursting, water freezing in the vacuum line during winter time, airtight membrane piercing or tearing, ground water table variations, electrical cuts at site and chemical incrustation on the pipes and pumps.

From these difficulties, the water freezing in the vacuum line and the chemical incrustation in the pumps and pipes had the worst effect in the overall performance of the pumps, as
these processes fully stopped them from running, for weeks at a time. The membrane piercing and the water table variation also had a significant effect in the vacuum pressure, although some level of vacuum was always present in the line.

In order to prevent water freezing in the lines, it is recommended to use thermal protection in the hoses and pipes exposed to the environment during the winter months. To reduce the effect that chemical incrustation has over the pumps, it is recommended to use a system of parallel pumps, in which one pumping system can be turned off for maintenance purposes on monthly basis, while the pumping process is switched to the second pumping system in place, and vice versa.

Due to the piercing and tearing observed in the membrane utilised, it is recommended to use at least two membranes as airtight cover, with the purposes of preventing any piercing or tearing from occurring at all, as finding and fixing any holes during the pumping process has proven to be extremely difficult.

The trenches around the test area should be excavated at least 0.5 m beneath the lowest water table level, considering the seasonal variations that might occur, so as to prevent any vacuum pressure reduction due to the loss of the seal around the area.

The field test was back–analysed using the SS and SSC models from PLAXIS version 2010. The design parameters were determined by means of conventional oedometer tests. Acceptable results were obtained for the ground vertical displacements, although the models seem to overestimate the heave. The SSC model tends to produce higher values of compression as it considers the effect of creep.
5. CONCLUSIONS

5.1 INTRODUCTION

A vacuum consolidation field test was designed and successfully implemented at Ballydermot bog, in order to investigate the field performance of vacuum consolidation in peat deposits, and to evaluate the viability of implementing this technique as a method for the construction and improvement of roads over peat in Ireland. The use of traditional laboratory tests to determine the parameters for the design of vacuum consolidation projects in peat was also investigated. The performance of two different finite element models to simulate the observed behaviour of the peat, using the parameter determined in the laboratory, was also assessed.

A field and laboratory investigation was conducted on the geotechnical properties of Ballydermot peat. The field investigation comprised two manual boreholes using spiral and gouge augers, two cable percussion boreholes and three field shear vane tests. The laboratory investigation included classification tests and nine oedometer tests.

Based on the field and the laboratory investigation, the soil profile of the test area at Ballydermot bog is mainly composed of three layers: (i) a 0.8 m thick man-made fill, (ii) a 3.2 m pseudo-fibrous peat layer, and (iii) a 1.9 m gravel layer with very high contents of fines and sand, reducing with depth.

Due to the extensive drainage for harvesting purposes, the peat layer at Ballydermot bog has undergone a continuous settlement process over the last 60 to 70 years. The peat depth in 1935 was measured to be 8.7 m, and to date approximately only 3.2 to 4.0 m remains.

This chapter presents the main conclusions from the laboratory investigation and the vacuum consolidation field tests. Finally, recommendations for future work are given.
5.2 LABORATORY INVESTIGATION

- A statistical study has shown that there is no significant difference on the water content values determined for peat under temperatures of 80°C and 105°C, and that the differences can be attributed to chance variation. It can then be concluded that for engineering purposes a temperature of 105°C is acceptable.

- The relationship between the loss on ignition and specific gravity, proposed by Skempton and Petley (1970), has been shown to fit the measured values for Ballydermot peat.

- The coefficient of permeability, $C_k$, follows a linear relationship in the semi-logarithmic space $e - \log k$. The $C_k$ values determined using both Casagrande and Taylor graphical constructions were very similar between them. The relationship $C_k = 0.25e_o$ suggested by Mesri et al. (1997) for peat was found to be in good agreement with the results presented here.

- The End of Primary approach adequately modelled the stress – strain behaviour of Ballydermot peat.

- Janbu’s resistance concept adequately described the stress – strain behaviour of all tests, and the strain – time behaviour of most load increments. However, the long term tests did not behave linearly as expected, but after certain time the linearity of the curve was distorted showing scatter behaviour. This behaviour may be explained by the tertiary compression observed in peat soil, due to the decay of the organic matter, however further research is needed.

- The creep resistance number, $r$, of peat seems to be dependent on the stress level as it does in clay.

- The Soft Soil and the Soft Soil Creep models correctly depicted the stress – strain behaviour shown by Ballydermot peat on the oedometer tests. The strain – time behaviour was adequately described by the SSC model, but not by the SS model as
Chapter 5 CONCLUSIONS

this model does not consider creep. However this last one was able to determine the final vertical strain value.

