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EXPERIMENTAL INVESTIGATION ON THE STABILISATION OF IRISH PEAT

BY

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DISSERTATION SUBMITTED TO THE UNIVERSITY OF DUBLIN IN FULL
FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE OF DOCTOR OF PHILOSOPHY

DEPARTMENT OF CIVIL, STRUCTURAL & ENVIRONMENTAL ENGINEERING

TRINITY COLLEGE, DUBLIN
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ABSTRACT

This thesis presents the findings of a study of the behaviour of a typical Midland Irish peat when mixed with a binder. The study shows that the strength of the peat can be considerably increased by the addition of various binders, however the degree of improvement depends on the type of peat and on the binder itself. A comprehensive investigation was carried out into the behaviour of one peat/cement mixture. The engineering properties of the stabilised peat were considerably improved over those of the original peat. Inspection of the stabilised peat using an electron scanning microscope indicated that this improvement was due to filling of the peat voids with cementitious material, there was no bonding of the cementitious material and the organic matter. The geotechnical parameters of the stabilised soil were determined in a comprehensive laboratory test programme and the findings of this programme were used as input to a finite element analysis of a stabilised soil structure, consisting of a stabilised surface layer and a stabilised column, which was constructed in a large-scale testing laboratory chamber. The settlements of the treated peat were significantly reduced and the rate of the consolidation accelerated in comparison with the original peat. The assumption of equal strain conditions between the stabilised peat column and the adjacent peat layer was verified; however, the draining role of the column was not confirmed. A good agreement was obtained between the finite element analysis and the experimental behaviour of the material in the testing chamber.
SUMMARY

The objective of this project was to enhance the current understanding of the methods that can be used to stabilise peat and the engineering properties of the stabilised soil. The stabilisation of peat is relatively new and, to date, most of the experience gained in this field was on the stabilisation of inorganic soils, mainly soft clays. Consequently, the approach taken in this project was to: (1) assess the feasibility of stabilising peat, (2) investigate the fundamental behaviour of the stabilised peat in the laboratory and (3) assess the applicability of the stabilisation technique at large-scale, in a calibration chamber.

Two typical Irish Midlands peats were selected for this study. The results of the unconfined compression tests performed on both peats stabilised with different mixtures showed that: (1) the stabilisation of peat was best achieved with cement or slag based binders and (2) preloading of the stabilised soil immediately after mixing played a fundamental role in the strength achieved; the study also showed that the behaviour of the stabilised soil became brittle and significant increase in strength was measured compared to when the stabilised soil was not preloaded. The examination of the micro-structure of the stabilised peat showed that the mechanism of stabilisation of peat was by filling the voids. No interaction occurred between the organic matter and the hydrated cementation components.

The mechanical behaviour of the cement-stabilised peat, as assessed from a series of laboratory tests, showed the following: (1) the stabilised peat behaved as a stiff material, (2) the compressibility and creep behaviour were significantly reduced compared to the original peat, (3) the permeability of the stabilised peat decreased, compared to the original peat, if the soil was preloaded after mixing and (4) the shear strength of the stabilised peat increased significantly.

The use of this stabilisation technique in a large-testing laboratory chamber of 2.3m high and 1.68m diameter, where a cement-stabilised peat column and a cement-stabilised surface peat layer were formed, proved to be efficient in reducing the settlements of the treated peat and accelerating the rate of consolidation. In-situ assessment of the strength of the stabilised material showed that despite using a laboratory mixer, the strength of the column and the stabilised mass was heterogeneous. Some unstabilised lumps of peat were
found in the column. Equal strain conditions were verified at the top of the stabilised peat column; however, the permeability of the stabilised peat was not higher than that of the original peat. The increase in the rate of settlement was due to the higher value of the coefficient of consolidation $c_v$ of the stabilised peat column than that of the peat.

A good agreement was obtained between the experimental and simulated behaviour using soil parameters derived from laboratory and in-situ tests. The correlation used between the drained stiffness modulus $E'_{50}$ and the undrained shear strength (i.e. $E'_{50} \approx 20c_u$, back analysed from standard oedometer tests performed on stabilised peat samples recovered from the testing chamber) seemed to yield a good estimation of the stiffness of the stabilised material in the testing chamber.
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CHAPTER 1
INTRODUCTION
1. INTRODUCTION

1.1. BACKGROUND TO THE PROJECT

Peats cover a great part of Ireland and of many other European countries. These soils are characterised by a high compressibility and low shear strength. The increasing demand for transport infrastructures such as highway and railway embankments, and the scarcity of available land are such that it is no longer possible to avoid constructing over these soils. Furthermore, the increasing sensitivity of society to the environmental impacts of infrastructural projects encourages the development of in-situ methods of treating such soils in order to avoid the traditional method of replacing these very soft and compressible soils.

The soil stabilisation technique is one of several in-situ methods of treating soft soils. It has been widely used in Scandinavia and Japan since the early 1970's for soft clays. Lime, lime-cement and cement columns are formed in the soft ground and the interaction of the binder with the original soil leads to the formation of new phases which are shown to improve the mechanical characteristics of the treated soil.

In 1997, the European Union initiated a project called "EuroSoilStab", in which Trinity College Dublin was one of the laboratories involved. This project was aimed at developing methods for deep soil stabilisation of organic soils. The present work carried out was concerned with examining potential binders for peat, laboratory testing of the stabilised peat and a large-scale laboratory test on stabilised peat.

Most of the reported investigations carried out on the stabilisation of organic soils have focused mainly on the development of strength with time or the study of the effect of organic matter on the setting and hydration of cement binders. Binder stabilised peat is a new material that has not been fully investigated previously, thus little is known on the mechanisms involved in the stabilisation of peats. A number therefore arise: how does the binder interact with the organic matter? what are the main products formed? and how does the organic matter interfere with the hydration of the binder used? Similarly, many questions remain to be answered on the mechanical behaviour of the stabilised peat in terms of compressibility, permeability and shear strength.
If the binder stabilisation of peat proves successful in the laboratory, its transfer to the field is still subject to assessing its efficiency in terms of reducing the settlements and improving the overall stability of the treated peat. The main assumptions currently used in design, such as equal strain conditions between the stabilised column and the adjacent soil and the draining role of the stabilised columns, were initially developed for lime-stabilised soft clays. Their validity for peat, or organic soils in general, is still not known and remains to be investigated. Furthermore, for this technique to be accepted, it is necessary to be able to simulate the experimental behaviour of the stabilised peat using soil parameters derived from laboratory and in-situ tests.

1.2. OBJECTIVES OF THE PROJECT

The objectives of the project were as follows:

(i) To investigate potential binders and the optimum peat-binder combination for improving the geotechnical properties of two typical Irish Midlands peats, one from Ballydermot Bog and the other from Raheenmore Bog, and to examine the main changes or phases occurring at microscopic level in order to understand the mechanism of stabilisation. An optimum peat-binder combination was selected from the results of the unconfined compression tests carried out,

(ii) To investigate the mechanical behaviour of the stabilised peat on the optimum peat-binder combination selected. The compressibility, permeability and shear strength of the stabilised peat was investigated.

(iii) To investigate the behaviour of the soil stabilisation technique in a large-scale testing instrumented chamber, where the original peat is treated in the same manner as in the field and loaded to simulate an embankment loading. The experimental behaviour of the material in the large-testing chamber will be back analysed with a finite element model in order to assess whether it is possible to simulate the behaviour of the stabilised material using soil parameters derived from laboratory and in-situ tests,
(iv) To compare the main findings of this project with the current knowledge in soil stabilisation of mineral soils.

1.3 OUTLINE OF THE THESIS

This thesis is composed of eight chapters outlined as follows:

Chapter 2: reviews the chemical processes that occur when lime, cement and other products are added to clay or sand and summarises the extent of strength increase achieved. The effect of the presence of organic matter on these chemical processes is discussed. This chapter also reviews the mechanical properties of the stabilised soil as well as the design approaches taken and the field methods used to stabilise the ground.

Chapter 3: presents a description of the two peat sites selected and the results of classification tests along with other laboratory tests such as triaxial, oedometer, permeability and ring shear tests carried out on samples of peat recovered from both locations.

Chapter 4: presents the mixing and the laboratory testing procedure used for the laboratory preparation and testing of stabilised soil. The binder design is reviewed along with the experiences of other laboratories. The results of a full suite of laboratory tests, including consolidation, permeability and shear tests, carried out on cement-stabilised Ballydermot peat are presented.

Chapter 5: outlines details of the preparation, construction and loading procedures adopted for the testing of a stabilised soil structure in a large-scale testing chamber. Details of the instrumentation used for the monitoring of the test are presented as well as results of a parametric finite element study of the calibration chamber test, which was used to design the stabilised soil structure in the large-scale testing chamber.

Chapter 6: presents results of the large-scale testing chamber test. The material in the chamber was subjected to loading and unloading sequences. After the completion of the tests, in-situ measurement of the strength of the stabilised material was performed.
Samples of the stabilised material were also recovered for laboratory testing. Results of the back analysis of the chamber test using numerical methods are presented.

Chapter 7: interprets the findings of the main experimental results obtained in the laboratory and relates these to the expected behaviour of the stabilised soil and to the findings of other researchers. The discussion covers both results obtained on laboratory mixed stabilised peat and those obtained from the large-scale testing chamber test. The results of the back-analysis of the large-scale testing chamber test are also discussed.

Chapter 8: presents the main findings of this research and proposals for future research.

Additional data are provided in the following appendices:

Appendix I: reviews the main laboratory mixing procedures used for forming the stabilised soil. Several factors such as the mixing time, curing procedure and the procedure of testing are discussed.

Appendix II: reviews reported finite element studies on stabilised columns. The effect of column spacing, stiffness of the stabilised material and the main correlation used in the numerical modelling are discussed.

Appendix III: presents results of the unconfined compression tests carried out on both Raheenmore and Ballydermot peat stabilised with different mixtures.

Appendix IV: presents details of the Plaxis finite element soil models used in the back analysis of the large-scale testing chamber test.
CHAPTER 2

LITERATURE REVIEW
2. LITERATURE REVIEW

2.1 INTRODUCTION

The addition of materials to soils to improve their geotechnical properties has been in use since the early part of the twentieth century. Lime and cement were both widely used in road construction to improve the strength of the subgrade or sub-base material in the 1940's and 1960's. This technique of soil improvement was taken further in the 1970's in both Sweden and Japan by developing methods of injecting materials and mixing them in-situ with the natural soil to form strengthened soil columns. The methods of mixing have been improved over the years and new additives have been developed enabling a great variety of soils to be treated.

This chapter reviews the chemical processes that occur when lime, cement and other products are added to clay or sand and summarises the extent of strength increase achieved. The effect of the presence of organic matter on these chemical processes is discussed. This chapter also reviews the design approaches taken and the field methods used to stabilise the ground.

2.2 LIME STABILISATION OF SOFT CLAYS AND OTHER INORGANIC SOILS

Lime stabilisation refers to the stabilisation of soil by the addition of either quick lime CaO or hydrated lime Ca (OH)$_2$ to the soil. The first use of this technique seems to go back to 1924 in the state of Missouri (USA) as reported by Clare & Cruckely (1957). Mateos (1964) reports that the laboratory investigation of lime stabilisation of soils began as early as 1947 at Iowa University and was still in progress in 1964. However, extensive laboratory research did not start until the 1960's in the United States, Sweden and Japan.

The experience shows that when the soil is mixed with the lime, two phenomena take place: (1) there is a quick change in the properties related to the natural water content of the soil (consistency limits), (2) there is an increase in strength of the soil/lime mixture.
According to Locat et al. (1990), the mechanisms believed to occur between the lime and the clay are as follows:

(i) cation exchange,
(ii) flocculation and particle aggregation,
(iii) lime carbonation,
(iv) pozzolanic reactions between lime, silica and alumina.

Mitchell (1976) described cation exchange of clay as the ability of the clay, under a given set of environmental conditions, to adsorb cations of specific type and amount. The exchange reactions involve the replacement of a part or all of the adsorbed ions by ions of another type. The ease of replacement depends mainly on the valence, relative abundance of the different ion types, and ion size. The general order of replaceability of the common cations is given by the lytropic series, $\text{NH}_4^+ < \text{Mg}^{2+} < \text{Ca}^{2+} < \text{Al}^{3+}$, i.e any cation will tend to replace that to the left of it and monovalent cations are generally replaced by multivalent cations. The addition of lime to the soil will supply an excess concentration of $\text{Ca}^{2+}$, which in turn will replace other cations present in the clay. In some cases, the exchange complex is already $\text{Ca}^{2+}$ saturated before the lime is added and cation exchange does not occur. (Thompson, 1966).

The flocculation and agglomeration were described by Rogers et al. (1996) as being a result of the greater attraction between clay particles that occurs due to their closer proximity. This attraction results from the reduction in the thickness of the adsorbed water layer. Thompson (1966) suggested that flocculation and agglomeration are primarily responsible for changes of soil properties related to the natural water content of the soil. Kezdi (1979) attributes flocculation to cation exchange as calcium cations occupy the place of other cations, usually sodium cations, in the adsorption complex of the clay minerals. This explains somewhat, the reason why clay minerals with high cation exchange capacity (CEC) such as montmorillonite undergo large changes in properties on mixing with lime whereas those with low CEC such as illite, undergo small changes.

Carbonation of lime is the reaction of the lime with the carbon dioxide, $\text{CO}_2$, present in the air. This reaction leads to the production of calcium and magnesium carbonate and when this occurs in a soil the precipitate results in a slight cementation effect.
Eades & Grim (1960) reported formation of calcium carbonate when lime-treated soils were laboratory-cured in the open air.

The pozzolanic reactions are caused by the interaction of the lime with the silicate and aluminate minerals in the soil in the presence of water. This interaction produces cementitious products which are calcium silicate hydrates (CSH) commonly called tobermorite and calcium aluminate hydrates (CAH). In the course of the soil/lime interaction, these products are regarded as the most important compounds that contribute to an increase in strength of the mixture. According to Kezdi (1979), it is still not clear how this reaction is brought about. The solubility of the silicate and aluminate minerals is a function of the pH, see Figure 2.1, and it is believed that the rise in pH of the soil pore water, as a result of addition of lime to the soil, will in turn give rise to an increased solubility of the silicate and aluminate minerals present in the soil.

![Figure 2.1. Solubility of silicate and aluminate as a function of the pH (Kezdi 1979).](image)

The dissolution of silicates and aluminates due to the high pH leads to the rearrangement of these compounds. Eades (1962) suggested that the high pH causes silica to be dissolved out of the structure of the clay minerals and it combines with the Ca$^{++}$ to form calcium silicates. This reaction will continue as long as Ca(OH)$_2$ exists in the soil, and there is silica available. Therefore, this suggests that the pozzolanic reaction starts first of all with the dissociation of the calcium hydroxide Ca(OH)$_2$ into Ca$^{++}$ and OH$^-$. If one considers the solubility of both silicate and aluminate with the variation of pH, both products can be found at different states depending on the pH of the solution as shown in Figure 2.2.
Figure 2.3. Chemical equations of formation of CSH and CAH.

In practice, the pozzolanic reactions cannot be simply represented by the chemical equations shown in Figure 2.3. This is confirmed by the great variability of the products formed as a result of the interactions between lime and the soil. Diamond et al. (1964) showed different products are formed depending on the experimental conditions. The mode of attack of clay minerals by calcium hydroxide was postulated to involve dissolution at the edges of the silicate particles due to the high pH maintained by the calcium hydroxide, followed by precipitation of the reaction products.
Brand (1962) studied the main phases developing in the pozzolanic reactions occurring after mixing lime with the soil by a means of microscopic, electron microscopic and chemical examinations. The author suggested that the clay minerals participate in these reactions and that the rate and intensity of the reactions are greatly promoted by the high pH developed through the addition of calcium. The interaction between lime and the soil was found to occur in four phases spread over time as described in Figure 2.4. The initial stage starts with the dissociation of Ca(OH)$_2$ into Ca$^{2+}$ and OH$^-$ which creates a transition to the gel phase. The finest soil particles start agglomerating which results in a significant change in the soil structure and its consistency. The gel phase is believed to consist of tobermorite-type calcium silicate hydrates and is therefore considered as the most important phase in terms of strength development as it will connect and cement the mineral grains. The only difference between this interaction and that, which occurs with ordinary Portland cement is that when using lime the silicate gel is only formed after attack and removal of silica from the clay minerals of the soil. Ingles & Metcalf (1972) claim that the action of cementing the mineral grains proceeds only whilst water is present and able to carry calcium and hydroxyl ions to the clay surface (i.e. whilst pH is still high).

Figure 2.4. Development of the individual reaction stages (Kezdi 1979).
From the gel phase “neoliths” start developing, slowly creating a “neolithic” phase which is believed to be an important part of the soil consolidation. The carbonation phase will only occur if samples are stored in air. Figure 2.5 shows the mechanism of lime stabilisation of clay soils proposed by Ingles (1970).

![Mechanism of lime stabilisation of clay soils](image)

Figure 2.5. Mechanism of lime stabilisation of clay soils (Ingles 1970).