- Due to the consolidation process induced by the drainage of Ballydermot bog, the preconsolidation pressure of the peat was found to be higher when compared to that of other bogs in the Irish midlands.

5.3 TCD/NRA VACUUM CONSOLIDATION FIELD TEST

- Based on rainfall, water table and settlement measurements, it was observed that the water table levels and the ground vertical displacements show a seasonal variation, and are particularly dependant on the rainfall intensity.

- The clayey silty sandy gravel underlying the peat, acts as a drainage permeable layer, inducing a downwards flow reducing the pore water pressures and increasing the effective stress of the deeper peat layers, thus increasing the effective stress and the shear strength as it was observed in the field shear vane results.

- The vacuum consolidation method was successfully implemented at Ballydermot bog, showing that the method can be used as a ground improvement technique on peat soils for practical applications such as road improvements.

- In general, the settlement and pore water pressure behaviour in the two different PVD spacing areas was very similar through the entire project, indicating that the vacuum pressure is adequately transmitted for both PVD sections.

- Two vacuum generating systems were tested during the project. The first system used a jet pump to create the vacuum, reaching vacuum pressures up to 80 kPa, though in an intermittent manner. The second system used a liquid ring pump as the main vacuum generator, and a more stable pressure was maintained, with vacuum pressures up to 71 kPa.
Chapter 5

CONCLUSIONS

- The granular bed, flexible pipes and prefabricated vertical drains were effective in transmitting most of the vacuum pressure to the peat layer, and inducing a uniform reduction in the pore water pressure at all depths, for both spacing subareas, even beneath the bottom of the PVDs.

- The pore water pressure reduction observed on the edge of the test area was similar to that observed at the centre of the area, though slightly smaller during most of the pumping period. The bigger differences occurred during the summer months when the water table levels around the test area were lower, probably compromising the airtight seal provided by the water.

- The pore water pressure reduction in the gravel layer was significantly lower compared to the reduction observed in the peat, showing that the gravel acts as a permeable layer through which vacuum pressure is lost.

- The vacuum pumping did not have any significant effect in the water table and surface levels outside the test area, as there was no appreciable settlement of the surface and the water table varied with the rainfall rather than with the vacuum variations. Furthermore, the pore water pressures measurements showed that no vacuum pressure was transmitted to the peat outside the treated area.

- The variations of water table level influenced the vacuum level as the water acts as a seal around the test area, preventing air from coming into the peat deposit. It was observed that during the drier seasons when the water table dropped, the vacuum pressure was reduced; and that after rainy periods and rises in the water table, the vacuum pressure increased.

- The trenches around the test area should be excavated at least 0.5 m beneath the lowest water table level, considering the seasonal variations that might occur, so as to prevent any vacuum pressure reduction due to the loss of the seal around the area.
Chapter 5 CONCLUSIONS

- Flow of water from the gravel layer underlying the peat was observed from the piezometric head measurements. This was further confirmed by the volume of water measured by the water meter, and indirectly confirmed by the calcium incrustation observed in the pumps and pipes, and the basic pH values measured in the main tank water samples.

- A high rate of settlement was observed initially, as approximately half the settlement of the entire project was induced on the first two weeks, and two thirds of it on the first month. However, due to the vacuum pressure becoming intermittent for six months, the rate of settlement was reduced. After the second vacuum generating system was in place, the settlement rate increased again, though not the level of the first month.

- Despite the difference in the spacing of the prefabricated vertical drains, the settlement of both subareas followed each other closely during the entire testing period at all depths.

- Significant cracking appeared around three sides of the test area, indicating lateral inward displacement was occurring. No cracking was observed on the north side as traffic of heavy machinery was continually passing.

- Water freezing in the vacuum line affected the overall performance of the pumping systems by reducing vacuum pressures, or even fully preventing the vacuum transmission to the granular bed. In order to prevent water freezing in the lines, it is recommended to use thermal protection in the hoses and pipes exposed to the environment during the winter months.

- Chemical incrustation was observed in the pumps and pipes, also affecting the performance of the pumps, even fully stopping them from running. To reduce the effect that chemical incrustation has over the pumps, it is recommended to use a system of parallel pumps, in which one pumping system can be turned off for maintenance purposes on monthly basis, while the pumping process is switched to the second pumping system, and vice versa.
Due to the piercing and tearing observed in the membrane utilised, it is recommended to use at least two membranes as airtight cover, as finding and fixing any holes during the pumping process has proven to be extremely difficult.

Acceptable results were obtained by using the Soft Soil and the Soft Soil Creep to back-analyse the strain – time behaviour shown by the peat during the field test, although the models seem to overestimate the heave. The SSC model tends to produce higher values of compression as it considers the effect of creep.

The design parameters for the finite element analysis were determined by means of conventional oedometer tests. Hence, the geotechnical properties of the peat obtained in conventional tests can be used in the design of vacuum consolidation projects.

The pore water pressures calculated using the SS and SSC models did not followed the behaviour shown by the measured values. This is because of the variation of the total stress introduced by the surcharge load applied instantaneously on the surface, to artificially increase the pore water pressure to let them dissipate afterwards.

5.4 RECOMMENDATIONS FOR FUTURE RESEARCH

A significant amount of data has been collected during the vacuum consolidation field test. This data can be used in the development of more realistic analytical and numerical models, which incorporates the vacuum transmission mechanism along the PVD and in the ground.

Field and laboratory tests should be conducted using instrumented PVDs of different properties to investigate how this would affect the vacuum transmission mechanism.

The vacuum generating systems should be further investigated with the purpose of improving their efficiency.
• A laboratory apparatus should be developed, in order to investigate the behaviour of peat under a more controlled environment.

• The effect of secondary compression of peat under vacuum consolidation should be investigated in both the field and the laboratory.
REFERENCES


REFERENCES


REFERENCES


REFERENCES


REFERENCES


REFERENCES


REFERENCES


REFERENCES


REFERENCES


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APPENDIX A: FIELD AND LABORATORY INVESTIGATION RESULTS
APPENDIX A: FIELD AND LABORATORY INVESTIGATION RESULTS
Figure A.1: Daily rainfall measurement before vacuum pumping, for all stations.

Figure A.2: Daily rainfall measurement during and after vacuum pumping, for all stations.
Appendix A
FIELD AND LABORATORY INVESTIGATION RESULTS

Figure A.3: Borehole 1 log sheet.

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**Appendix A**

**FIELD AND LABORATORY INVESTIGATION RESULTS**

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**Project Name:** Vacuum Consolidation Field Trial, Ballydermot

**Hole ID:** BH2

**Client:** Trinity College

**Location:** Ballydermot

**Start date:** 10/03/2010 **End date:** 11/03/2010

**Type of drilling:** CP **Hole diameter:** 200 mm

**Co-ordinates:**

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Type</th>
<th>Samples / tests</th>
<th>Result</th>
<th>Water Depth</th>
</tr>
</thead>
</table>

**Strata Description**

- **Peat FILL**
  - 0.90 m: B 0.90
  - 0.90 m: P 1.00

- **PEAT**
  - 0.90 m: B 1.90
  - 1.00 m: P 2.00

- **Stiff grey slightly sandy gravelly CLAY**
  - 1.90 m: B 2.80
  - 2.80 m: P 3.20
  - 3.20 m: B 3.50

- **Dense grey claybound GRAVEL**
  - 5.00 m: SPT C 5.30

- **OBSTRUCTION - rock fragments - possible bedrock**
  - 5.80 m: SPT C 5.80
  - 5.80 m: B 5.80
  - 6.00 m: U 6.00

- **End of Borehole at 7.00 m**

**Remarks:**

All descriptions based on visual observations
Groundwater end day 15th March 1:00m BGL
Groundwater start day 17th March 4:00m BGL
Groundwater end 11th March 5:00m BGL

**KEY**

- **SPT:** Standard Penetration Test
- **U:** Unit samples
- **C:** Core samples
- **B:** Bulk disturbed sample
- **S:** Small disturbed sample
- **T:** Test split spoor

**Figure A.4:** Borehole 2 log sheet
Appendix A  FIELD AND LABORATORY INVESTIGATION RESULTS

![Graphs showing particle size distribution and plasticity chart for BH-1](image)

**Figure A.5:** Mineral soils found in BH-1 (a) particle size distribution, (b) plasticity chart.
Appendix A  FIELD AND LABORATORY INVESTIGATION RESULTS

Figure A.6: Mineral soils found in BH-2 (a) particle size distribution, (b) plasticity chart.
Figure A.7: $e - \log \sigma'$ curve and EOP model for BH-1 Oed 2 $D = 0.90$ to 1.55 m.

Figure A.8: $e - \log \sigma'$ curve and EOP model for BH-1 Oed 4 $D = 0.90$ to 1.55 m.
Figure A.9: $e - \log \sigma'$ curve and EOP model for BH-1 Oed 1 $D = 1.50 - 2.30$ m.

Figure A.10: $e - \log \sigma'$ curve and EOP model for BH-1 Oed 4 $D = 1.50 - 2.30$ m.
Figure A.11: \( e - \log \sigma' \) curve and EOP model for BH-1 Oed 2 \( D = 2.40 - 3.20 \) m.