Considering the processes described earlier, the addition of lime to the soil affects the physical properties of the soil such as its consistency limits and shear strength. The flocculation and agglomeration, which occur as a result of the lime/soil reaction, will in turn result in changes in the plastic limit of the soil as shown in Figure 2.6.

Figure 2.6 shows results obtained by Mateos (1964) for three different clayey soils from Iowa (USA). The effect of lime on the plasticity of different types of clay can be clearly seen. The lime is shown to increase the plastic limit ($\omega_p$) of soils to a certain level, thereafter the plastic limit levels off. This increase in plastic limit is accompanied by a decrease in liquid limit. Consequently, the plasticity index decreases as well (Mateos, 1964). The change in plastic limit seems to be dependent on the amount and type of clay present in the soil.
Figure 2.6. Effect of addition of lime on the plastic limit of soils. (Number after the letter indicates the percentage of the specific clay mineral: I-41 represents a soil with 41% illite) (Mateos 1964).

Rogers et al. (1996) reported some results obtained, after (Dumbleton, 1962), on London clay treated with lime. A considerable increase in plastic limit occurred, almost immediately after mixing, with the addition of only 1% lime. However, very little change in plastic limit was observed once 4% lime has been added. This observation brought about the definition of a lime fixation level for clays (Rogers et al. 1996; Mateos 1964). The latter author suggested that the lime fixation level is the process that brings the changes in plastic limit of a soil until it levels off. Therefore, only lime that is in excess of the fixation level contributes to cementation. The lime fixation level for London clay was found to lie between 2 and 4% lime content added to the soil.
The cementitious products, which result from the addition of lime, will produce an increase in the unconfined shear strength of the stabilised soil. Figure 2.7 shows the strength development with both curing time and lime content for various soils stabilised with lime.

![Figure 2.7](image)

Figure 2.7. (a) The effect of age on strength of various soils stabilised with 5% lime (Ingles, 1970). (b) The effect of lime addition on compressive strength (Kezdi, 1979).

The lime stabilisation technique has, according to Ingles & Metcalf (1972), little effect in highly organic soils and also in soils with little or no clay content. It is most effective in clayey gravels and particularly suited to the stabilisation of heavy clay soils. Regarding the clayey soils, lime reacts more quickly with montmorillonitic clays than with kaolinitic clays and the difference may amount to a few weeks.
2.3 CEMENT STABILISATION OF SOFT CLAYS AND OTHER INORGANIC SOILS

According to Ingles & Metcalf (1972), cement is widely used for soil stabilisation. The addition of even small quantities, up to 2%, will modify the properties of the stabilised soil. Usually cement stabilisation of soils is carried out with Normal Portland Cement but sometimes, special cements such as Rapid Hardening Cement or Sulphate Resisting Cement are used depending on the type of soil treated. In contrast to the lime/soil interaction, the interaction between cement and soil is different depending on whether the soil is a fine-grained silt and clay or a granular soil.

According to Kezdi (1979), in fine-grained silts and clays the hydration of cement creates quiet strong bonds between the various mineral substances and forms a matrix, which efficiently encloses the non-bonded soil particles. The matrix will develop a cellular structure on whose strength the entire construction depends, since the strength of the clay particles within the matrix is quiet low. In granular soils, the cementation effect is similar to that of concrete, the only exception is that the cement paste does not fill the voids. Therefore, the soil is only cemented at the contact points. The better graded the particle sizes the greater the number and the larger the interparticle contact surfaces, the stronger the effect of cementation.

Normal Portland Cement particles are composed of C₃S (tricalcium silicate), C₂S (bicalcium silicate), C₃A (tricalcium aluminate) crystals and a solid described as C₄AF (tetracalciumalumino-ferrite). The major constituents are C₃S and C₂S. When the cement clinker is exposed to water, the anhydrous components react to form hydrated products, principally calcium silicate hydrate (CSH) and well crystallised calcium hydroxide (Portlandite) (Diamond, 1976). CSH is considered as the most important compound as regards strength production. The chemical reactions occurring between cement compounds and water are shown in Figure 2.8 (Brunauer & Copeland 1964).

From the chemical reactions it can be seen that hydration of C₃S and C₂S produces tobermorite gel and releases free lime. The tobermorite gel plays the leading role as regards strength production and bonding. As suggested by Kezdi (1979), in order to have additional bonding forces produced in the cement-clay mixture, the silicates and
aluminates must be soluble. These latter compounds will react with the free lime, released during hydration of the cement, as explained previously in the lime stabilisation of soils.

\[
\begin{align*}
2(3\text{CaO} \cdot \text{SiO}_2) + 6\text{H}_2\text{O} & = 3\text{CaO} \cdot 2\text{SiO}_2 \cdot 3\text{H}_2\text{O} + 3\text{Ca(OH)}_2 \\
(\text{C}_3\text{S}) & \text{(tobermorite)} \\
2(2\text{CaO} \cdot \text{SiO}_2) + 4\text{H}_2\text{O} & = 3\text{CaO} \cdot 2\text{SiO}_2 \cdot 3\text{H}_2\text{O} + \text{Ca(OH)}_2 \\
(\text{C}_2\text{S}) & \text{(tobermorite)} \\
4\text{CaO} \cdot \text{Al}_2\text{O}_3 \cdot \text{Fe}_2\text{O}_3 + 10\text{H}_2\text{O} + 2\text{Ca(OH)}_2 & = 6\text{CaO} \cdot \text{Al}_2\text{O}_3 \cdot \text{Fe}_2\text{O}_3 \cdot 12\text{H}_2\text{O} \\
(\text{C}_4\text{AF}) & \text{(calcium aluminoferrite hydrate)} \\
3\text{CaO} \cdot \text{Al}_2\text{O}_3 + 12\text{H}_2\text{O} + \text{Ca(OH)}_2 & = 3\text{CaO} \cdot \text{Al}_2\text{O}_3 \cdot \text{Ca(OH)}_2 \cdot 12\text{H}_2\text{O} \\
(\text{C}_3\text{A}) & \text{(tetracalcium aluminate hydrate)}
\end{align*}
\]

Figure 2.8. Chemical reactions of hydration of cement compounds (Brunauer & Copeland, 1964).

Based on the compounds produced by the hydration of cement, Kezdi (1979) suggested the occurrence of two processes during the course of interaction between cement and clay:

(i) The primary process includes the hydrolysis and the hydration of cement, in the course of which the usual hydration products appear and the pH of the water increases. The calcium hydroxide produced in this period can react with the clay minerals much more strongly than ordinary lime,

(ii) The calcium ions released during cement hydration, initially transform the clay into “calcium clay” and increase the intensity of the flocculation that had been initiated by the increased total electrolyte content due to addition of cement. Calcium hydroxide then attacks the clay particles and the silicates and aluminates dissolved in the pore water will mix with the calcium ions and additional cementing product is precipitated.

It should be noted that calcium hydroxide consumed in the second process is partly replaced by the lime produced during hydration of cement. Kezdi (1979) claims that cementation products formed in the second process are formed over the clay particles causing the flocculated clay grains to be bonded at the contact points. This may create
stronger bonds between the hydrating cement paste and the clay particles coating the cement grain.

Ingles & Metcalf (1972), in an attempt to explain the mechanism whereby cement changes the properties of the treated soil, suggested that cement forms either strong nuclei, distributed throughout the mass, or else that a skeleton of hydrated cement is formed throughout the voids in such a manner as to restrain the unaffected soil.

As discussed previously in § 2.2, the formation of cementitious compounds in the cement and soil mixture increases the strength of the mixture. Significant strength can be achieved as can be seen from Figure 2.9 which illustrates the variation of the unconfined compressive strength of various soils with both cement content and curing age (from Metcalf 1959).

![Graphs](image)

Figure 2.9. (a) Effect of cement content on strength for various soils stabilised with OPC and cured for seven days. (b) Effect of age on strength of various soils stabilised with 5% cement (Metcalf, 1959).
Products other than cement and lime have also been considered for soil stabilisation. Some of these products were used to investigate potential commercial use, such as industrial wastes and by products, and others were added to either cement or lime to improve their reactivity. As emphasised by Ingles & Metcalf (1972), many proffered soil stabilisers are patented products developed individually or by particular industries, so that their use can only be made after careful field trials. Those authors reported the use of a multitude of stabilisers such as phosphoric acid, sodium chloride, sodium hydroxide, resins and polymers and calcium salts but no results are presented to illustrate the improvement achieved.

Moh (1962) reported the use of alkali sodium compounds to improve the cement stabilisation of various soils from non-plastic sands to heavy clays. Different types of sodium additives were used and their efficiency was found to be dependent on the type of soil treated. That author suggested the use of this method as a more economical means of stabilising soils responsive to cement treatment. The major consequences of the incorporation of such compounds were summarised by the author as follows: (1) an increase in pH in the pore fluid, (2) a marked reduction in the calcium ion concentration, (3) a marked increase in the sodium-calcium ratio in the solution. The ultimate results of these effects were: (a) an increased rate and extent of solubilisation of soil silica, (b) retardation of the precipitation of calcium silicate gel, (c) formation of highly hydrated silicate gels containing a substantial quantity of sodium. These phenomena, according to the author, lead to the production of an increased volume of cementitious gel than would be produced from soil-cement alone.

Marks & Haliburton (1972) used sodium chloride additives to accelerate the lime-clay reactions. The addition of 1% sodium chloride to the soil-lime mixture was found to substantially increase the compressive strength and the rate of strength gain of the Permian Red clay from Oklahoma (USA). Laboratory investigations of the lime stabilised specimens with NaCl additives, using differential thermal analysis, showed that sodium chloride altered the mineralogy of treated clays and produced new mineral formations when used in conjunction with lime. NaCl acted as a catalyst to accelerate and enhance lime-clay reactions by causing earlier new mineral formations.
Esrig (1998) considered that gypsum was among the more important binder components utilised in past years. The combination of gypsum and lime or cement produces ettringite, which is needle-like shaped, and has been observed to increase the final strength of the stabilised ground. However, the swelling characteristics of the ettringite cause unacceptable heaving of highway pavements and structural floor slabs. The same author reported results obtained from the stabilisation of Boston Blue Clay (BBC). Cement and quicklime were the primary constituents of the binders. Other additives such as flyash, gypsum and coal slag were added to reduce cost. The results obtained showed that the unconfined compressive strengths of all binder mixtures tested on BBC, except those using hydrated lime, increased with time at essentially the same rate. The strength of the treated soil increased more slowly when hydrated lime rather than quicklime was added to cement. However, the strength achieved at 28 days of the soil treated with hydrated lime was 30% larger than counterparts using the same amount of quicklime.

Kujala & Nieminen (1983) and Holm et al. (1983) reported the use of gypsum combined with lime for the stabilisation of clayey soils. Kujala & Nieminen (1983) used a mixture of waste gypsum and lime to stabilise four different clays from southern Finland. After 28 days of curing time, the shear strength of the gypsum lime stabilised clay was two to four times higher than that of the lime stabilised clay. Mineralogical changes were studied by means of x-ray diffraction and thermoanalysis. Results showed that the greater initial shear strength was, among other things, the result of ettringite growing out of the surface of clay particles. The ettringite formed caused a strong pressure of expansion, which in turn had considerable effect on the strength of gypsum lime stabilised clay.

Holm et al. (1983) used combinations of gypsum and unslaked lime in a field project. Findings similar to those of Kujala & Nieminen (1983) were reported. The shear strength of gypsum lime stabilised clay increased much faster than lime stabilised clay and was two to four times higher from 10 to 100 days after mixing. The gypsum lime stabilised clay was found to reach near-maximum shear strength after 3 months from mixing and then increased slowly, whereas, the shear strength of lime stabilised clay kept increasing over some years after mixing. The use of gypsum together with lime was found to extend the range of clays that can be potentially stabilised. Clays with water content up to 140% were stabilised with good effects.
Andersson et al. (2000) reported the increasing use of granulated blast furnace slag, in combination with cement, for the stabilisation of organic soils in Sweden. A combination of cement/granulated blast furnace slag in a proportion of 50:50, added to the organic soil, was extensively investigated in the laboratory along with a combination of lime/cement in a proportion of 50:50. Results of the unconfined compression tests showed that the strength achieved by the former binder was more than three times higher than for the lime/cement binder. The combination of cement and granulated blast furnace slag proved to be very encouraging for the stabilisation of organic soils.

Pulverised Fuel Ash (PFA) is often used in conjunction with lime or cement for the stabilisation of soils. PFA is a pozzolan material that reacts with CaO and water to form a cementitious material. Bell (1990) claims that because there is slower gain in strength when PFA is mixed with lime, cement has usually been preferred as the main binder. The author reported the use of lime-PFA slurry to stabilise soft soils underneath embankments. A small increase in strength of 15-30% was observed.

2.5 EFFECT OF ORGANIC MATTER ON THE SETTING AND HARDENING OF CEMENT

It is generally recognised that organic soils can retard or prevent the proper hydration of cement in cement-soil mixtures. According to Ingles & Metcalf (1972), "...any soil, with the exception of highly organic materials, may be treated with cement and will exhibit an improvement in properties: increase in strength..." The main approach adopted by the researchers investigating the effect of organic matter on the hydration of cement was to identify the principal components present in the organic matter that were more likely to interfere with the hydration.

The investigation of the effect of the organic matter on the setting and hardening of cement started as early as 1954 (Clare & Sherwood). The authors investigated the effect of organic matter on the setting of cement by studying the retarding effect of individual organic compounds added to clean sand mixed with cement. The strength developed from the addition of each individual organic compound to the sand and also of clean sand with no additives was used to assess the effect of the different compounds on the setting of cement.
Unconfined compression test results showed that specimens containing some organic additives have been found to have significant lower strength than with no additives. The organic compounds were grouped according to their "activity" in retarding the setting of cement and the most active compounds were found to be glucose and nucleic acid. The authors tested an organic sandy soil mixed with cement. Results of the strength tests showed that the amount of organic matter could not be correlated with the compressive strength. It was concluded that the retardation of the setting of cement was not related to the total amount of organic matter but probably to "some" active fraction. The retarding effect was found to be overcome by the addition of calcium chloride to the cement or by using super rapid hardening cement.

The authors suggested that the retardation of the hardening of the organic soil-cement mixtures is due to the retention by the organic matter of calcium ions liberated during the hydrolysis of the calcium silicates and aluminates in the cement grains. Therefore the calcium ions retained are not available for forming the compounds constituting the cement matrix. The authors suggested that the activity of the organic matter should be related to its capacity to adsorb calcium ions from solutions of high pH and that the fraction particularly responsible probably contains carboxylic acid or phenolic functional groups -see Figure 2.10.

![Figure 2.10](image1)

Figure 2.10. (a) Effect of the capacity to adsorb calcium ions on the unconfined compressive strength of soil-cement mixtures. (b) Effect of soil pH on the unconfined compressive strength of a soil-cement mixture (Clare & Sherwood, 1954).
Further investigations were carried out by Maclean & Sherwood (1962). The authors investigated the effect of the different fractions extracted from the organic matter on the setting of ordinary Portland cement. The effect of the different fractions was assessed by mixing them with an organic-free sand and determining the compressive strength at a curing age of 7 days of the organic contaminated sand when stabilised with 10% cement. Results showed that most of the fulvic and humic acid fractions extracted reduced the strength obtained with the organic free sand mixed with cement. The most active fraction was found to be the hydroxy-quinone type, which is known to form complexes with metal ions such as those of calcium. The authors believed that this property of forming complexes was the cause of the activity of certain organic compounds towards cement. A trend has been observed between the pH of the soil-cement mixture, measured 15 min. after mixing, and the unconfined compressive strength achieved at 7 days. The authors suggested that the pH of the soil-cement mixture 15 min. after mixing could be a good indicator of the presence of any active organic matter that interferes with normal hardening of the cement. Figure 2.11 is very illustrative of the relationship between the pH and the unconfined compressive strength.

![Figure 2.11](image_url)

Figure 2.11. (a) Relation between pH and unconfined compressive strength of soil-cement mixtures. (b) Effect of soil pH value on the unconfined compressive strength of soil-cement mixtures (Maclean & Sherwood, 1962).
According to Stevenson (1994), the COOH functional group present in the organic matter plays a prominent role in the complexation of di- and trivalent cations by humic acids. The maximum binding capacity of humic substances for any given metal ion is approximately equal to the content of acidic functional groups, primarily COOH. Tsutsuki & Kuwatsuka (1978), in an examination of over 30 humic acids from a wide variety of soils, found that COOH group increased in amounts during humification (i.e. COOH group increases as the degree of humification of the peat increases).

Thompson (1966) suggested that the organic matter retards lime-soil reactions by the means of a “masked effect” which the author describes as the effect of preventing the clay from yielding silica or alumina, or both, for reaction with lime. The author compared the strength achieved for two soils, one in a natural state and the other one treated with hydrogen peroxide to remove the organic matter, mixed with 5% lime. Results showed that the soil treated with hydrogen peroxide displayed considerably greater strengths than the natural soil. The author concluded that the removal of the organic matter eliminated the “masking effect” on the clay surfaces.

Arman & Munfakh (1972) investigated the potential of improving the engineering characteristics of organic soils, found in Louisiana, with lime. The organic content of the soils tested was around 20%. Results of unconfined compression tests showed an increase in strength with addition of lime. The authors concluded that the organic matter does not block the pozzolanic reaction in these soils.

Kuno et al. (1989) examined the effectiveness of various chemical admixtures for soils having different humic acid contents. The tests were conducted on 35 very soft soils from Japan. The results obtained showed that the humic acid content greatly influenced the soil improvement effect generated by the binder mixed with the soil. Slaked lime and quicklime were found to have very small effect when humic acid was present, even in small amounts. The use of cement was found to be applicable for soils having a high humic acid content but with water contents less than 200%.

Very interesting work was carried out by Okada et al. (1980). The authors have investigated the mechanisms whereby the peat retards the cement hydration. For this purpose several Japanese peats having different characteristics were tested. The peats
tested had an organic content ranging from 65% to 89.5%. The organic matter was fractionated into bitumen, humic and fulvic acid. The composition of the extracted fractions was analysed and the humic acid was found to be insoluble in water and inactive in Ca(OH)$_2$ solution whereas both fulvic acid and bitumen were found to be active in Ca(OH)$_2$ solution. The authors concluded that it is conceivable that cement hydration is retarded by fulvic acid and/or bitumen. This finding was contrary to what has been previously thought, which was that humic acids were the compounds responsible for retardation of cement hydration. However, according to the authors, the past definition of humic acids included bitumen (i.e. when analysing humic acids, bitumen was not removed by the extraction methods used). Therefore, the authors recommend that the bitumen present must be removed in further investigation of the effect of humic acids on cement hydration.

In a second investigation, the same authors considered the strength properties of different peats and kaolinite clay stabilised with normal Portland cement. Test results showed that the strength of peat stabilised with cement was as follows: peat without bitumen > dried peat > original peat. The strength achieved for dried peat was higher than original peat because of the oxidation that changes the characteristics of a part of the humic and the fulvic acids. The authors concluded that bitumen was probably one, but not the only, compound that retards the hydration of cement. Figure 2.12 shows results obtained by Okada et al. (1980).

![Figure 2.12. Unconfined compressive strength of organic soil hardened with 40% weight normal Portland cement (Okada et al. 1980).](image-url)
A similar investigation was carried out by Kujala et al. (1996). The authors investigated the effect of a commercially available humic acid on the strength properties of various stabilised soils. It was found that the humus and the humic acid have a significant effect on the strength achieved in soils stabilised with hydraulic binding agents. The coarse-grained samples tested scarcely gained in strength after stabilisation when humic acid was added. The effect was similar for fine-grained clay but to a lesser extent. The authors suggested that it is possible to use the humus content of a soil as a criterion for the assessment of soils that will react poorly to stabilisation.

2.6 MECHANICAL PROPERTIES OF THE STABILISED SOIL

To understand the mechanism of interaction between the soil and the stabilised columns in the field it is necessary to have a good knowledge of the main mechanical features of the stabilised soil. The addition of a binder to the soil can result in a mixture of greater strength than the original soil, and this is more likely to induce some changes in the behaviour of the stabilised soil. The investigation of the mechanical behaviour of stabilised soil has attracted many researchers (Kivelo 1998; Ahnberg 1996; Uddin et al. 1999).

The behaviour of lime stabilised soil is well documented (Broms 1984). However, the increasing use of combinations of lime-cement and other by-products of the cement and iron industry has led to studies of other mixes. The strength properties of the stabilised soil are found to be affected by the type of stabiliser used as is the stress-strain relationship. Figure 2.13 taken from Lahtinen & Kujala (1990) is very illustrative of this. For instance, the addition of cement to the lime is shown to increase the strength achieved by the stabilised soil and reduces the strain at failure- the behaviour of that stabilised soil becomes more brittle.

The undrained shear strength of the stabilised soil is usually determined from unconfined compression tests. Several empirical relationships were established between the unconfined compressive strength and the secant stiffness modulus of the stabilised soil. However, the use of triaxial tests enables more data to be obtained on the effect of the in-situ stresses and the direction of loading on the shear strength of the stabilised soil. It should pointed out that the majority of the test results reported in the literature were
obtained from fully saturated laboratory-mixed samples and this, of course, is not always representative of the field conditions.

![Stress-strain curves of a soil stabilised with different binders (Lahtinen & Kujala 1990).](image)

Brandl (1983) reported a rise in the friction angle and cohesion after stabilisation of the soil with either lime or cement. The friction angle and particularly the residual angle of shearing resistance rose effectively with the addition of small amounts of lime. Ahnberg (1996) in a suite of consolidated undrained and drained triaxial compression tests carried out on different types of soils stabilised with different stabilisers found that the stabilised soil behaves in a different way depending on the strength of the sample and on the confining pressure. The failure in undrained tests was found to occur at low strains, 1 to 2% for stabilised silty clay and 1 to 17% for stabilised clay and the clayey gyttja. The evaluated effective stress angles of shearing resistance reported were between 34 to 44°. The residual angle of shearing resistance did not differ significantly from that at peak strength. The cohesion intercept was found to increase with increasing strength of the stabilised material and amounted to 25 to 48% of the undrained shear strength measured. However, at large deformations, the cohesion intercept was lower than that measured at failure. The author concluded that the behaviour of the stabilised soil resembles that of cemented and overconsolidated soil.
Uddin et al. (1999) reported some results of triaxial tests carried out on cement-stabilised Bangkok soft clay. The stress-strain response was found to vary with the cement content. Higher amounts of cement induced sharper peak strength and large strain softening. The addition of cement changed the behaviour of the normally consolidated clay to that of an overconsolidated clay. The failure envelope obtained from the triaxial tests was found to be curved with the overall curvature depending on the initial confining pressure applied.

Kivelo (1998) reported results of triaxial tests carried out to investigate the undrained shear strength of lime-cement columns. Two types of soil were tested: clay and organic clay. In order to simulate the in-situ conditions, the samples were not saturated. The undrained shear strength was found to increase with increasing confining pressure, as would be expected from an unsaturated soil. The total angle of shearing resistance measured was found to increase from 30° to 41° as the amount of lime-cement stabiliser increased from 74 kg/m³ to 103 kg/m³. The results of the triaxial tests carried out on the stabilised organic clay showed that the undrained shear strength increased with increasing confining pressure when this is low, whereas for higher confining pressures the failure envelope was horizontal. Figure 2.14 is illustrative of the behaviour of the stabilised organic clay. The total angle of shearing resistance measured for the stabilised organic clay was quite high (i.e. 44° to 45°) and was found to be independent of the amount of stabiliser used. The ratio of the undrained peak cohesion and the unconfined compressive strength for the stabilised organic clay varied between 0.21 and 0.26 irrespective of the strength of the sample tested.

Figure 2.14. Mohr circle of stabilised organic clay (Kivelo 1998).
Soil stabilisation is an in-situ ground improvement technique, which involves mixing the soil with a binder such as cement or lime to improve the engineering properties of the soil. This technique has been used since the 1970's, mainly in Scandinavia and Japan, and is at present being used increasingly in many other European countries, particularly in the Netherlands. The main method is to form columns of improved ground by the following processes:

(i) The dry mix method, which consists of mixing the soil with a dry powdered binder. A shaft with a mixing tool is driven down to the bottom of the planned column, the binder is then injected pneumatically into the soil and mixed at the same time as the mixing tool is withdrawn. See Figure 2.15.

(ii) The wet mix method, mainly used in Japan, consists of mixing the soil with a binder in a wet state (i.e. the binder is initially mixed with water before being mixed with the soil). The mixing tool is driven into the soil and the slurry is injected at the same time as the mixing tool is driven down and withdrawn. In the wet mix method, the binder is injected on the way down and up. See Figure 2.16.

The main dry mix methods used are: the Dry Jet Mixing (DJM), lime cement columns and Trevimix (Bruce et al. 1998). Recently, a new technique for soil stabilisation has been developed and is referred to as “mass stabilisation”. This new technique consists of mixing the whole soil volume by a great number of movements, horizontally and vertically, by the mixing tool, so that a homogeneous reinforced soil slab is formed by the effect of the stabiliser. The mass stabilisation offers the advantage of treating the soil in-situ, therefore, there is no excavation or replacement of the soft soil involved.

A typical construction sequence of stabilised columns using the DJM method is shown in Figure 2.15.
Figure 2.15. Construction sequence of DJM method (Porbaha et al. 1998).

Figure 2.16. Description of the wet mix method. (a) Lime-cement column rig. (b) Binder is being initially mixed with water, pumped in-situ, before being pumped to the ring. (c) Mixing tool in action. Binder is being injected in slurry as the mixing tool penetrates the soil (Keller 1998).
2.7.1 Design of lime, cement and lime/cement columns

The objective of ground improvement using stabilised columns is to transfer most of the load from the original soil to the stiffer columns. The design theory currently used for lime-cement columns was initially designed for soft lime columns (Broms, 1984). The addition of cement to the lime columns was found to enhance the strength achieved by the stabilised soil and also enabled the stabilisation of organic soils, where lime columns proved to be inefficient. However, the addition of cement is shown to alter the mechanical properties of lime columns by yielding a higher strength and stiffness and also by affecting the failure strain. The higher the strength the lower the failure strain and the more brittle is the behaviour of the column. Therefore, the assumption of full interaction between the unstabilised soil and the columns, used in the design of lime columns, is not always justified.

In fact, Kivelo (1998) reported some cases where failure of lime-cement columns stabilised embankments occurred despite the fact that the factor of safety was sufficient. According to the same author, there are no design methods available in Sweden, at present for lime-cement columns with a high shear strength and high stiffness. The maximum shear strength used in design is generally limited to 100 to 150 kPa, for full interaction to occur between the columns and the original soil, regardless of the results obtained in the laboratory and field tests (Carlsten & Ekstrom, 1995). Some centrifuge model tests were carried out on cement-columns stabilised embankments (Kitazume et al., 1996) to investigate the failure mechanisms. The tests showed the cement-columns were very brittle and failed at low strains. Broms (1999) reported that failure strains for stabilised soil can be as high as 5% when the unconfined compressive strength of the columns is lower than 200 kPa and as low as 0.5% to 2% when the unconfined compressive strength exceeds 300 kPa. Failure strains of 0.1% to 1% have been reported by Tatsuoka et al. (1996) for cement columns. Therefore, if the failure strain of the stabilised column is low, the columns will fail before fully mobilising the peak strength of the unstabilised soil. As suggested by Broms (1999), a failure strain of 1% for a column of 0.6m diameter would only involve a lateral displacement of 10 to 30 mm. Therefore, in the light of these findings, it can be said that the current design method used for lime-cement columns is not applicable if the strength achieved is high and the failure strain low.
Because of the difference in the stress-strain properties of lime, cement and lime-cement stabilised soil, the stabilised columns are classified in Finland in low, medium and high strength columns. Low strength columns have a maximal undrained shear strength set to 100 kPa. Full interaction is assumed between the soil and the columns. Medium strength columns have a maximal undrained shear strength of between 100 kPa and 150 kPa. High strength columns have strengths in excess of 150 kPa. The same design methods are used for the high strength columns as for driven precast concrete piles. The contribution of the unstabilised soil between the columns is not considered.

According to Carlsten (1996), lime-cement columns are often dimensioned on the basis of the shear strength of the soil/admixture determined from unconfined compression tests. The maximum undrained shear strength used in design is assumed equal to \( \frac{q_u}{2} \); \( q_u \) is the unconfined compressive strength. Sometimes consolidated undrained triaxial tests are carried out when the confining pressure applied during the test corresponds to the estimated lateral pressure in-situ. As the strength of the lime/cement columns increases with time, after mixing, the extent of strengthening will depend on how soon the columns are loaded. Brandi (1983) showed that for clay and silty sands stabilised with lime, the strength increased linearly with the logarithm of time. Broms (1984) suggested that a preliminary estimation of the strength developed in the field could be made from results obtained in the laboratory as shown in Figure 2.17.

![Figure 2.17. Increase of shear strength with time (Broms, 1984).](image-url)
2.7.1.1 Failure criterion for stabilised soil

Lime-cement columns are usually designed using total stresses with the undrained shear strength given by the expression:

$$\tau_{fu,\text{col}} = c_{u,\text{col}} + \sigma_f \tan \phi_{u,\text{col}}$$

(2.1)

Where $\phi_{u,\text{col}}$ is the undrained angle of shearing resistance of the stabilised soil, $c_{u,\text{col}}$ is the undrained cohesion of the stabilised soil and $\sigma_f$ is the normal total pressure on the failure plane. According to Broms (1999), test results showed that the $\phi_{u,\text{col}}$ varies between 25 and 45° with occasional values as high as 60°. $c_{u,\text{col}}$ is usually assumed to be equal to 0.5 $q_u,\text{col}$, where $q_u,\text{col}$ is the unconfined compressive strength. Broms (1999) reviewed values of the ratio $2c_{u,\text{col}}/q_u,\text{col}$, as a function of the angle of shearing resistance. The ratio was found to decrease from 0.637 at $\phi_{u,\text{col}}=25^\circ$ to 0.414 at $\phi_{u,\text{col}}=45^\circ$.

Kivelo (1998) proposed a failure envelope for the stabilised soil as shown in Figure 2.18, which is similar to that proposed by Broms (1984). The assumed failure criterion was based on results of undrained triaxial and direct shear tests carried out on partially saturated samples obtained in-situ from lime-cement columns. At low stress the undrained shear strength can be represented by an apparent cohesion and angle of friction. However, when the normal stress exceeds a certain critical stress, the friction angle is reduced to zero and the maximum undrained shear strength can be assumed to remain constant.

![Figure 2.18. Assumed failure diagram of lime stabilised soil (Kivelo 1998).](image-url)
Broms (1984) considered that failure takes place along joint planes in unconfined columns and that the shear strength along the fissures will govern the compressive strength rather than the shear strength of the clay matrix. In the case of an expanding cavity, the radial stress at failure $\sigma_{rf}$ is given by the expression:

$$\sigma_{rf} = \sigma_{r0} + C_{u,soil} \left[ 1 + \ln \frac{E_{soil}}{2C_{u,soil}(1+\nu_{soil})} \right]$$  \hspace{1cm} (2.2)$$

Where $\sigma_{r0}$ is the initial total lateral earth pressure, $C_{u,soil}$ is the undrained shear strength of the soil, $E_{soil}$ is the elastic modulus of the soil and $\nu_{soil}$ is the Poisson’s ratio of the soil.

Hughes & Withers (1974) suggested, from a detailed examination of many field records of quick pressuremeter tests, that $1 + \ln 4$. Therefore, the total lateral stress at failure is given by the expression: $\sigma_{rf} = \sigma_{r0} + 4C_{u}$. Hansbo (1994) reported, for clays, that the elastic modulus is found to be within the range of 150$c_{u}$ to 500$c_{u}$ which, in undrained conditions (i.e. $\nu_{soil} = 0.5$) yields values of $\sigma_{rf}$ between $\sigma_{r0} + 5c_{u}$ and $\sigma_{r0} + 6c_{u}$. According to the same author, in most cases in practice, it is assumed that $\sigma_{rf} = \sigma_{r0} + 5c_{u}$. Kivelo (1998) suggest a factor of 4.5 such as $\sigma_{rf} = \sigma_{r0} + 4.5c_{u}$. But there still remains some uncertainty on the term $\left[ 1 + \ln \frac{E_{soil}}{2C_{u,soil}(1+\nu_{soil})} \right]$ and this suggests that it should be determined from load tests in the field. Broms (1999) introduced another term ($m_{soil} q_{0}$), $m_{soil}$ is being the stress concentration factor for the unstabilised soil and $q_{0}$ is the applied unit load, such as: $\sigma_{rf} = \sigma_{r0} + 5c_{u} + m_{soil} q_{0}$. Therefore, as the lime-cement column approaches failure, the lateral stress becomes the major principal stress and the expression of the vertical stress is given by:

$$\sigma_{v} = \frac{\sigma_{rf} (1 + \sin \phi)}{1 - \sin \phi}$$  \hspace{1cm} (2.3)$$

or

$$\sigma_{v} = (\sigma_{r0} + 5c_{u}) \left( \frac{1 + \sin \phi}{1 - \sin \phi} \right)$$  \hspace{1cm} (2.4)$$
For estimating the ultimate undrained compressive strength of the columns, Kivelo (1998) proposed a new failure criterion where two cases are considered:

(1) the normal stress $\sigma_n$ in the columns on the failure plane is less than a certain critical value $\sigma_{n.L}$ then:

$$\sigma_{u, col} = K_p \sigma_v + 2c_{u, col} \sqrt{K_p}$$  \hspace{1cm} (2.5)

Where $K_p$ is the coefficient of passive earth pressure and is equal to $\frac{1 + \sin \phi}{1 - \sin \phi}$.

(2) when the normal stress $\sigma_n$ in the columns on the failure plane exceeds the critical stress $\sigma_{n.L}$ then:

$$\sigma_{u, col} = \sigma_p + 2c_{u, col}\sqrt{K_p + \sigma_{c.L} (K_p - 1)}$$  \hspace{1cm} (2.6)

$\sigma_p$ being the passive lateral pressure and $\sigma_{c.L}$ the critical confining pressure.