Figure A.12: \( e - \log \sigma' \) curve and EOP model for BH-1 Oed 4 \( D = 2.40 - 3.20 \) m.
# Appendix A

## FIELD AND LABORATORY INVESTIGATION RESULTS

### Table A.1: Compression - time behaviour parameters for Oedometer 1 D = 0.90 - 1.55 m.

<table>
<thead>
<tr>
<th>Stress (kPa)</th>
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<th>$m_e$ (m$^3$/MN)</th>
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<th>$t_{1000}$ (min)</th>
<th>$c_{v-e}$ (m$^3$/year)</th>
<th>$c_{v-t}$ (m$^3$/year)</th>
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<th>$r_t$</th>
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### Table A.2: Compression - time behaviour parameters for Oedometer 2 D = 0.90 - 1.55 m.

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270
Table A.3: Compression - time behaviour parameters for Oedometer 4 D = 0.90 - 1.55 m.

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<th>Stress</th>
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Table A.4: Compression - time behaviour parameters for Oedometer 1 D = 1.50 – 2.30 m.

<table>
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Table A.5: Compression - time behaviour parameters for Oedometer 2 D = 1.50 – 2.30 m.

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<th>$c_{ve}$ (m$^3$/year)</th>
<th>$k_e$ (m/s)</th>
<th>$k_i$ (m/s)</th>
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Table A.6: Compression - time behaviour parameters for Oedometer 4 D = 1.50 – 2.30 m.

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<th>$t_{100,e}$ (min)</th>
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<th>$c_{ve}$ (m$^3$/year)</th>
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<th>$k_i$ (m/s)</th>
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### Appendix A

#### FIELD AND LABORATORY INVESTIGATION RESULTS

Table A.7: Compression - lime behaviour parameters for Oedometer 1 D = 2.40 – 3.20 m.

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Table A.8: Compression - time behaviour parameters for Oedometer 2 D = 2.40 – 3.20 m.

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### Table A.9: Compression - time behaviour parameters for Oedometer 4 D = 2.40 – 3.20 m.

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Figure A.13: BH-1 Oed 1 D = 0.90 to 1.55 m: (a) $e - \log t$, and (b) $\Delta H - \sqrt{t}$.
Figure A.14: BH-1 Oed 2 D = 0.90 to 1.55 m: (a) $e - \log t$, and (b) $\Delta H - \sqrt{t}$.
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Figure A.15: BH-1 Oed 4 D = 0.90 to 1.55 m: (a) $e - \log t$, and (b) $\Delta H - \sqrt{t}$. 

[Diagram showing void ratio ($e$) vs. time (min) and root time (min) vs. $\Delta H$ for different pressures.]
Figure A.16: BH-1 Oed 1 D = 1.50 to 2.30 m: (a) $e - \log t$, and (b) $\Delta H - \sqrt{t}$. 
Figure A.17: BH-1 Oed 2 D = 1.50 to 2.30 m: (a) $e - \log t$, and (b) $\Delta H - \sqrt{t}$. 
Figure A.18: BH-I Oed 4 D = 1.50 to 2.30 m: (a) $e - \log t$, and (b) $\Delta H - \sqrt{t}$. 
Figure A.19: BH-1 Oed 1 D = 2.40 to 3.20 m: (a) $e - \log t$, and (b) $\Delta H - \sqrt{t}$. 
Figure A.20: BH-1 Oed 2 D = 2.40 to 3.20 m: (a) $e = \log t$, and (b) $\Delta H = \sqrt{t}$. 

*Ho = 18.40 mm*
Figure A.21: BH-1 Oed 4 D = 2.40 to 3.20 m: (a) $e - \log t$, and (b) $\Delta H - \sqrt{t}$. 

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Figure A.22: BH-1 Ocd 2 D = 0.90 to 1.55 m: (a) $\varepsilon - \sigma'_v$, and (b) $M - \sigma'_v$. 

$m = 5.95$

$R^2 = 0.9738$
Figure A.23: BH-1 Oed 4 D = 0.90 to 1.55 m: (a) $\varepsilon - \sigma'_v$, and (b) $M - \sigma'_v$. 

|$m$ = 5.86  
$R^2$ = 0.9684
Figure A.24: BH-1 Oed 1 D = 1.50 to 2.30 m: (a) $\varepsilon - \sigma''$, and (b) $M - \sigma''$. 

\[ m = 5.30 \]
\[ R^2 = 0.9969 \]
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Figure A.25: BH-I Oed 4 D = 1.50 to 2.30 m: (a) $\varepsilon - \sigma'_v$, and (b) $M - \sigma'_v$. 

(a)

(b)

$\sigma'_v$ (kPa)

$\varepsilon$

$M$

$\sigma'_v$ (kPa)

Vertical strain,

Vertical effective stress, 

$m = 4.52$

$R^2 = 0.9862$
Figure A.26: BH-1 Oed 2 D = 2.30 to 3.20 m: (a) $\varepsilon - \sigma''$, and (b) $M - \sigma''$. 