The critical normal stress $\sigma_{n.L}$ on the failure plane can be evaluated from the following equation:

$$\sigma_{n.L} = \frac{c_{u, col} (\sqrt{K_p - 1}) + 0.5 \sigma_{c.L} (K_p - 1)}{\tan \phi_{u, col}}$$  \hspace{1cm} (2.7)

Therefore for $\phi_{u, col} = 0$, eq. 2.5 becomes:

$$\sigma_{f, col} = \sigma_v + 2c_{u, col}$$  \hspace{1cm} (2.8)

or

$$\sigma_{f, col} = q_{u, col} + (\sigma_{t0} + 5c_{u, soil})$$  \hspace{1cm} (2.9)

Where $q_{u, col} (= 2c_{u, col})$ is the unconfined compressive strength of the stabilised column.

And the long-term ultimate bearing capacity of lime/cement columns can be given by the expression (Broms 1999):

$$q_{col} = 2c_{col} \sqrt{K_p + K_p (\sigma_{v0} + 5c_{u, soil} + m_{soil} q_0 - u_{col}) + u_{col}}$$  \hspace{1cm} (2.10)
Where $u_{\text{col}}$ is the pore pressure in the column, $c'_{\text{col}}$ is the cohesion of the stabilised column, $\sigma_{v0}$ is the initial overburden pressure, $K_p$ is the coefficient of passive earth pressure.

Both lime-cement columns and the soil will contribute to the support of the applied load. The distribution of this load between the column and the unstabilised soil is calculated under the idealised assumption that horizontal sections remain horizontal (i.e. same settlements occur in columns and in unstabilised soil at each level). Therefore, the load is successively transferred to the columns and then to the bottom surface of the lime/cement columns. It is normally assumed that the columns behave as elastic, perfectly plastic materials as shown in Figure 2.19.

![Stress-strain diagram](image)

Figure 2.19. Assumed stress-strain relation of stabilised soil (Broms 1984).

The maximum stress in the stress-strain relationship for stabilised soil is commonly called "creep load". According to Broms (1999) the creep load of the columns is about 65% to 80% of the ultimate load. Broms (1984) suggested that in practice the creep load of lime columns after 1 year could be estimated from the unconfined compressive strength $c_{\text{compr}}$ at a curing age of 90 days of laboratory-mixed specimens, as follows:

$$\sigma_{\text{creep}}^{\text{col}} = (3 \text{ to } 5) c_{\text{lab}}^{\text{compr}}$$

(2.11)

However, this empirical correlation should be used with caution as it was established for Swedish soils and therefore cannot be extrapolated to other soils. According to Carlsten (1996), for the design of lime-cement columns, the ultimate strength, $\sigma_{\text{ult}}$, is calculated from the following empirical formula:

$$\sigma_{\text{ult}} = 2. c_{u,\text{col}} + 3. \sigma_h$$

(2.12)
Where $\sigma_h$ is the total horizontal pressure of the soil between the columns and $c_{u,\text{col}}$ is the undrained shear strength of the column. $\sigma_h$ can be set equal to the total original vertical pressure because of the lateral expansion of the columns when the binder is mixed in. The empirical relationship seems to be a conservative application of equation (2.5), with $\phi_{\text{col}} = 30^\circ$. The load applied on the surface ($q$) is carried by both the column ($q_1$) and the soil ($q_2$). The maximum load that can be carried by a single column is given by:

$$q_1^{\text{creep}} = 0.65 \cdot a \cdot \sigma_{\text{ult}}$$

(2.13)

Where $a = \text{column area ratio} = A/c^2$ (for columns placed in a square grid)

$A = \text{area cross-section of columns.}$

$c = \text{distance between column centres.}$

0.65 represents the creep load set at 65% of the ultimate load.

### 2.7.1.2 Estimation of the settlements

For an estimation of the settlements occurring in both the column and the unstabilised soil, this latter is assumed to follow the stress-strain law shown in Figure 2.20.

![Figure 2.20. Assumed stress-strain relation of unstabilised soft soils (Carlsten, 1996).](image)

For the calculation of the settlements in the unstabilised soil, the stress-strain relationship of the unstabilised soil is divided into three main regions of effective stress. A stress region
below the preconsolidation pressure $\sigma^'p$, a stress region comprised between $\sigma^'p$ and $\sigma^’L$ and a stress region higher than $\sigma^’L$. Therefore, assuming equal strain conditions between the stabilised column and the unstabilised soil, the settlement equation to be used for the calculation of the settlement in the unstabilised soil will depend on the term $\sigma^'v_0 + \frac{q_2}{1-a}$.

Where $q_2$ is the load taken by the unstabilised soil and $a$ is the column area ratio. More detail in this is given by Carlsten (1996).

The settlement in the column is given by the following expression:

$$S_{col} = \sum \frac{\Delta h}{a} \frac{q_1}{M_{col}}$$  \hspace{1cm} (2.14)

Where $S_{col} =$ settlement in the column

$\Delta h =$ layer thickness

$M_{col} =$ column compression modulus determined from oedometer tests.

$M_{col}$ can be estimated from the modulus of elasticity of the column $E_{col}$ and from Poisson’s ratio $\nu_{col}$ as follows:

$$M_{col} = E_{col} \frac{(1-\nu_{col})}{(1+\nu_{col})(1-2\nu_{col})}$$  \hspace{1cm} (2.15)

According to Broms (1999), for lime columns it is common to assume that $M_{col} =$ 50 to 150 $c_{u, col}$ and for lime-cement columns $M_{col} =$ 150 to 250 $c_{u, col}$. Ekström et al. (1994) showed that the back-calculated $M_{col}$ values were approximately five times larger than those determined in the laboratory. $E_{col}$ is determined from unconfined compression tests carried out on laboratory mixed samples. The $E_{col}$ determined in the laboratory is usually higher than those measured on samples recovered from the columns, therefore, Ekström (1994) suggests that $E_{col}$ should not be determined from laboratory prepared samples. $E_{col}/c_{u, col}$ is assumed to be about 200 for lime-cement columns and about 250 to 300 for cement columns (Carlsten & Ekström 1997).

The calculated settlements in the column are compared with settlements calculated in the clay. If the settlement in the column, $S_{col}$, is greater than in the unstabilised clay, $S_{clay}$, the calculated load should be reduced in the column and more load is gradually transferred to
the unstabilised clay until \( S_{\text{col}} \) becomes equal to \( S_{\text{clay}} \) (this procedure involves iteration calculations). If however, the settlement in the unstabilised clay is greater than in the column, the column cannot be subjected to any further loading and the settlement \( S_m \) that occurs in the column and the in the clay is equal to \( S_{\text{clay}} \).

Regarding the estimation of settlement with time, the lime columns are considered to act as vertical drains with regards to the dissipation of pore pressures induced by loading. According to Carlsten (1996), the permeability in the column is assumed to be 1000 times that of the unstabilised clay and the permeability of lime-cement columns is about 400-800 times that of the unstabilised clay. In a field test, where lime columns were used, Broms & Broman (1977) reported that the lime columns installed acted as vertical drains. Hansbo (1994) reported the use of lime columns for the stabilisation of soft normally consolidated clay. The permeability of the lime columns was measured in-situ by means of the pressure-permeameter and the results showed that the ratio of column permeability to clay permeability was about 200. The same author suggested that the permeability of the lime columns is generally assumed to be equal to 100-1000 times the permeability of clay. However, the assumption that \( k_{\text{col}} > k_{\text{soil}} \) is not always true, particularly for cement columns as reported by (Okumura, 1996). Due to this reduction in permeability, cement columns cannot be considered as vertical drains. Kitazume (2000) reported that the high rate of settlement observed in stabilised columns is due to the high values of coefficient of consolidation \( c_v \) measured on treated soils, due to a low coefficient of volume compressibility \( m_v \), rather than to a high permeability. According to that author's experience on Japanese soils stabilised with both cement and lime, the permeability of the treated soil is much smaller than that of the original soil. Ahnberg & Holm (1999) showed, on laboratory-mixed specimens, that the permeability decreased from \( 10^{-6} \) m/s in the peat to about \( 10^{-8} \) to \( 10^{-7} \) m/s in the stabilised peat, whereas for gyttja samples the permeability was in the same range for both stabilised and unstabilised gyttja.

The degree of consolidation \( U \) as function of time can be estimated from the expression suggested by Hansbo (1979):

\[
U = 1 - \exp \left[ \frac{-2 c_v t}{R^2 f(n)} \right]
\]  
(2.19)
Where $C_{vh}$ is the horizontal coefficient of consolidation, $R$ is the radius of influence of the columns, $t$ is the consolidation time, $r$ is the column radius, $n = R/r$, $k$ is the coefficient of permeability and $L$ is the length of the drainage path in the columns. Large-scale tests carried out in Sweden and Finland indicate that the effective radius of lime columns is 0.25m and that the disturbance of the surrounding soil from the installation of the columns is small in contrast with sand drains (Broms, 1984). According to the same author, the horizontal coefficient of consolidation $C_{vh}$ is often three to four times $C_y$. Carlsten & Eriksson (1995) found that the above equations agreed well with that interpreted from observed settlements in the field up to about 80% to 90% of the total observed settlement.

### 2.7.1.3 In-situ tests on stabilised columns

The difference in mixing efficiency between the field and the laboratory is likely to affect the strength achieved in the field. Therefore, there is a need to assess the strength of the columns as well as their homogeneity once the stabilisation work is performed in the field.

Site penetration tests are the most common means of determining the strength of the columns in the field. These tests include the common Cone Penetration Test (CPT) and the pull out resistance test commonly called “PORT”. In this test, a wire runs through the column to the surface and the undrained shear strength of the column is calculated from the net force needed to pull the probe up through the column. However, these tests are more suitable for soft to semi-hard columns. In addition to the penetration tests, tests can also be performed on core samples recovered from the column. In some cases, it is possible to check the quality of columns by extracting the whole column, thus rendering the visual inspection of the column possible. The columns are usually tested at different curing times: 7-28 days in Sweden and 28-56 days in Finland as reported by Halkola (1999). However, it is advisable to perform in-situ tests on the columns at a curing time close to the design age of the columns. Figure 2.21 shows different means of performing quality control on columns in the field.
According to Halkola (1998), the undrained shear strength of the column is related to the measured penetration resistance by dividing it by an empirical factor $N_c$ that varies from 10 to 15. One of the shortcomings of the CPT is that long columns (> 7m) cannot be tested as the CPT may come out of the column for high lengths. In Japan, the assessment of the strength of the columns is almost entirely based on core samples but the samples recovered in hard soils tend to be very disturbed, hence the in-situ strength is underestimated. The quality of the whole column can be checked using the column sampler shown in Figure 2.21 (c). The column sampler is driven into the soil to the depth of the column, then pulled up and split into two halves with the column in one half of the sampler.

Figure 2.21. (a) Column CPT. (b) Vane penetrometer. (c) Whole stabilised column being sampled. (d) Core samples being taken from stabilised column (Halkola, 1998).
2.7.2 Applications of the soil stabilisation technique

A large number of case histories has been reported in the literature covering a wide variety of soil stabilisation techniques. Recently soil stabilisation has also been used in some environmental projects for stabilising or fixing contaminated soils (Japerse & Ryan, 1992). Porbaha et al. (1998) reviewed the different areas of use of the dry mix technology. The authors listed several areas of applications such as: increasing bearing capacity, reduction of settlement, prevention of sliding failure and remediation of contaminated ground.

Soyez et al. (1983) performed a full-scale field test using lime columns to check their efficiency and also to assess whether the design methods usually used in practice agreed with the observed behaviour. A reference untreated area was left for comparison. The treated areas were loaded by an embankment constructed in stages. The observed behaviour showed that lime columns reduced the settlement of the treated area and also increased the rate of consolidation. The main assumptions of the Broms design method were found to be accurate.

Holm et al. (1983) used lime columns to reduce settlements under an embankment. The soil profile was composed of muddy clay underlain by very soft, medium sensitive varved clay. Settlement results showed that lime columns reduced the settlements by 60% in the clay layer compared with a vertical sand drain area.

Tatsuoka et al. (1997) reported some results obtained on the cement-stabilisation of the Trans-Tokyo Bay Highway project where both the dry and wet mix methods were used. The wet method was found to yield better quality and was easier to control. The deformation and strength characteristics of the cement-treated sand and clay in the field were found to be similar to typical sedimentary soft rocks.

Lin et al. (1999) reported the use of the cement deep mixing method to reduce settlements at bridge approaches. The soil treated was soft marine clay with a maximum layer thickness of 20m. The clay was characterised by a high compressibility and undrained shear strength of 10–20 kPa. Monitoring of the construction and post construction phase showed that the settlement and lateral movement of the soft clay treated with cement columns was reduced significantly. Furthermore, this technique allowed the construction of
a 5m embankment to its full height in a short period of time with acceptable post-construction total and differential settlements at the bridge approach.

Another case of using deep cement mixing method for treating soft soils was reported by Bergado et al. (1999). The soil treated was a 16m thick layer of soft Bangkok clay having an undrained shear strength varying from 10-25 kPa. The embankment height was 2.5m and the amount of cement used was 150kg/m$^3$. Quality control tests performed on the cement columns showed an undrained shear strength varying from 300 to 600 kPa. Settlement observations showed a settlement of 0.15 m to 0.35 m. The finite element predicted settlements for the treated and untreated ground were 0.38 m and 1.26 m respectively.

A very detailed field case history, on the use of cement stabilised columns on organic soils, was reported by den Haan et al. (2000). The field test comprised the construction of an embankment to a height of 5 m over subsoil consisting of organic clay and peat down to a depth of 9 m. The strength achieved in the field was found to be much lower than estimated from laboratory-mixed specimens. Most of the stabilised columns tested were found to have one or more zones with low strengths suggesting that the stabilised columns were not uniform. The striking finding of this study was the case where the column strength, at 28 days curing time, decreased by almost 50% after $10^3$ days. According to the authors, this is believed to be possibly due to acid attack.

Andersson et al. (2000) reported the use of a combination of mass stabilisation and stabilised columns in a road stabilisation project. These soil improvement methods were required to take a 2.5m high trial embankment on a clay and peaty soil. The mass stabilisation was formed in the peat and the stabilised columns in the clay. The peat had very low undrained shear strength, about 5 to 10 kPa. After stabilisation, the strength of the stabilised peat varied between 40 to 150 kPa at 30 days curing age. The settlements measured were about 300 to 400mm depending on the embankment height. After reducing the embankment height, settlements were found to practically stop.
2.8 SUMMARY

A literature review of the main chemical reactions occurring between lime, cement and the original soil was carried out. It was shown that the lime reacts with the clay according to a succession of four mechanisms: (1) cation exchange, (2) flocculation and particle aggregation, (3) lime carbonation and (4) pozzolanic reactions between lime and both silica and alumina present in the clay. Lime stabilisation was found to be inefficient in organic soils because of their very low clay content. Both lime and cement stabilisation of clay were shown to improve the strength of the original soil. Products other than cement and lime are being considered for soil stabilisation. Gypsum and blast furnace slag, in particular, are increasingly used in combination with cement for the stabilisation of organic soils. The organic matter was found to affect the hydration of cement products by retaining the calcium ions released during the hydrolysis of cement. The humus substances contained in the organic matter are believed to form complexes with the calcium. The humus content of an organic soil could possibly be used as a criterion for the assessment of the extent of reaction to stabilisation.

Reported data on the mechanical properties of stabilised soils showed that the shear strength is dependent on the type of stabiliser used. Cement based stabilisers tend to yield higher strengths and more brittle behaviour in compression than lime or lime-cement columns. The angle of shearing resistance and the cohesion usually increase after stabilisation. Total angles of shearing resistance reported in the literature for both stabilised clay and organic clay are between 34° to 44° and 44° to 45°. The failure strength was found to be independent of the normal stress at high stress levels.

The current applied design theory of lime-cement column was reviewed. This theory was designed initially for soft lime columns, although, the increasing use of cement and other binders can result in a more brittle stabilised material than obtained in normal lime stabilised material. Therefore, the design theory used for lime columns can no longer be applied as the original soil and the stabilised columns do not fully interact. Furthermore, the assumption of stabilised columns acting as vertical drains is not verified for lime-cement and cement columns. The permeability of the former columns is generally found to
be of the same order as that of the original soil whereas, the permeability of cement columns is lower than that of the original soil.

Reported case histories on soil stabilisation showed that this technique has been used successfully in a variety of projects on different types of soft soils. The main application reported was for the construction of embankments on soft soils. The use of the soil stabilisation technique resulted in a reduction of the settlements in the treated soil. Most of the current case histories refer to the stabilisation of inorganic soils, however results from embankments on stabilised organic soils are becoming available from the EuroSoilStab project. That project has shown that organic soils can be stabilised satisfactorily provided significant binder is used. There remains, however, some uncertainties regarding the long-term chemical stability and leaching of the stabilised organic soil.
CHAPTER 3

SOIL PROPERTIES
3. SOIL PROPERTIES

3.1 INTRODUCTION

This chapter gives a description of the soil conditions encountered at the two sampling sites selected for this project: Raheenmore Bog and Ballydermot Bog. Both are raised bogs and are part of what is known as the Bog of Allen, which is located in the Irish Midlands. It was initially intended to use Raheenmore Bog as the major site for the project due to the availability of data on the soil from a joint Irish-Dutch project, which commenced on the site in October 1989. However, a location could not be found from which sufficient peat could be obtained to fill the large-scale testing chamber (see chapter 5) without environmental implications. It was therefore necessary to source the peat from Ballydermot Bog.

This chapter presents a description of the sites and the results of classification tests along with other laboratory tests such as triaxial, oedometer, permeability and ring shear tests carried out on samples of peat recovered from both locations.

3.2 SITE DESCRIPTIONS

3.2.1 Location of sites

Both sites are located in the Irish Midlands. Raheenmore Bog is located in Co. Offaly. It is approximately 200 ha in size and is 7 km south of Tyrrellspass and 5 km north of Daingean. Raheenmore Bog was designated a National Natural Reserve in 1970 and is under the management of the national parks and Wildlife Service (office of Public Works). It was listed as being of international importance in 1981 by An Foras Forbartha mainly because it is the largest remaining example of the Midland raised bog type.

Ballydermot Bog, located in the Glashabaun area of Co. Offaly, is approximately 8 km north of Rathangan and 6 km south of Edenderry. A power station is located 4.5 km to the west of the bog. Ballydermot Bog is a cut-away bog in which development and drainage
Figure 3.1. Location of the sampling sites.
Figure 3.2. Raheenmore Bog.

Figure 3.3. Ballydermot Bog.
3.2.2 Geology of the sites

Both sites are located, geographically, in the central Irish lowlands region, which consists mainly of lower carboniferous limestone bedrock overlain by glacial deposits. The raised bogs of central Ireland commenced their existence soon after the ice-sheets had melted in the central lowlands. The peat formed in lake beds and waterlogged depressions where the land was washed by nutrient-rich groundwater providing ideal conditions for the growth of a fen vegetation around 9,000 years ago. At first, beds of reed developed but as their dead remains accumulated they were replaced by rushes and sedges, which eventually covered the lake and grew above the water table. Further growth above the water table was from nutrients derived from rainwater, which encouraged the growth of certain species of sphagnum able to survive under such conditions. The raised bogs took over from the fens around 7,000 years ago and are characterised by a convex surface sloping gently towards the periphery. Typical schematic representation of the stratigraphy of a Midlands raised bog is shown in Figure 3.4. A classical profile of a raised bog in the Irish Midlands is also shown in Figure 3.5.

Figure 3.4. Schematic representation of the stratigraphy of a Midlands raised bog. (Mitchell 1990).
3.2.3 Ground conditions

In Raheenmore Bog, the peat layer reaches depths down to 15 m towards the centre and is underlain by lacustrine clay and glacial till. The lacustrine layer consists mainly of clays, which are generally silty but occasionally sandy or gravely. The sand and gravel content increases towards the edges of the site, corresponding to the original lakeshore. The thickness of the lacustrine clay layer varies from 0 to 5.6 m with an average depth of 3 m. The typical peat layer stratigraphy is fen/woody peat overlain by strongly humified sphagnum peat topped by poorly humified sphagnum peat (Rijsdijk and Van der Meer, 1990). At the time of sampling, the water table was at about 15-20 cm below the ground surface.

The original average peat depth in Ballydermot Bog was 7 metres. Following production, drainage and shrinkage over the years, only 4 m of peat remain at present. The peat layer is underlain by shelly marl and boulder clay. The typical stratigraphy of the site would be boulder clay overlain by shelly marl, reed swamp peat topped by old sphagnum moss. At the time of sampling, the water table was at 2.5 m below the surface of the vertical face at
Ballydermot Bog. The vertical face was separated from the cut-away area by a drainage ditch as shown in Figure 3.6.

Figure 3.6. Ballydermot Bog. Vertical face from where peat samples were recovered.

( Drainage ditch separating the vertical face from the cut-away area is shown as well).

3.3. SOIL PROPERTIES

Undisturbed peat samples were taken at a depth of 0.5 m-1.5 m in Raheenmore Bog and 0.5 m-2.5 m in Ballydermot Bog using the U-100mm diameter piston sampler. Samples were carefully waxed and stored in the laboratory. Undisturbed block samples, 1x1x1m in dimension, were also taken from the latter site. These large peat samples were obtained from a vertical face 2.5m high in the cut-away bog. Raheenmore peat was brown and fibrous whereas Ballydermot peat was brown and amorphous. Soil identification tests were carried out in accordance with ETC 5 testing procedure. Details of Raheenmore peat properties are shown in Table 3.1.
Table 3.1: Physical and chemical properties of peat tested

<table>
<thead>
<tr>
<th>Properties</th>
<th>Raheenmore peat</th>
<th>Ballydermot peat</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sampling depth</td>
<td>[m]</td>
<td>0.5 – 1.5</td>
</tr>
<tr>
<td>Bulk density</td>
<td>[t/m³]</td>
<td>1.05 – 1.1</td>
</tr>
<tr>
<td>Natural water content</td>
<td>[%]</td>
<td>1200 – 1400</td>
</tr>
<tr>
<td>Von Post classification</td>
<td>[H₆]</td>
<td>H2</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Organic content</td>
<td>[%]</td>
<td>98 – 99</td>
</tr>
<tr>
<td>Specific gravity</td>
<td></td>
<td>1.4</td>
</tr>
<tr>
<td>pH</td>
<td></td>
<td>5.3</td>
</tr>
<tr>
<td>Cation exchange capacity</td>
<td>m.e/100g</td>
<td>50.45</td>
</tr>
<tr>
<td>Calcium</td>
<td>[Mg/l]</td>
<td>1230</td>
</tr>
<tr>
<td>Magnesium</td>
<td>[Mg/l]</td>
<td>1170</td>
</tr>
<tr>
<td>Potassium</td>
<td>[Mg/l]</td>
<td>108</td>
</tr>
<tr>
<td>Sodium</td>
<td>[Mg/l]</td>
<td>286</td>
</tr>
</tbody>
</table>

The ion content of the soil was determined using atomic adsorption.

A profile of soil properties with depth for Ballydermot peat is shown in Figure 3.7.

![Figure 3.7. Profile of soil properties with depth for Ballydermot peat. (The transitional layer between peat and shelly marl is clearly discernible at 4m deep). Samples recovered using a hand auger.](image)
The classification tests carried out show that both peats are highly organic with organic content greater than 90%; however, the degree of humification differs. Ballydermot peat is at a stage of nearly complete decomposition whereas Raheenmore peat is very slightly decomposed. Visual inspection of Ballydermot peat samples showed the presence of some reeds in the peat at depths of 1.5m to 2.0m.

The liquid limit for both peats could not be determined, as readings obtained in the penetrometer apparatus were not consistent. For the same sample tested, a difference of 4mm penetration was sometimes obtained at the same water content due to the non-homogeneity of the peat and also due to the non-uniformity of the fibre distribution throughout the sample. Some relationships between liquid limit and organic content have been reported in the literature (Miyakawa 1960; Skempton & Petley 1970). Skempton & Petley (1970) found for a buried fen peat from Kings Lynn and Avonmouth that \( \omega_L - 50 = 5N \) with \( \omega_L \) being the liquid limit and \( N \) is the loss in ignition. Hobbs (1986) quotes values between 200 and 600% for fen peat and between 800 to 1500% for bog peats.

A certain quantity of clay must be present in the soil in order to determine the plastic limit. Both peats tested are characterised by a high organic content rendering the determination of plastic limit very difficult. Furthermore, according to Hobbs (1986), the numerical value of plastic limit of the peat is of little interest as no correlation has been found between the plasticity index and any other properties.

The capacity of both peats tested to absorb calcium was measured. This was assessed by determining the amount of calcium removed from a saturated solution of Ca(OH)\(_2\) when it is shaken up with 50g soil sample. The amount of calcium absorbed per 100g of dry soil was 18.24g for Raheenmore peat and 7.27g for Ballydermot peat. The capacity of Raheenmore peat at absorbing calcium is therefore twice higher than for Ballydermot peat.

A simple method for the determination of the humic acid content was developed by the author. The method developed involved an extraction with NaOH followed by a precipitation of humic acids by lowering down the pH to 1 using HCl. The precipitate collected was dialysed till chloride free and eventually the retentate was freeze-dried. However, one of the shortcomings of this method was that the humic acids content
measured is only a crude estimation, as the product of freeze-drying will contain other co-precipitated non-humic material. In order to obtain accurate values of humic acid content as well as other materials such as bitumen and fulvic acids, an analysis in the XAD-8 resin is necessary, as the different components contained in the humus are separated in this process. However, Yamazaki et al. (1989) reported that each type of organic matter (humin, humic acid and bitumen) excluding fulvic acid could be calculated, knowing the ignition loss (Li in %) of the organic soil, according to the following correlations:

\[
\text{Humin (\%)} = 3.05 \times 10^{-3} \times \text{Li}^{2.05}
\]
\[
\text{Humic acid (\%)} = 9.34 \times 10^{-1} \times \text{Li}^{0.78}
\]
\[
\text{Bitumen (\%)} = 9.44 \times 10^{-3} \times \text{Li}^{1.46}
\]

Applying these correlations for Raheenmore (RA) and Ballydermot (BA) peat yields the following concentrations.

<table>
<thead>
<tr>
<th></th>
<th>Humin (%)</th>
<th>Humic acid (%)</th>
<th>Bitumen (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RA peat</td>
<td>36.8</td>
<td>33.4</td>
<td>7.6</td>
</tr>
<tr>
<td>BA peat</td>
<td>35.3</td>
<td>32.8</td>
<td>7.3</td>
</tr>
</tbody>
</table>

### 3.3.1 Chemical analysis of the pore water

The chemical composition of the soil pore water is considered to have a big influence on the results achieved in soil stabilisation, particularly when dealing with organic soils. For this purpose, a chemical analysis of the main ions present in the peat pore water was carried out.

The pore water was extracted in the triaxial cell by applying an isotropic pressure on the soil specimen and the out-flowing pore water was collected into a bottle, which was sealed immediately. The pore water collected was filtered through a 0.45\(\mu\)m pore size filter, which is the conventional applied size for the separation of the solid and dissolved phases, and analysed for \(\text{Ca}^{2+}, \text{Na}^+, \text{K}^+, \text{Mg}^{2+}, \text{Cl}^-, \text{SO}_4^{2-}\). The concentration of cations \(\text{Ca}^{2+}, \text{Na}^+, \text{K}^+\) and \(\text{Mg}^{2+}\) was measured using atomic absorption spectrophotometry and concentration of \(\text{Cl}^-\) and \(\text{SO}_4^{2-}\) using ion chromatography. Laboratory water was analysed
along with peat pore water to allow for comparison and assessment of the effect of the pore water on the binder reactions. Results of the chemical analysis on Ballydermot peat pore water are given in Table 3.3.

From the chemical analysis, the concentration of sulphate was found to be nearly fourfold higher in the peat pore water than in laboratory water. This could be explained by the fact that in bogs with high water table, sulphate is reduced under anaerobic conditions but when the bog is drained, which is the case in Ballydermot bog, anaerobic conditions do not prevail and this causes the sulphate to be accumulated in the peat layer above the water table. Another source could possibly be the pollution by sulphuric acid derived from the emission of sulphur dioxide from the fire peat station located nearby the site. Some ashes were observed on the ground surface at Ballydermot.

Table 3.3. Chemical composition of both Ballydermot peat pore water and laboratory water.

<table>
<thead>
<tr>
<th></th>
<th>Peat pore water</th>
<th>Laboratory water</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ca (mg/l)</td>
<td>15</td>
<td>22.2</td>
</tr>
<tr>
<td>Na (mg/l)</td>
<td>17</td>
<td>6.55</td>
</tr>
<tr>
<td>K (mg/l)</td>
<td>3.3</td>
<td>0.85</td>
</tr>
<tr>
<td>Mg (mg/l)</td>
<td>4.8</td>
<td>2.3</td>
</tr>
<tr>
<td>Cl (mg/l)</td>
<td>18.76</td>
<td>13.74</td>
</tr>
<tr>
<td>SO₄ (mg/l)</td>
<td>44.25</td>
<td>14.27</td>
</tr>
</tbody>
</table>

A feature of the peat pore water is the presence of humus substances, which are known to exhibit a buffering over a wide pH range. This buffering capacity is of considerable practical significance, as it will give information on the amount of hydroxide from the binder that will be neutralised by the pore water. In general, soils that are rich in humus are well buffered.

A base titration method was used to assess the buffering capacity of the pore water with a 0.05M NaOH solution. The pH of pore water was measured with increasing amounts of titrant. Laboratory water was also used for a better illustration of the buffering capacity of
pore water. Typical titration curves obtained are shown in Figure 3.8. The gradual rise in pH with added base attests to the high buffering capacity of pore water compared with laboratory water. Buffering is shown over a wide pH range.

Figure 3.8. Titration curves of peat pore water and laboratory water.

3.4. ELECTRON MICROSCOPE ANALYSIS OF PEAT

Scanning electron microscope pictures of the Ballydermot virgin peat were taken using a P.G.T energy dispersive X-ray spectrometer, which allows samples to be mounted for scanning without any preliminary preparation. Typical scanning electron microscope pictures of Ballydermot peat are shown in Figure 3.9.
Figure 3.9. Electron microscope pictures of Ballydermot peat. (a) to (d) Very porous cellular microfabric. (e) Stems cylindrical in shape and appear to have a wide range of sizes. (f) Stem cross sectional. (g) and (h) Material sheet-like in shape.
The scanning electron microscope (SEM) pictures taken on Ballydermot peat specimens show a very porous cellular micro fabric, which has a sponge-like structure and therefore is readily pervious to water flow. The large volume of voids confirms the high capacity for water holding of the peat. The approximate diameter of the voids in the microstructure of peat is \( \approx 10-20\mu m \). Peat fibres examined were found to range from 0.3mm for the shortest to 3.7mm for the longest. Quantitative-XRD analysis on peat fibres showed that the chloride concentration was the highest among concentrations of (Mg, Al, S, K, Ca, Fe and Si). See Figure 3.10.

![Figure 3.10. Quantitative -XRD analysis of peat fibres.](image)

### 3.5. ONE-DIMENSIONAL COMPRESSIBILITY CHARACTERISTICS

The standard oedometer tests carried out on both Raheenmore and Ballydermot peat were performed on 76.2 mm diameter and 19.05 mm high soil specimens. The specimens tested were always left to soak in water 24 hours prior to testing and were loaded with a load increment ratio (LIR) of 1 and time increment duration of 24 hours. Some long-term consolidation tests were also performed to investigate the creep behaviour of peat. It should be noted that there was no control of the ambient temperature in the laboratory. Results of consolidation tests carried out on both Ballydermot and Raheenmore peat, plotted in terms of the void ratio \( e \) versus \( \sigma'_v \) (\( \sigma'_v \) being the vertical effective stress applied), are shown in Figure 3.11.
The following features can be observed from Figure 3.11:

(i) High initial void ratio characterising both peats, $e_0 \approx 12$ for Ballydermot peat and $e_0 \approx 21.1$ for Raheenmore peat,

(ii) The slope of the normally consolidated part is not linear. The coefficient of compression $c_c$ decreases with increasing vertical effective pressure $\sigma'_v$, therefore it is difficult to define a characteristic value of $c_c$ from the $e - \log \sigma'_v$ curve,

(iii) There is an apparent preconsolidation pressure of $\approx 15$ kPa for Ballydermot peat and less than 5 kPa for Raheenmore peat.

The ground water level in peat deposits is in general close to the surface, and since the weight of peat may be less than that of water, the effective overburden pressure is generally negligible. In the case of Raheenmore peat, the ground water level at the time of sampling was at 10-20cm below the surface level. As Raheenmore Bog is one of the few remaining undisturbed bogs in the Irish Midlands, the preconsolidation pressure of the peat is more likely to be negligible and this is confirmed by the results obtained in the oedometer.
The preconsolidation pressure, of \( \approx 15 \) kPa, measured on Ballydermot peat is quite high for a peat deposit, however the samples tested were taken from a vertical face exposed to water table drawdown. The development of cut-away bogs is generally accompanied by a lowering of the water table using drainage ditches. The Ballydermot peat samples tested in this project were recovered from a vertical face separated from the cut-away area by a drainage ditch as shown in Figure 3.5.

The coefficient of compression \( C_c \), determined from the slope of the normally consolidated part of the \( e \)-\( \log \sigma' \) curve reflects the non-homogeneity of the peat material and confirms the difficulty in defining \( C_c \). Therefore, an approximate value of \( C_c \) has been estimated from the linear part of the normally consolidated portion of the \( e \)-\( \log \sigma' \) curve. The Calculations of the coefficient of compression yielded an average value of \( C_c = 6.12 \) for Ballydermot peat and \( C_c = 6.5 \) for Raheenmore peat. The ratio \( C_c/(1+e_0) \) was found to be equal to 0.45 for Ballydermot peat and 0.3 for Raheenmore peat.

Figures 3.12a and b show typical curves of settlement versus log time obtained at two different ratios of \( \frac{\sigma_1^\prime}{\sigma_p} \) (i.e. ratio of the effective consolidation pressure to the preconsolidation pressure). The settlement versus log time curves did not always display the classical S-shape, which shows clear demarcation between the primary and secondary compression.