$M = 4.51$ 
$R^2 = 0.9918$
Figure A.27: BH1-1 Oed 4 D = 2.30 to 3.20 m: (a) $\varepsilon - \sigma''$, and (b) $M - \sigma''$. 

\[ m = 4.63 \]
\[ R^2 = 0.9997 \]
Figure A.28: BH-1 Oed 1 D = 0.90 to 1.55 m, $\sigma'_v$ = 50 kPa: (a) $e - t$, and (b) $R - t$. 

$r_v = 82.10$
$R^2 = 0.8739$
Figure A.29: BH-1 Oed value \( D = 0.90 \) to 1.55 m, \( \sigma'_v = 6.25 \) kPa: (a) \( \varepsilon - t \), and (b) \( R - t \).
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Figure A.30: BH-1 Ocd 2 D = 0.90 to 1.55 m, $\sigma' = 12.5$ kPa: (a) $\varepsilon - t$, and (b) $R - t$. 

$R = 188.15$  
$R^2 = 0.9902$
Figure A.31: BH-1 Oed 2 D = 0.90 to 1.55 m, $\sigma'_v = 25$ kPa: (a) $\varepsilon - t$, and (b) $R - t$. 

\[ r_v = 107.63 \]
\[ R^2 = 0.9999 \]
Figure A.32: BH-1 Oed 2 D = 0.90 to 1.55 m, $\sigma'_v = 50$ kPa: (a) $e - t$, and (b) $R - t$. 
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Figure A.33: BH-1 Oed 2 D = 0.90 to 1.55 m, $\sigma'_c = 100$ kPa: (a) $\varepsilon - t$, and (b) $R - t$. 

(a)

(b)
Figure A.34: BH-1 Ocd 2 D = 0.90 to 1.55 m, $\sigma_{v} = 200$ kPa: (a) $\varepsilon - t$, and (b) $R - t$. 

$\tau_{c} = 97.09$
$R^{2} = 0.9946$
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Figure A.35: BH-1 Oed 4 D = 0.90 to 1.55 m. \( \sigma'_v = 6.25 \text{ kPa} \): (a) \( \varepsilon - t \), and (b) \( R - t \).
Figure A.36: BH-1 Oed D = 0.90 to 1.55 m, $\sigma' = 12.5$ kPa: (a) $\varepsilon - t$, and (b) $R - t$. 

\[ r_s = 119.86 \]
\[ R^2 = 0.9878 \]
Figure A.37: BH-1 Ocd 4 D = 0.90 to 1.55 m, $\sigma' = 25$ kPa: (a) $\varepsilon - t$, and (b) $R - t$. 
Figure A.38: BH-1 Oed 4 D = 0.90 to 1.55 m, $\sigma'_{vd} = 50$ kPa: (a) $\varepsilon - t$, and (b) $R - t$. 

$$r_s = 83.02$$

$$R^2 = 0.9895$$
Figure A.39: BH-1 Oed 4 $D = 0.90$ to 1.55 m, $\sigma'_v = 100$ kPa: (a) $\varepsilon - t$, and (b) $R - t$. 

- (a) 
- (b) 

$R = 101.96$ 
$R^2 = 0.9944$
Figure A.40: BH-1 Oed 4 D = 0.90 to 1.55 m, $\sigma' = 200$ kPa: (a) $\varepsilon - t$, and (b) $R - t$. 
Figure A.41: BH-1 Oed 1 D = 1.50 to 2.30 m, $\sigma'_v = 12.5$ kPa: (a) $\varepsilon - t$, and (b) $R - t$. 

\[ r_s = 106.36 \]
\[ R^2 = 0.9979 \]
Figure A.42: BII-1 Oed 1 D = 1.50 to 2.30 m, $\sigma'_v = 25$ kPa: (a) $e - t$, and (b) $R - t$. 
Figure A.43: BH-1 Oed 1 D = 1.50 to 2.30 m, $\sigma'_v = 50$ kPa: (a) $\varepsilon - t$, and (b) $R - t$. 

\[ r_s = 79.86 \]
\[ R^2 = 0.9791 \]
Figure A.44: BH-1 Ocd 1 D = 1.50 to 2.30 m, $\sigma'_{v0} = 100$ kPa: (a) $\varepsilon - t$, and (b) $R - t$. 
Figure A.45: BH-1 Oed 1 D = 1.50 to 2.30 m, $\sigma'_v = 200$ kPa: (a) $\varepsilon - t$, and (b) $R - t$. 
Figure A.46: BH-1 Oed 2 D = 1.50 to 2.30 m, $\sigma'_v = 50$ kPa: (a) $\varepsilon - t$, and (b) $R - t$. 