The shape of the settlement versus log time curve appears to be dependent on the stress increment applied. For small pressure increments, around the preconsolidation pressure, the settlement versus log time curves seem to follow the Leonard & Girault (1961) type III curve in which there is no reversal of slope at or near the end of the primary stage. Whereas, for large stress increments, exceeding the preconsolidation pressure, the settlement versus log time curves seem to follow the Leonard & Girault (1961) type II curve. Therefore, the end of the primary consolidation could not be determined from the settlement versus log time curves.
Figure 3.12. (a) and (b) Settlement versus time for Ballydermot peat samples.

Work carried out by Collesili & Cortellazo (1997) on Italian peat showed that the root time method underestimates the time to the end of the primary consolidation compared with that estimated from pore pressure dissipation. However, Mesri et al. (1997) claimed that the estimation of the end of primary consolidation from the log time method, when possible, is almost identical to that indicated by dissipation of pore pressure measurements. Al-Ibrahim (1987), on a study on some Irish organic silt soils, showed from consolidation tests in the Rowe cell apparatus that the root time method gave a reasonable estimate of the primary consolidation as did the log time method when this could be used. For this study, it was decided to use the root time method when the log time method could not be used.
The coefficient of consolidation \( c_v \) was determined from the root time method as the main settlement curves did not exhibit the typical S shape required for the \( \log t \) method. Values of \( c_v \) ranged from 0.18 to 8.87 m\(^2\)/year for Ballydermot peat and 0.23 to 6.6 m\(^2\)/year for Raheenmore peat. Similar values of \( c_v \) were found for Ballydermot peat samples at depths of 0.25 and 0.6 m, however, at a depth of 1.0 m, \( c_v \) was found to be less than half that measured at depths of 0.25 m and 0.6 m for the same range of stress. The coefficient of consolidation \( c_v \) was found to vary with the effective consolidation pressure with the same trend for both peats. It decreased sharply as the effective consolidation pressure increased beyond the preconsolidation pressure- see Figure 3.13.

![Figure 3.13. Variation of the coefficient of consolidation with the vertical effective stress.](image)

The creep behaviour of both peats tested was examined in standard incremental oedometer tests with load duration of 24 hours and in long-term consolidation tests with load duration of one month. The coefficient of secondary compression, defined here as \( C_\alpha (= \Delta e/\Delta \log t) \) was determined from the slope of the \( e-\log t \) curves after the end of primary consolidation as assessed by the root time method. Figure 3.14 shows the variation of \( C_\alpha \) with the vertical effective pressure. All three tests carried out on Ballydermot peat showed consistent behaviour; \( C_\alpha \) is shown to increase, reaching a maximum value after the preconsolidation pressure and then starts decreasing.
The value of the ratio $C_a/C_c$ obtained from three consolidation tests carried out on Ballydermot peat was evaluated using the graphical method described by Mesri et al. (1987). From Figure 3.15 it can be seen that the best-fit line yields a value of $C_a/C_c = 0.055$. Mesri et al. (1994) reported values of $C_a/C_c = 0.06 \pm 0.01$ for peat deposits, which are in agreement with the experimental findings in this study.

Some long-term consolidation tests were carried out on Ballydermot peat to investigate the relationship between secondary compression and time. The tests were carried out at single...
load increments of 10 and 50 kPa. The choice of load increments applied was to investigate any effect in the relationship settlement-time with respect to the preconsolidation pressure of the peat. The load increments applied were kept for a period of 82 days. Results of the long-term consolidation tests carried out on Ballydermot peat are shown in Figure 3.16.

![Figure 3.16. Settlement versus time (semi-log scale) for Ballydermot peat.](image)

From the results of the long-term consolidation tests, creep settlement of the specimens tested is shown to continue over the whole duration of the test. A striking feature of the creep behaviour of the peat tested is that the settlement versus time curves seem to become steeper after a certain period of time suggesting an increase in compression. This behaviour has been reported by several authors (Dhowian & Edil 1981; Mesri et al. 1997 and Colleselli & Cortelazzo 1998).

Dhowian & Edil (1981) defined this increase in compression as a “tertiary compression”. However, Mesri et al. (1997) pointed out that this behaviour is more likely to be due to an imperfection of the laboratory testing procedure rather than an intrinsic property of peat. Variation of the ambient temperature as well as the biodegradation of peat with time could possibly mislead the interpretation of the secondary compression behaviour.
3.6. PERMEABILITY CHARACTERISTICS

Tavenas et al. (1983) in a very comprehensive experimental research work on the permeability of natural soft clays concluded that the most reliable method of determining the permeability of a fine-grained soil is in the triaxial apparatus. In the author’s experience it was found more practical to carry out permeability tests in the Rowe cell apparatus as it enables the investigation of the permeability of peat over a wide range of void ratios. Also, the non-uniform and high change in diameter that occurs during the consolidation of peat in the triaxial cell renders the determination of the permeability more difficult in that apparatus.

Tavenas et al. (1983) concluded in their review of the main existing relationships between the coefficient of permeability and the void ratio, that none of the relationships reported is generally valid for all soils but they suggested the use of the following one:

\[
\log k = \log k_0 - \frac{(e_0 - e)}{C_k}
\]

(3.1)

Where \( C_k \) is the permeability change index and \( k_0 \) is the coefficient of permeability at \( e_0 \).

The permeability of Ballydermot peat was measured in the Rowe cell apparatus. The peat was saturated with a back-pressure of 100 kPa and a differential head of 5 kPa was applied (i.e. \( i = 25 \)). Results obtained are shown in Figure 3.17.

![Figure 3.17. Void ratio e versus coefficient of permeability k for Ballydermot peat.](image-url)
As can be seen from Figure 3.17, the coefficient of permeability of Ballydermot peat follows the linear semi-log relationship reported by Tavenas et al. (1983). The permeability change index measured was equal to 5 and was thus lower than 0.5 $e_0$ (i.e. $e_0 = 17$) as found by Tavenas et al. (1983) for soft clays. However, Mesri et al. (1997) found that for peat deposits $C_k$ was close to $e_0/4$, which is in good agreement with findings in Ballydermot peat.

The permeability of peat is generally found to be anisotropic (Hobbs 1986; Lefebvre et al. 1984). Evidence in the literature (Inganas 1978; Rycroft et al. 1975) shows that for highly decomposed peat, the difference between the horizontal and vertical permeability is negligible. Therefore the horizontal permeability for Ballydermot peat could be assumed to be equal to the vertical permeability.

3.7. SHEAR STRENGTH PARAMETERS

A set of laboratory tests aimed at assessing the shear strength parameters of Raheenmore peat was previously carried out within the EuroSoilStab project. The testing procedure used and the test results are presented in Farrell & Hebib (1998). The main findings of this study may be summarised as follows:

(i) There was a high build up of pore water pressure, approaching the cell pressure, in undrained triaxial compression tests. The apparent effective peak angle of shearing resistance $\phi'$ was about 55°,

(ii) Peat samples tested in drained triaxial compression tests deformed in an almost one-dimensional manner up to an axial strain of 10%. Failure as defined by peak deviator stress was not reached at 35% axial strain,

(iii) An effective angle of shearing resistance $\phi'$ of about 38° was measured in both the direct shear box and the ring shear test. The direct simple shear test (DSS) yielded lower value of $\phi' = 31°$ using the assumption $\phi' = \tan^{-1} (t_f/\sigma'_n)$.

The concurrence of the results of the ring shear and direct shear box test would suggest that $\phi'$ measured is representative of the matrix whereas $\phi'$ measured in triaxial compression is
more representative of the matrix and the reinforcing effect of the fibres. The low value of \( \phi' \) measured in the direct simple shear test (DSS) has been explained by Farrell et al. (1998) following a finite element study of the behaviour of a peat sample in the DSS using the Plaxis finite element model. The results showed that \( \phi' \) (i.e. \( \phi' = \tan^{-1}(\tau_f / \sigma'_n)_{\text{max}} \)) underestimated the true \( \phi' \) of peat. Furthermore, the finite element analysis indicated that, at failure, the horizontal and vertical effective stresses are about equal. This implies that for \( \Psi = 0 \) (angle of dilation), \( \sin \phi' = (\tau / \sigma'_v)_{\text{max}} \) at failure.

Figure 3.18 is illustrative of the different triaxial test results obtained on Raheenmore peat.

![Figure 3.18. Overall effective stress path for Raheenmore peat. (Farrell & Hebib 1998).](image)

In the light of the findings of the above tests on Raheenmore peat, it was considered that the triaxial and ring shear tests were the most appropriate to characterise the properties of Ballydermot peat. The ring shear tests were carried out in the Bromhead ring shear apparatus. Samples were isotropically consolidated at four different effective consolidation pressures of 20, 40, 60 and 80 kPa. The testing procedure used was as described in Harris & Watson (1997). The triaxial tests were carried out on samples of 38mm diameter and 76mm high. Filter drain paper was used during the tests to speed up consolidation. A back-pressure of 200 kPa was applied in all tests. Both consolidated undrained, with pore pressure measurement, and drained compression tests were performed. Both tests were carried out at three effective consolidation pressures of 40, 80 and 160kPa. A correction was accounted for membrane and filter paper effects according to B.S 1377.
Results of the ring shear tests, plotted in terms of the maximal shear stress versus the normal effective consolidation pressure, are shown in Figure 3.19. It can be seen that the angle of shearing resistance derived from ring shear tests is quiet low, $\phi' = 21^\circ$, compared to that measured on Raheenmore peat. An apparent cohesion was measured ($c' \approx 4.8$ kPa) but this could probably be due to testing procedure. The shear strength parameters derived from ring shear tests are believed to be representative of the matrix and therefore do not take into account the reinforcing effect of the fibres.

![Graph of maximal shear stress versus normal effective stress](image)

Figure 3.19. Maximal shear stress versus normal effective stress obtained from ring shear tests on Ballydermot peat.

The same behaviour as for Raheenmore peat was observed for Ballydermot peat under undrained triaxial conditions: (1) failure was not reached within the axial strain imposed, (2) there was high build up of pore pressure during the test, approaching almost the cell pressure, (3) a high angle of shearing resistance measured, $\phi' = 68^\circ$, (4) high bulging and distortion of the samples was observed at the end of the tests, as shown in Figure 3.21.

The effective stress path obtained from the consolidated undrained compression tests (CUTC) is shown in Figure 3.20. The most striking feature that can be observed is the high build up of pore pressure shortly after the start of the test. The ratio of excess pore pressure to the deviatoric stress (i.e. $\Delta u/q$) was found to be equal to 1.7 at 1% axial strain and even 2.8 for the sample consolidated at 160 kPa. This explains, quiet well, the initial sharp veer to the left of the effective stress.
Figure 3.20. Effective stress path for Ballydermot peat. Isotropically consolidated undrained triaxial compression tests.

Figure 3.21. Ballydermot peat sample after testing at 20% axial strain. Observe the high distortion that has occurred.

The stress-strain curve obtained from the consolidated drained triaxial compression (CDTC) tests did not display any peak strength within the range of axial strain imposed and the stress-strain behaviour was similar to that of a work hardening material; therefore peak strength could only be assumed at some arbitrarily strain- see Figure 3.22. The considerable distortion of the samples, observed during the tests, was such that the derived shear strength parameters for Ballydermot peat are of questionable validity. Previous experience in testing Raheenmore peat (Farrell & Hebib 1998) showed the angle of shearing resistance of Raheenmore peat to increase as the arbitrarily selected failure strain was changed from 20% to 40% axial strain.
Figure 3.22. Stress-strain curves for Ballydermot peat. CDTC tests.

Measurement of the volume change during shearing showed the peat to deform almost in a one-dimensional manner up to an axial strain of 5% suggesting that little lateral deformation occurs at low strains- Figure 3.23. This behaviour is believed to be due to peat fibres, which prevent the peat sample from deforming laterally.

Figure 3.23. Volumetric strain versus axial strain for Ballydermot peat. CDTC tests.

The overall effective stress paths obtained from both CUTC and CDTC tests are plotted in Figure 3.24.
Figure 3.24. Overall effective stress path for Ballydermot peat.

Figure 3.25 shows the Mohr circle obtained from CDTC tests assuming peak strength at 20% axial strain.

Figure 3.25. Mohr-circle plotted from results of CDTC tests on Ballydermot peat.

3.8 SUMMARY

Two sampling sites were selected for this project: Raheenmore Bog and Ballydermot Bog, this latter site being the main source of peat for this soil stabilisation project. Both sites are located in the Irish Midlands and are both raised bogs. Basic soil identification tests
showed that both peats had high water content exceeding 800% and were highly organic, with organic content greater than 90%. However, the degree of decomposition differed. Ballydermot peat was at nearly complete decomposition stage whereas Raheenmore peat was very slightly decomposed. Both peats had an acidic pH, 5.3 for Raheenmore peat and 4.9 for Ballydermot peat. Electron microscope analysis of Ballydermot peat showed a very porous cellular micro fabric.

Consolidation tests carried out on both peats showed a very compressible behaviour. Ballydermot peat displayed an apparent preconsolidation pressure of about 15 kPa, which is quite high for a peat deposit. This preconsolidation pressure is more likely to be due to water table drawdown resulting from the development of the site for the production of milled peat. The coefficient of compression $C_c$ was found to be equal to 6.12 for Ballydermot peat and 6.5 for Raheenmore peat. The coefficient of consolidation ($C_v$) values for both Ballydermot and Raheenmore peat were found to range between 0.18-8.87 m$^2$/year and 0.23- 6.6 m$^2$/year. The coefficient of secondary compression $C_a$ ($= \Delta e/\Delta \log t$) was found to vary between 0.2-0.49 for Ballydermot peat and 0.12-0.3 for Raheenmore peat. A relationship $C_a/C_c = 0.055$ was obtained for Ballydermot peat.

The permeability of Ballydermot peat was found to vary in the range of $7 \times 10^{-8}$ to $3 \times 10^{-9}$ m/s, within the range of effective stresses applied. The $e$-$k$ (semi-log scale) plot was found to be linear. The permeability change index $C_k$ was found to be equal to 5, which is very close to $e_0/4$, a relationship suggested by Mesri et al. (1997).

The shear strength behaviour was investigated with both triaxial and ring shear tests. The behaviour in undrained conditions was greatly affected by fibre reinforcement. A high build up of pore pressure was observed during the tests. The effective angles of shearing resistance measured, when tested in undrained triaxial compression, were very high; 55° for Raheenmore peat and 68° for Ballydermot peat. The determination of the shear strength parameters in drained triaxial compression tests is really questionable for peat. No peak strength was measured within the range of axial strain applied. The high distortion of samples observed at high strains renders the calculation of stresses inaccurate. Ring shear tests always displayed peak strength, so the angles of shearing resistance derived were 21° for Ballydermot peat and 38° for Raheenmore peat. Theses angles are quite low compared to those derived from triaxial tests.
This chapter reviewed the main engineering properties of both peats. Parameters derived for Ballydermot peat are used as input parameters for the design and the numerical modelling of the large-scale laboratory test.
CHAPTER 4

LABORATORY TESTING OF STABILISED PEAT
4. LABORATORY TESTING OF THE STABILISED PEAT

4.1 INTRODUCTION

The preparation of the stabilised soil specimens in the laboratory should reflect the in-situ conditions of such soil when mixed and cured in the field with current installation plant, or must be such that the laboratory results can be related to field performance. This raises several questions such as mixing energy, dry or wet mixing methods, surcharge applied during curing and the degree of saturation. Much work has been done over the recent years in the comparison of laboratory and field methods for lime columns and this knowledge forms the basis of the methods described in this section. The author was part of a research team working within the EuroSoilStab project in which Trinity College was a member. For the soil stabilisation technique to be efficient, the further knowledge of the mechanical properties of the stabilised peat is necessary.

This chapter presents the mixing procedure used for the laboratory preparation of stabilised soil. The binder design is presented along with the experiences of other laboratories. The results of a full suite of laboratory tests, including consolidation, permeability and shear tests, carried out on cement-stabilised Ballydermot peat are presented.

4.2 TCD MIXING PROCEDURE

A literature review of the main laboratory procedures adopted by others for the preparation of laboratory-stabilised specimens is presented in Appendix I.

The method adopted for the preparation of the stabilised specimens for laboratory tests in this research project is that adopted within the EuroSoilStab project. This procedure was mainly based on Swedish and Finnish experience on soil stabilisation, which was aimed at providing a unique procedure, which allows reproducibility of samples and comparison of results obtained in the different laboratories.

The procedure involves mixing the peat for a duration of 5 minutes before adding the stabiliser, in a dry state, and mixing it with the peat for another 5 minutes. The mixer used
was a small dough-mixer as shown in Figure 4.1. After mixing, the stabilised peat was poured into plastic tubes 65 mm diameter and 320 mm high and let to cure under a surcharge of 18 kPa. This pressure was generally applied within 10 min after pouring the stabilised peat into the plastic tubes. Specimens for triaxial, permeability and oedometer tests were stored in plastic tubes of 101.4 mm diameter and 500 mm high and were also left to cure under a pressure of 18 kPa as well. All specimens were kept immersed in water during the curing period. A geotextile was placed at the top and bottom of all the specimens to allow free access to water at both ends. The specimens were cured at a temperature of 13-16°C.