$r_1 = 85.79$

$R^2 = 0.7296$
Figure A.47: BII-4a D = 1.50 to 2.30 m, \( \sigma'_v = 6.25 \text{ kPa} \): (a) \( \varepsilon - t \), and (b) \( R - t \).
Figure A.48: BH-1 Oed 4 D = 1.50 to 2.30 m, $\sigma'_v = 12.5$ kPa: (a) $e-t$, and (b) $R-t$. 

$r_e = 73.96$

$R^2 = 0.9835$
Figure A.49: BH-1 Oed 4 D = 1.50 to 2.30 m, $\sigma' = 25$ kPa: (a) $e - t$, and (b) $R - t$. 

(a) 

(b) 

$r^2 = 0.9787$
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(a)  
Vertical strain, \( \varepsilon \) vs. time (min)

(b)  

\[ \sigma' = 84.15 \]
\[ R^2 = 0.9836 \]

Figure A.50: BH-1 Oed 4 D = 1.50 to 2.30 m, \( \sigma' \), = 50 kPa: (a) \( \varepsilon \) vs. \( t \), and (b) \( R \) vs. \( t \).
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Figure A.51: BHI-1 Oed D = 1.50 to 2.30 m, \( \sigma' \), = 100 kPa: (a) \( \varepsilon - t \), and (b) \( R - t \).
Figure A.52: BH-I Oed 4 D = 1.50 to 2.30 m, $\sigma'_{vd} = 200$ kPa: (a) $\varepsilon - t$, and (b) $R - t$. 
Figure A.53: BH-1 Oed 1 D = 2.40 to 3.20 m, $\sigma'_v = 50$ kPa: (a) $\varepsilon - t$, and (b) $R - t$. 
Figure A.54: BH-I Ocd 2 D = 2.40 to 3.20 m, $\sigma'_v = 6.25$ kPa: (a) $\varepsilon - t$, and (b) $R - t$. 

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Figure A.55: BH-1 Oed 2 D = 2.40 to 3.20 m, $\sigma'_v = 12.5$ kPa: (a) $\varepsilon - t$, and (b) $R - t$. 
Figure A.56: BH-1 Oed 2 D = 2.40 to 3.20 m, \( \sigma'_{\gamma} = 25 \) kPa: (a) \( \varepsilon - t \), and (b) \( R - t \).
Figure A.57: BH-1 Oed 2 D = 2.40 to 3.20 m, $\sigma'_v = 50$ kPa: (a) $\varepsilon - t$, and (b) $R - t$. 

$r_s = 136.52$

$R^2 = 0.9842$
Figure A.58: B11-1 Oed 2 D = 2.40 to 3.20 m, $\sigma'_v = 100$ kPa: (a) $\varepsilon - t$, and (b) $R - t$. 

$r_s = 101.72$

$R^2 = 0.9935$
Figure A.59: BH-1 Oed 2 D = 2.40 to 3.20 m, $\sigma'_v = 200$ kPa: (a) $\varepsilon - t$, and (b) $R - t$. 

$R_c = 131.39$

$R^2 = 0.9814$
Figure A.60: BHI-1 Oed 4 D = 2.40 to 3.20 m, $\sigma'_{v} = 6.25$ kPa: (a) $\varepsilon - t$, and (b) $R - t$. 

$r = 279.54$

$R^2 = 0.9916$
Figure A.61: BH-1 Oed 4 D = 2.40 to 3.20 m, $\sigma'_v = 12.5$ kPa: (a) $\varepsilon - t$, and (b) $R - t$. 

- $r_c = 132.29$
- $R^2 = 0.9995$
Appendix A  FIELD AND LABORATORY INVESTIGATION RESULTS

Figure A.62: BH-1 Oed 4 D = 2.40 to 3.20 m, $\sigma' = 25$ kPa: (a) $e - t$, and (b) $R - t$. 
Figure A.63: BH-1 Oed 4 D = 2.40 to 3.20 m, $\sigma_{\prime}^e = 50$ kPa: (a) $e - t$, and (b) $R - t$. 

\[ r = 99.21 \]
\[ R^2 = 0.9836 \]
Figure A.64: BH-1 Ocd 4 D = 2.40 to 3.20 m, \( \sigma_L = 100 \) kPa: (a) \( \varepsilon - t \), and (b) \( R - t \).
Figure A.65: BH-1 Oed 4 D = 2.40 to 3.20 m, $\sigma'_c = 200$ kPa: (a) $\varepsilon - t$, and (b) $R - t$. 
Appendix A
FIELD AND LABORATORY INVESTIGATION RESULTS

Figure A.66: $e - \ln t$ curve for BH-1 Oed 1 $D = 0.90$ to 1.55 m.