Figure 4.1. Dough-mixer used for mixing peat and the stabiliser.

4.3 DESIGN OF BINDERS

Five different, commercially available, binders were used: cement, pulverised fly ash, lime, pelletized blast furnace slag and gypsum. The cement used was an Ordinary Portland Cement. The pulverised fuel ash was from coal combustion. The pelletized blast furnace slag used was composed of 95% raw slag and 5% clinker. The lime used was a quicklime. The chemical composition of the different binders, as provided by the manufacturers is given in Table 4.1.
Table 4.1. Chemical composition of the stabilisers used.

<table>
<thead>
<tr>
<th></th>
<th>SiO₂ (%)</th>
<th>Al₂O₃ (%)</th>
<th>CaO (%)</th>
<th>Fe₂O₃ (%)</th>
<th>MgO (%)</th>
<th>SO₃ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>20.9</td>
<td>4.2</td>
<td>63.9</td>
<td>2.94</td>
<td>3.13</td>
<td>3.06</td>
</tr>
<tr>
<td>Gypsum</td>
<td>6.57</td>
<td>3.21</td>
<td>36.19</td>
<td>1.28</td>
<td>2.43</td>
<td>49.47</td>
</tr>
<tr>
<td>Lime</td>
<td>1.5</td>
<td>0.45</td>
<td>94</td>
<td>0.25</td>
<td>0.55</td>
<td>0.4</td>
</tr>
<tr>
<td>Pfa</td>
<td>54</td>
<td>20.5</td>
<td>5.05</td>
<td>11</td>
<td>1.15</td>
<td>1.1</td>
</tr>
<tr>
<td>Bfs</td>
<td>33.04</td>
<td>10.48</td>
<td>42.99</td>
<td>0.66</td>
<td>7.77</td>
<td>0.98</td>
</tr>
</tbody>
</table>

pfa: pulverised fuel ash, Bfs: blast furnace slag.

Altogether seven mixtures were tested at binder amounts of 150, 200, 250 kg/m³ – dry weight of binder per soil volume. The composition of the mixtures used was as follows:

- **C**: cement
- **CPL (80, 20)**: cement 80% - pfa 20% - lime 3%
- **CPL (40, 60)**: cement 40% - pfa 60% - lime 3%
- **CPL (20, 80)**: cement 20% - pfa 80% - lime 3%
- **S**: blast furnace slag
- **SC (60, 40)**: slag 60% - cement 40%
- **SG (85, 15)**: slag 85% - gypsum 15%

The pfa used in this project was for the purpose of investigating the advantages of replacing cement. As the pfa does not possess hydraulic properties on its own, it requires lime to activate its chemical pozzolanic activity. The optimum lime content for pfa was determined following the procedure described in Sivapullaiah et al. (1985). An optimum lime content of 3% was found to produce maximum pH to pfa-lime mixture—see Figure 4.2. As the lime distributes in both soil and PFA, the lime content was calculated with respect to the soil and pfa weight. Due to the high organic content of the peats mixed, mixtures composed of lime alone were not tested, as the lime requires the presence of a certain amount of clay minerals (i.e. aluminates and silicates) in the soil for the pozzolanic reaction to occur.
Figure 4.2. Variation of the pH of the PFA-lime mixture with lime content.

The mixture of blast furnace slag and gypsum was modelled after work carried out by Odadjima et al. (1995) and den Haan (1998). Odadjima et al. (1995) have proposed a flow chart for selecting stabilisers based on the humic and fulvic acid content of the peat tested. Therefore, knowing the humic and fulvic acid content of the peat mixed, it is possible to select a stabiliser with the right SAC ratio (ie. SAC = \( \frac{(SO + AlO)}{CaO} \)).

As stated earlier in § 3.1, it was initially intended to perform the main tests on Raheenmore peat. Therefore, preliminary tests for the assessment of the strength increase due to stabilisation were performed on Raheenmore peat and results were already available when the decision was taken to source the peat from Ballydermot bog and use this latter as the major site for the project. Thus, it was decided to carry out preliminary tests on Ballydermot peat, only with the mixtures that performed best on Raheenmore peat (i.e. cement and blast furnace slag-gypsum mixtures).

The chemical composition of the mixtures used is given in Table 4.2.
Table 4.2. Chemical composition of the mixtures experimented.

<table>
<thead>
<tr>
<th></th>
<th>SiO₂ (%)</th>
<th>Al₂O₃ (%)</th>
<th>CaO (%)</th>
<th>Fe₂O₃ (%)</th>
<th>MgO (%)</th>
<th>SO₃ (%)</th>
<th>SAC</th>
<th>Blaine value cm²/g</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>20.9</td>
<td>4.2</td>
<td>63.9</td>
<td>2.94</td>
<td>3.13</td>
<td>3.06</td>
<td>0.11</td>
<td>3270</td>
</tr>
<tr>
<td>Cement/pfa/lime 80:20:3</td>
<td>27.56</td>
<td>7.47</td>
<td>54.95</td>
<td>4.55</td>
<td>2.75</td>
<td>2.68</td>
<td>/</td>
<td>3950</td>
</tr>
<tr>
<td>Cement/pfa/lime 40:60:3</td>
<td>34.18</td>
<td>10.73</td>
<td>43.18</td>
<td>6.17</td>
<td>2.35</td>
<td>2.28</td>
<td>/</td>
<td>3550</td>
</tr>
<tr>
<td>Cement/pfa/lime 20:80:3</td>
<td>47.42</td>
<td>17.25</td>
<td>19.64</td>
<td>9.39</td>
<td>1.56</td>
<td>1.5</td>
<td>/</td>
<td>3210</td>
</tr>
<tr>
<td>Bfs</td>
<td>33.04</td>
<td>10.48</td>
<td>42.99</td>
<td>0.66</td>
<td>7.77</td>
<td>0.98</td>
<td>0.26</td>
<td>3980</td>
</tr>
<tr>
<td>Bfs/cement 60:40</td>
<td>28.18</td>
<td>7.96</td>
<td>51.35</td>
<td>1.57</td>
<td>5.91</td>
<td>1.81</td>
<td>0.19</td>
<td>3410</td>
</tr>
<tr>
<td>Bfs/gypsum 85:15</td>
<td>29.06</td>
<td>9.38</td>
<td>41.97</td>
<td>0.75</td>
<td>6.96</td>
<td>8.25</td>
<td>0.42</td>
<td>3600</td>
</tr>
</tbody>
</table>

4.4 LABORATORY TEST EQUIPMENT AND TESTING PROGRAMME

Series of unconfined compression tests were carried out on both Raheenmore and Ballydermot peat using the binders listed in Table 4.2. The results obtained showed that the cement binder yielded the highest strength for the stabilised Ballydermot peat. This was therefore selected for further testing.

Unconfined compression tests were carried out at different curing times of 7 and 28 days for Raheenmore peat and 7, 28, 90 and 360 days for Ballydermot peat. Samples were removed from the water tank, trimmed to a height of 130mm and tested for unconfined compression strength at a rate of 2.6mm/min. It should be noted that sometimes the specimens tested, particularly those which achieved high strengths, had to be pushed out of the plastic tubes using a hydraulic press which can apply high forces. The load was measured by a means of a 500 kg capacity proving ring and the axial strain using a dial gauge. Local strain measurements were used to compare the stiffness measured in standard unconfined compression tests with that measured at mid section of the specimen. The
strains were determined by a means of a camera device that measures the relative displacement between fixed dots plotted on the sample over the original distance. Two rows of three dots located at 1/4, 2/4 and 3/4 of the sample length were used. The accuracy of measurement, precision and other details of the camera device used can be found in Gill (1999). The camera device was only used for one test.

Consolidation tests were carried out in a standard oedometer apparatus on 76.2 mm diameter and 19.05 mm high specimens. These were loaded with a load increment ratio (LIR) of 1.0 and time increment duration of 24 hours. Some long-term consolidation tests, with a time increment of one month, were performed to investigate the relationship between creep and time. Oedometer rings were lubricated before each test in order to minimise any bonding that may occur between the stabilised soil and the ring.

Constant head permeability tests were carried out in both the Rowe cell and in the triaxial cell. Rowe cell specimens were 76.2mm diameter and 19.05mm high. A back-pressure of 100 kPa was found sufficient for the saturation of the specimens mounted in the Rowe cell and a “B” value over 95% was easily achieved. Whatman N° 54 filter paper was used as an interface layer between the specimen and the porous stone, as leaching of the cement material frequently led to clogging of the porous stone. The permeability was measured at different effective stress increments. The specimens tested in the triaxial cell were 100mm diameter and 100mm high. Specimens were enclosed in a membrane and an all-round pressure was applied. It was necessary to apply a back-pressure of 250 kPa in order to achieve a satisfactory B value. This back-pressure limited considerably the range of effective stress pressures that could be applied (i.e. the maximum cell pressure that could be applied was 400 kPa thus permitting a maximum effective consolidation pressure of 150 kPa).

Triaxial tests were carried out on specimens of 101.4mm diameter and 200mm high. A digital Tritest 50 ELE triaxial testing frame was used. This frame has a microprocessor controlled drive unit that provides an interface for computer control and operator programming of speed. A GDS PC-controlled stress path triaxial testing system was used, which allows complex loading and stress paths to be applied to the sample. The GDS automated system is composed of an eight channel 16 bit data acquisition device and two controllers, one for the cell pressure and one for the back pressure and measuring volume
change. The controllers had a resolution of \(1\text{mm}^3\) for volume change and 1 kPa for the pressure applied. The pore pressure was measured at the base of the pedestal using a pore pressure transducer. The axial displacement was measured both locally and externally. The external axial displacement was measured by a means of an LVDT transducer whereas, the local axial displacement was measured over the middle third of the specimens height by a means of miniature LVDT's equally spaced and mounted at the circumference of the specimens.

LVDT's used in this project were miniature submerged LVDT's SM3 with a maximum travel of \(\pm 3\text{mm}\) similar to those used previously by Murphy (1996). The four LVDT's were housed within another four plastic holders at about 70mm vertically above the LVDT's holders. These latter were glued onto the membrane at 90° intervals around the sample and the holders were linked by flexible rubber bands that were pin-connected with the holders. See Figure 4.3. The radial strain was measured at mid-height of the sample by a means of a miniature LVDT mounted on a Wykeham Farrance radial strain belt as shown in Figure 4.3. The LVDT's used for measuring axial strains had a resolution of 0.001% strain over the middle third and an accuracy of 0.007% strain whereas the LVDT used for measuring the radial strain had a resolution of 0.002% strain over the circumference of the specimen and an accuracy of 0.0075% strain.

![Figure 4.3. Triaxial set-up and instrumentation used for local strain measurement.](image-url)
A preliminary check was carried out for all samples to assess the degree of saturation. Initial \( B (= \Delta u/\Delta \sigma_3) \) values measured indicated a poor saturation of the samples. Therefore, a saturation stage with a differential pressure of 5 kPa (i.e. difference between the cell and back pressure) up to a back-pressure of 300 kPa was applied over a period of 24 hours. "B" tests performed after the saturation stage yielded values over 90%. Thus, the pore pressure response was measured for consecutive cell pressure increments until the B factor remained constant at which stage it was assumed that there were no air voids present.

Specimens were allowed to consolidate under the relevant consolidation pressure overnight, however, in general the time for 90% consolidation, determined from the root time method was less than 2 hours. Therefore, saturation and consolidation of each sample was completed within 48 hours. The rate of shearing applied was 4.5% per day for both drained and undrained tests. This rate of shearing was found to be adequate, according to the criterion given by Bishop & Henkel (1962), for complete equalisation of pore pressures and also to capture the small strain stiffness data.

The testing programme used for the stabilised Ballydermot peat is shown in Table 4.3.

<table>
<thead>
<tr>
<th>Test</th>
<th>Cell</th>
<th>Curing time, (days)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unconfined compression test</td>
<td></td>
<td>7, 28, 90, 360</td>
</tr>
<tr>
<td>Oedometer test</td>
<td></td>
<td>28, 90, 180, 360</td>
</tr>
<tr>
<td>Permeability</td>
<td>Rowe cell</td>
<td>90, 360</td>
</tr>
<tr>
<td></td>
<td>Triaxial cell</td>
<td>90, 180</td>
</tr>
<tr>
<td>Triaxial</td>
<td></td>
<td>90</td>
</tr>
</tbody>
</table>

### 4.5 TEST RESULTS

#### 4.5.1 Unconfined compression tests

The unconfined compressive strength was determined on a total of 58 specimens comprising seven different applied mixtures. The unconfined compression tests were carried out at 7 and 28 days curing time for Raheenmore peat and 7, 28, 90 and 360 days
for Ballydermot peat. The results are shown in Figures 4.4 and 4.5. The type of mixture, proportion and mechanical indices of the specimens tested are given in Appendix III.

Figure 4.4. (a) Unconfined compressive strength at 7 days curing time. Raheenmore peat.

Figure 4.4. (b) Unconfined compressive strength at 28 days curing time. Raheenmore peat.
Figure 4.5. Unconfined compressive strength. Stabilised Ballydermot peat.

Figure 4.4 (a) shows the strength achieved at 7 days curing time for Raheenmore peat samples. The cement stabiliser yielded the highest strength. For a dosage of 150 kg/m³, a small increase in strength was achieved for all other binders. There is hardly any increase of strength with increase of binder dosage for CPL (20,80). The strength achieved for the mixtures containing blast furnace slag at 7 days curing time was lower compared to other binders. The 28-day strength for Raheenmore peat samples is shown in Figure 4.4 (b). There is a substantial increase in strength for all binders except for CPL (20,80). For the CPL mixtures, the achieved strength decreased with increase of pfa content in the stabiliser. Mixtures containing blast furnace slag showed a greater increase in the strength achieved between 7 and 28 days curing time.

The addition of cement to the blast furnace slag in a proportion of 40:60 did not seem to increase the strength, compared to the blast furnace slag alone. However, addition of gypsum almost trebled the strength achieved with blast furnace slag alone. When high amounts of stabiliser were used (i.e. 200 and 250 kg/m³), blast furnace slag-gypsum stabiliser reached higher strength than cement based stabilisers. The performance of the blast furnace slag based stabiliser was very poor for an amount of binder of 150 kg/m³.

Results obtained for the cement-stabilised Ballydermot peat showed an increase in strength with increasing amount of binder. There was a considerable increase in strength with time.
The blast furnace slag-gypsum binder was not successful in stabilising Ballydermot peat. There was barely any strength increase even after 365 days curing time- see Figure 4.5.

A typical unconfined compressive stress-strain curve of the stabilised peat is shown in Figure 4.6. The curve shows a brittle failure and the main failure mode observed was by crushing. For samples, which displayed high strength a bedding error was observed in the stress–strain curves and this has been corrected according to the procedure suggested by den Haan (1997).

![Figure 4.6. Stress-strain curve. Raheenmore peat – blast furnace slag (250 kg/m³) at 28 days.](image)

The stress–strain curve of an unconfined compression test with local strain measurement is plotted in Figure 4.7 against the standard method of strain measurement. A secant stiffness modulus, $E_{50}$, three-fold higher was measured when using local strain measurement.

In Figure 4.8 (a), the unconfined $E_{50}$ is plotted against the unconfined undrained shear strength. A correlation $E_{50} = \frac{q_f/2}{\varepsilon_{50}} \approx 100 \ c_u$ is obtained. This relationship shows little scatter, particularly at low unconfined compressive strength. The failure strains, defined as the strain at $q_{\text{max}}$, measured for the different mixtures tested are shown in Figure 4.8 (b). Figure 4.8 (b) shows that the failure strain generally decreases with the increase in unconfined compressive strength. Failure seems to occur at an average axial strain of about 2.5% for high strengths. Very brittle behaviour was observed for specimens that achieved high strength.
Figure 4.7. Stress-strain curve. Raheenmore peat-blast furnace slag/cement (200 kg/m³) at 28 days.

Figure 4.8. (a) $E_{50}$ versus $c_u$. (b) Failure strain versus $c_u$. Results obtained from unconfined compression tests obtained on stabilised Raheenmore and Ballydermot peat.
4.5.2 Electron microscope analysis of stabilised peat

Scanning electron microscope (SEM) pictures of the stabilised peat were taken to investigate the interaction at microscopic level between the peat and the binder and also to investigate the different microstructural developments occurring during curing. SEM pictures taken on the stabilised peat are shown below.
Figure 4.9. SEM picture Ballydermot peat/cement 200 kg/m³ at 1 day

a The picture shows product of hydrated C₃S, which consists of honeycomb-like structure.

b Product of hydrated C₃S in form of flower-like structure.

c Cement platelets covering some part of the organic matter. The distribution of cement is not even over the surface. Peat pores are still discernible.

d Cement platelets filling some peat pores.

e Cement grain having same diameter as the peat pores

f Peat pore

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Figure 4.10. SEM picture of Raheenmore peat/Bfs-gypsum 200 kg/m$^3$ at 1 day

a Formation of some amorphous gel probably aluminate gel after reaction of C$_3$A with calcium sulphate. Minute particles can be observed on the grain surface more likely to be ettringite rods growing up.

b Same grain as in (a) at higher magnification. Ettringite-phase rods growing out of the grain surface.

c Stabilised peat texture covered with hexagonal platelets, the open peat cellular microfabric seems to have disappeared.

d Formation of CSH crystals, platelets of CH and some ettringite phase formation rods.

e Detail of d. Ettringite formation rods forming a tight mesh.

f Ettringite needle-like shaped formation growing out of the surface creating what is called an “outer product” which fills the voids originally filled with water.
Figure 4.11. SEM picture Ballydermot peat-cement at 28 days curing time. 200 kg/m$^3$

- **a** Represents the surface of stabilised peat covered with dense CSH platelet texture. The peat pores are no longer discernible.