Figure A.67: $e - \ln t$ curve for BH-1 Oed 2 $D = 0.90$ to 1.55 m.
Appendix A

FIELD AND LABORATORY INVESTIGATION RESULTS

Figure A.68: $\varepsilon - \ln t$ curve for BH-1 Oed 4 D = 0.90 to 1.55 m.

Figure A.69: $\varepsilon - \ln t$ curve for BH-1 Oed 1 D = 1.50 to 2.30 m.
Figure A.70: $\varepsilon - \ln t$ curve for BH-1 Oed 2 $D = 1.50$ to $2.30$ m.

Figure A.71: $\varepsilon - \ln t$ curve for BH-1 Oed 4 $D = 1.50$ to $2.30$ m.
Appendix A  FIELD AND LABORATORY INVESTIGATION RESULTS

Figure A.72: $\varepsilon - ln t$ curve for BH-1 Oed 1 $D = 2.40$ to $3.20$ m.

Figure A.73: $\varepsilon - ln t$ curve for BH-1 Oed 2 $D = 2.40$ to $3.20$ m.
Figure A.74: $\varepsilon - \ln t$ curve for BH-1 Oed 4 D = 2.40 to 3.20 m.

Figure A.75: Comparison of test results with SS and SSC models for BH-1 Oed 1 D = 0.90 to 1.55 m.
Figure A.76: Comparison of test results with SS and SSC models for BH-1 Oed 2 D = 0.90 to 1.55 m: (a) $e - \log \sigma'$, and (b) $e - t$. 
Figure A.77: Comparison of test results with SS and SSC models for BH-1 Oed 4 D = 0.90 to 1.55 m: (a) $\varepsilon - \log \sigma^\prime$, and (b) $\varepsilon - t$. 
Figure A.78: Comparison of test results with SS and SSC models for BH-I Oed 1 D = 1.50 to 2.30 m: (a) $\varepsilon - \log \sigma'$, and (b) $\varepsilon - t$. 
Figure A.79: Comparison of test results with SS and SSC models for BH-1 Oed 2 \( D = 1.50 \) to 2.30 m
Figure A.80: Comparison of test results with SS and SSC models for BH-1 Oed 4 D = 1.50 to 2.30 m: (a) $\varepsilon - \log \sigma_{ve}$, and (b) $\varepsilon - t$. 
Figure A.81: Comparison of test results with SS and SSC models for BH-1 Oed I D = 2.40 to 3.20 m
Figure A.82: Comparison of test results with SS and SSC models for BH-1 Oed 2 D = 2.40 to 3.20 m: (a) $\varepsilon - \log \sigma'_v$, and (b) $\varepsilon - t$. 
Figure A.83: Comparison of test results with SS and SSC models for BH-1 Oed 4 D = 2.40 to 3.20 m: (a) $\varepsilon - \log \sigma'$, and (b) $\varepsilon - t$. 

(a) 

(b)
APPENDIX B: STRESS PATHS IN VACUUM CONSOLIDATION

B.1 STRESS PATHS UNDER EMBANKMENT LOADING

Historically, the evaluation of the rate of pore water pressure dissipation and of the magnitude of settlement under surcharge loading has been based on the assumption of oedometric conditions, i.e. one-dimensional compression (Hight et al., 1987). In the oedometer tests, due to the confining ring the net lateral strain is zero, and with a $B = 1$ for a fully saturated soil, the initial excess pore water pressure ($\Delta u_0$) is equal to the increase in total vertical stress ($\Delta \sigma_v$) (Craig, 2004).

$$\Delta u_0 = B \Delta \sigma_v$$  \hspace{1cm} (B.1)

However, in practice, oedometric conditions would only be applicable in the case of a wide embankment over a thin compressible layer, such as Case 1 in Figure B.1(a). In Figure B.1(b), a-b1-c1 is the effective stress path under one-dimensional consolidation.

In cases where the zero lateral strain condition is not satisfied, immediately after loading there is a reduction in the horizontal effective stress ($\sigma_h'$) due to the initial excess pore water pressure ($\Delta u_0$) being greater than the total horizontal stress increment ($\Delta \sigma_h$), and thus lateral expansion will occur (Craig, 2004). In Figure B.1(b), a-b2-c2 is the effective stress path for in-situ loading and consolidation, where a-b2 represents an immediate change of stress after loading and b2-c2 a gradual change of stress as the excess pore water pressure dissipates.
B.2 STRESS PATHS UNDER VACUUM LOADING

Many publications incorrectly indicate that, since in vacuum consolidation the pore water pressure is decreased without variation of the total stress, the effective stress is equally increasing in all directions, and thus isotropic consolidation is expected (Qian et al., 1992; Park et al., 1997; Leong et al., 2000; Masse et al., 2001). Chai et al. (2005) stated that the effective stress path experienced by a soil element undergoing vacuum consolidation varies with depth, and that for a soil element at or near the ground surface, the effective stress...
path is close to isotropic consolidation, while at deeper locations it is closer to one-dimensional consolidation.

However, as can be seen from Figure B.2, even though the vacuum pressure induces an isotropic pore water pressure reduction, the variation of the horizontal effective stress will still depend on the $k_v$ conditions of the soil element.

\[
\sigma'_{ho} = k_v\sigma'_{vo} \\
\sigma'_v = \sigma'_{ho} + \Delta\sigma_{vwc}
\]

Figure B.2: Soil elements under vacuum consolidation: (a) initial stress state and (b) stress state during vacuum consolidation.

In order to consider the same two cases as before (Figure B.3), a thin and a thick compressible layer, it is to be noted that in vacuum consolidation prefabricated vertical drains are normally driven down to an specified depth in order transmit the vacuum load directly to the ground (see §2.2.3).

If the case of the thin compressible layer is analysed under vacuum consolidation conditions (Figure B.3(a)), the same zero lateral strain condition as explained above applies. In Figure B.3(b), a-b1-c1 is the effective stress path under one-dimensional vacuum consolidation.

In Case 2, as the vacuum load is transmitted directly to ground through the PVDs, the suction pressures will in fact increase the horizontal effective stress preventing lateral deformation. Thus, a soil element located in the centre of a thick soil deposit (Figure B.3(a)) undergoing vacuum consolidation will follow a one-dimensional consolidation stress path, a-b1-c1 as shown in Figure B.3(b).
Due to the consolidation movements in the treated area, the lateral stress of the elements around it is gradually reduce until tension stresses are reached, and thus vertical tension cracks are formed. Once the tension cracks appear, there is no further lateral restrain on the ground and inwards lateral deformation occurs due to the suction pressure inside the treated area.
B.3 EXAMPLE OF VACUUM CONSOLIDATION STRESS PATHS

The following example presents the stress path of a soil element located a depth of 1.0 m. The soil is normally consolidated. Note that the stresses and pressures marked with the subscript (abs) indicate that the pressure includes the atmospheric pressure.

Table B.1 gives the geotechnical properties and initial stress state of the soil element, Table B.2 shows the calculations according to Figure B.2 and Figure B.3, and Figure B.4 presents the stress path, failure line and $k_0$ line for the soil element.

<table>
<thead>
<tr>
<th>Table B.1: Soil properties and initial state of stress.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Atmospheric pressure (kPa)</td>
</tr>
<tr>
<td>Depth (m)</td>
</tr>
<tr>
<td>$c'$ (kPa)</td>
</tr>
<tr>
<td>$\phi'$ (°)</td>
</tr>
<tr>
<td>$k_0$</td>
</tr>
<tr>
<td>$\gamma$ (kN/m³)</td>
</tr>
<tr>
<td>$\gamma_w$ (kN/m³)</td>
</tr>
<tr>
<td>$u_s$ (kPa)</td>
</tr>
<tr>
<td>$u_{total, i}$ (kPa)</td>
</tr>
<tr>
<td>$\sigma_{ini}$ (kPa)</td>
</tr>
<tr>
<td>$\sigma_{ini}$ (kPa)</td>
</tr>
<tr>
<td>$\sigma_{total, i}$ (kPa)</td>
</tr>
<tr>
<td>$\sigma_{total, i}$ (kPa)</td>
</tr>
<tr>
<td>$\sigma'_w$ (kPa)</td>
</tr>
<tr>
<td>$\sigma'_w$ (kPa)</td>
</tr>
</tbody>
</table>

This simple example shows how vacuum consolidation follows a one-dimensional stress path, contrary to what the current literature available indicates. The example can be easily setup in a spreadsheet, and calculations can be carried for different geotechnical properties and depths.
Figure B.4: Vacuum consolidation stress path.
### Appendix B

**STRESS PATHS IN VACUUM CONSOLIDATION**

Table B.2: Vacuum consolidation stress path calculations.

<table>
<thead>
<tr>
<th>$\Delta \sigma_{vac}$</th>
<th>$u$</th>
<th>$u_{abs}$</th>
<th>$\sigma'_v$</th>
<th>$\sigma'_h$</th>
<th>$\sigma_v$</th>
<th>$s'$</th>
<th>$t'$</th>
<th>$s$</th>
<th>$t$</th>
<th>$s'_f$</th>
<th>$t'_f$</th>
</tr>
</thead>
</table>