- **b** Detail of (a). CSH platelets covering the surface of the peat. Dimension of platelets exceeding 10 μm. Fractures between platelets seem to be filled with rods growing out of the platelets.

- **c** Detail of (b). Tight mesh formed by the rods growing out of the CSH platelets.
The microstructure of stabilised Ballydermot peat after mixing showed that the cement started hydrating and this was characterised by the hydration of C₃S compounds in the form of honeycomb-like structure. The peat pores were still discernible at 1 day curing time. Cement platelets were observed to start filling and covering some of the peat pores.

Scanning electron microscope analysis of the Raheenmore peat-blastfurnace slag gypsum mixture showed formation of ettringite rods-like shaped growing in the peat voids thus forming a very tight mesh of ettringite crystals. Ettringite rods are shown to grow up freely in the voids without encountering any obstacle thus reaching lengths of a couple of microns.

A P.G.T energy dispersive X-ray spectrometer was used to analyse the chemical composition of the main phases formed in stabilised peat after 1 day curing time. Results showed that the platelets were mainly composed of calcium and silicate, whereas the rods growing on the surface of the solid phase were composed of high proportion of Ca and Mg followed by nearly equal proportions of S, Si and Al.

The microstructure of stabilised peat at 28 days curing times showed a texture covered with hexagonal platelets with average sizes over 10μm. A tight texture seems to characterise the microstructure of the stabilised peat. In any case, no interaction between the stabiliser and the organic matter could be observed.

4.5.3 Chemical analysis of the stabilised peat pore water

4.5.3.1 Effect of peat pore water on the hydration of cement

As stated earlier in § 2.5, the organic matter affects the hydration of cement even if, to present, the mechanism whereby the organic matter retards the hydration of cement is not clearly understood. Two approaches were used in this project to investigate any retarding effects due to organic matter. The first was to measure the heat given off during hydration of cement and the second was to investigate the different phases formed after hydration of the cement. Both tests were duplicated with laboratory water as a reference.
The heat evolution during the hydration of the cement paste was determined by curing the cement-paste in insulated wooden boxes of 10 cm³. The insulating material used had a thermal conductivity of 0.027 W/m°K. The peat pore water was extracted as described in § 3.3.4 and was mixed with cement in a cement/water ratio of 0.5. The increase in temperature during hydration was measured by a means of thermocouples embedded in the cement paste. Ambient temperature was also measured during the test. The results obtained are shown in Figure 4.12.

![Temperature change over time](image)

Figure 4.12. Heat evolution of a cement paste with time.

The increase in temperature for the cement-peat water paste was higher than for the cement-laboratory water paste. The ambient temperature was almost constant for the period considered. The overall increase in temperature of the cement-peat water paste was of about 4° to 6°C compared with less than 4°C for the cement-laboratory water paste. Both temperature curves showed two peaks, one at 2 hours after mixing and the second occurred between 15 to 20 hours after mixing. There is a shift to the right, of about two hours, of the main temperature peak for the cement-peat water paste relative to that of the cement-laboratory water paste.

The different products formed by the reaction between the cement and both peat pore water and laboratory water were identified by X-ray diffraction as shown in Figure 4.13.
Figure 4.13. X-ray diffraction patterns. (a) Cement-peat pore water. (b) Cement-laboratory water. (CSH = Calcium Silicate Hydrates, CH = Calcium Hydroxide, E = ettringite).

The X-ray diffraction patterns observed for both cement pastes were very similar. The main hydrated compounds observed were Calcium Silicate Hydrates (CSH), Calcium Hydroxide Ca(OH)$_2$ and ettringite. The peak of Ca(OH)$_2$ was found to be more intense for cement-laboratory water paste than for cement-peat pore water paste. A similar trend was observed for CSH, although to a lesser extent.
Infra-red spectrometry tests were carried out on both cement-peat pore water and cement-laboratory water pastes at different curing times. At the predetermined curing ages, the hydration of cement was stopped by crushing the specimens, washing them with isopropyl alcohol acetone then filtered off and dried in the oven at 50°C for 24 hours. Results obtained are showed in Figure 4.14. The specimens tested were cement-peat pore water cured at 3 hours and 28 days and cement-laboratory water cured at 3 days.

The IR-spectrograph pattern followed by the three specimens was more or less the same. In the region of 3300 to 3700 cm\(^{-1}\), two peaks were observed. One very sharp peak at 3650 cm\(^{-1}\) and one medium intensity peak at 3420 cm\(^{-1}\). The peak at 3650 cm\(^{-1}\) was observed for both cement-peat water specimens but not for the cement-laboratory water specimen. The two peaks observed are associated with a hydroxyl ion (OH) free stretching. The lack of overlap for the three specimens within the band 1800 - 3600 cm\(^{-1}\) should be noted. Further peaks were observed at 1670, 1420, 1120 and 970 cm\(^{-1}\). The peak at 1670 cm\(^{-1}\) is due to absorbed water and the peak at 1420 cm\(^{-1}\) is associated with CO\(_3\). The remaining peaks at 1120 and 970 cm\(^{-1}\) are associated with SO\(_4\) and SiO\(_4\) stretching.

Figure 4.14. Infra-red spectrographs of both cement-peat water and cement-laboratory water pastes.
4.5.3.2 Chemical analysis of the stabilised peat pore water

The chemical composition of the stabilised Ballydermot peat pore water was assessed by pressing out the stabilised peat pore water in a triaxial cell. The stabilised peat sample was pressurised and the outflowing pore water collected in a bottle that was sealed immediately. The age of the stabilised soil was 90 days but some chemical analyses were carried out on younger samples to quantify the amount of calcium present in the pore water. The chemical composition was determined using the methods described in § 3.3.1. Table 4.4 shows the chemical composition of both virgin and cement-stabilised peat.

Table 4.4. Chemical composition of cement-stabilised peat pore water and peat pore water.

<table>
<thead>
<tr>
<th></th>
<th>Pore water - Ballydermot peat</th>
<th>Pore water - peat/cement - 90 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ca$^{2+}$ (mg/l)</td>
<td>15</td>
<td>33</td>
</tr>
<tr>
<td>Na$^{+}$ (mg/l)</td>
<td>17</td>
<td>58</td>
</tr>
<tr>
<td>K$^{+}$ (mg/l)</td>
<td>3.3</td>
<td>60</td>
</tr>
<tr>
<td>Mg$^{2+}$ (mg/l)</td>
<td>4.8</td>
<td>1.75</td>
</tr>
<tr>
<td>Cl$^{-}$ (mg/l)</td>
<td>18.76</td>
<td>57.27</td>
</tr>
<tr>
<td>SO$_4^{2-}$ (mg/l)</td>
<td>44.25</td>
<td>31.46</td>
</tr>
</tbody>
</table>

Figure 4.15 shows the variation of calcium concentration, in cement stabilised Ballydermot peat specimens, over different curing times of 1, 7, 28 and 90 days.

![Graph showing the variation of calcium concentration in cement-stabilised peat pore water with curing time.](image-url)

Figure 4.15. Variation of calcium concentration in cement-stabilised peat pore water with curing time.
The chemical composition of both peat and cement-stabilised peat pore water shows an increase in concentration of (Na\(^+\), Ca\(^{++}\), K\(^+\), Cl\(^-\)) and a decrease in concentration of (Mg\(^{2+}\) and SO\(_4^{2-}\)) in the cement-stabilised peat pore water. The variation of the concentration of Ca\(^{++}\) with curing time in the cement stabilised peat pore water increases, reaching a maximum concentration at 7 days curing time and then decreases to a constant concentration for curing times of 28 and 90 days.

From the concentration of calcium and the pH measured in the cement stabilised peat pore water, the degree of saturation of the pore water with Ca(OH)\(_2\) was assessed. The concentration of [OH\(^-\)] was estimated from the pH of the pore water. According to Huang (1997), the product of solubility of saturated Ca(OH)\(_2\) liquid is: \(K_{sp} = 4.44 \times 10^{-5}\). Table 4.5 shows concentrations of (OH\(^-\)) and Ca\(^{++}\) in the pore water.

Table 4.5. Solubility product of cement stabilised peat pore water.

<table>
<thead>
<tr>
<th>Age, (days)</th>
<th>pH</th>
<th>Ca(^{++}), (mmol/l)</th>
<th>(OH(^-)), mmol/l</th>
<th>Solubility product</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>9.5</td>
<td>1.57</td>
<td>1.58</td>
<td>(3 \times 10^{-9})</td>
</tr>
<tr>
<td>7</td>
<td>12.2</td>
<td>1.79</td>
<td>15.84</td>
<td>(4.49 \times 10^{-7})</td>
</tr>
<tr>
<td>28</td>
<td>12.25</td>
<td>0.87</td>
<td>17.78</td>
<td>(2.75 \times 10^{-7})</td>
</tr>
<tr>
<td>90</td>
<td>12.4</td>
<td>0.82</td>
<td>25.11</td>
<td>(5.17 \times 10^{-7})</td>
</tr>
</tbody>
</table>

From Table 4.5, it can be seen that the cement-stabilised pore water is not saturated with Ca(OH)\(_2\). Solubility products obtained at different curing times are far below the solubility product of saturated Ca(OH)\(_2\) liquid.

4.5.4 Compressibility characteristics of cement-stabilised peat

The compression characteristics were investigated at three different curing times of 28, 90 and 240 days. Figure 4.16 shows results of the standard incremental consolidation tests performed on cement-stabilised Ballydermot peat specimens.
Figure 4.16. Void ratio versus effective vertical stress for cement-stabilised Ballydermot peat specimens. Amount of binder 200 kg/m³.

The apparent preconsolidation pressure increased from 18 kPa (i.e. surcharge applied immediately after mixing) to 210 kPa at 28 days and about 520 kPa at 240 days. The slight decrease in void ratio with curing time should be noted. The curved part defining the yield stress was well defined for all tests and the normally consolidated part of the curve did not exhibit any brittle behaviour, within the range of pressures applied.

The $e$-$\log\sigma'_v$ curves, $e$ being the void ratio after 24 hours of applying a load increment, for stabilised peat are quiet flat at low stresses up to the yield stress and then deformations become large as the yield stress is exceeded. The slope of the post-yield portion of the $e$-$\log\sigma'_v$ curve is shown to remain constant with curing time. After yielding, the $e$-$\log\sigma'_v$ curves seem to lie within the same curve. The normally consolidated line (ncl) of the virgin peat is plotted in Figure 4.16. It can be seen that after yielding of the cement-stabilised peat, the compression index is still lower than that of the original peat. The unload-reload loop for the specimen cured at 28 days is shown in Figure 4.17. This Figure shows that the reload curve has the same slope as the unloading line.
The behaviour of the cement-stabilised peat in the oedometer test was investigated in both its intact and remoulded state. The remoulded specimen was remoulded by hand, nearly at the same water content as the intact specimen, until breaking all the soil structure and then left to soak in water for a period of 24 hours for saturation. Results obtained are shown in Figure 4.18.

Figure 4.17. Consolidation test. Cement-stabilised peat specimen cured at 28 days. Amount of binder 200 kg/m$^3$.

Figure 4.18. $e$-$\log \sigma'_v$ curves for both intact and remoulded cement-stabilised Ballydermot peat. Amount of binder 200 kg/m$^3$. 
Figure 4.18 shows that the normally consolidated lines of both intact and remoulded cement-stabilised peat samples are almost identical after yielding.

Figure 4.19 shows the compression curve under isotropic conditions in the triaxial cell. A back pressure of 300 kPa was applied for saturating the specimen. Several load increments ranging from 25 to 1100 kPa were applied. It can be clearly seen that the specimen yields at a stress of about 310 kPa.

![Graph showing isotropic consolidation test](image)

**Figure 4.19.** Isotropic consolidation test. Cement-stabilised peat cured at 90 days. Amount of binder 200 kg/m$^3$.

The variation of the coefficient of volume compressibility $m_v$, for the specimens cured at 28, 90 and 240 days, with vertical effective stress is shown in Figure 4.20. The compressibility of the specimens cured at 90 and 240 days was similar, however the specimen cured for 28 days was more compressible. The compressibility is shown to decrease up to the yield stress, then increases after yield and drops again as the vertical stress increases.

The coefficient of consolidation $c_v$ calculated for the different curing times is shown in Figure 4.21; $c_v$ was determined using the root time method. The values of $c_v$ at the start of compression were about 5-6 m$^2$/year but increased quickly as yield stress was approached reaching a maximum at yield and then decreased with increasing load.
The creep behaviour of the cement-stabilised peat was investigated in standard incremental oedometer tests. Long-term consolidation tests with time increments of one month were also performed. The coefficient of secondary compression $C_α (= \Delta e/\Delta \log t)$ was determined from the slope of the $e$-$\log t$ curves 4 - 24h after a load increment was applied (the end of the primary consolidation was assessed using the root time method). The variation of $C_α$ with $\sigma'$ (in a semi-log scale) is shown in Figure 4.22. This Figure shows that the
coefficient of secondary compression $C_a$ is initially low, starts increasing as the yield stress is approached and increases sharply after exceeding the yield stress.

![Graph showing $C_a$ versus effective stress for cement-stabilised Ballydermot peat. Amount of binder 200 kg/m$^3$.](image)

Figure 4.22. $C_a$ versus effective stress for cement-stabilised Ballydermot peat. Amount of binder 200 kg/m$^3$.

The value of the ratio $C_a/C_c$ obtained from consolidation tests carried out on cement-stabilised peat specimens cured at 28, 90 and 240 days was evaluated following the graphical method described in Mesri et al. (1987). From Figure 4.23, it can be seen that the best-fit line yields a value of $C_a/C_c = 0.054$ for the specimen cured at 28 days and $C_a/C_c = 0.036$ for specimens cured at 90 and 240 days.

![Graph showing $C_a$ versus $C_c$ for cement-stabilised Ballydermot peat. Amount of binder 200 kg/m$^3$.](image)

Figure 4.23. $C_a$ versus $C_c$ for cement-stabilised Ballydermot peat. Amount of binder 200 kg/m$^3$. 
The results of a long-term consolidation test with load time increments of 1 month are shown in Figure 4.24. Only pre-yield behaviour was examined. It can be clearly seen that settlements are not a linear function of log time. It should be noted that there was no control on the ambient temperature during the test.

After the last load increment of 400 kPa was applied, long-term unloading stages with one-month time increments followed. The specimen was found to swell to a final height within 24 hours of unloading and the height remained constant throughout the unload stage. No revert to creep, within the time increment applied, was observed in the unloading stages.

![Figure 4.24. Settlement versus log time. Cement-stabilised Ballydermot peat. 90 days curing time. Amount of binder 200 kg/m³.](image)

4.5.5 Permeability characteristics of cement-stabilised Ballydermot peat

The assessment of the permeability of the cement-stabilised Ballydermot peat was obtained using both constant head tests such as the Rowe cell and the triaxial cell. Results obtained from constant head triaxial and Rowe cell tests on cement-stabilised peat showed the permeability of this material to be of the order of $4$ to $5 \times 10^{-10}$ m/s. These results were found for specimens loaded with a surcharge of 18 kPa immediately after mixing. A constant head permeability test carried out on a specimen, which had not been subjected to any surcharge during curing yielded a permeability of $7 \times 10^{-8}$ m/s.
Figure 4.25 shows results of a Rowe cell test carried out on a cement-stabilised peat sample recovered from the stabilised peat column formed in large-scale testing chamber. The relationship between the void ratio $e$ and the coefficient of permeability $k$ (in a semi-log scale) is shown to be linear. However, there was a noticeable decrease in permeability measured after yielding. The $e\text{-log } \sigma'_v$ curve is plotted in the same graph and it can be seen that after yielding the permeability decreases by a factor of 50.

Figure 4.25. Void ratio versus permeability for sample recovered from the stabilised peat column formed in the chamber. Amount of binder 250 kg/m$^3$.

Indirect measurements of permeability of cement-stabilised peat was carried out in the constant rate of strain (CRS) apparatus. Figure 4.26 shows values of permeability obtained from a CRS test carried out on a cement-stabilised peat sample at a curing age of 240 days. The coefficient of permeability $k_v$ was determined using the expression reported by Lechowicz et al. (1996): $k_v = \frac{g \cdot \rho_w \cdot H \cdot \frac{de}{dt}}{2u_b}$, where $H$ is the height of the specimen, $\frac{de}{dt}$ is the rate of vertical strain and $u_b$ is the pore pressure at the undrained bottom. It can be clearly seen that the permeability of cement-stabilised peat falls quickly after yielding.
4.5.6 Shear strength parameters of cement-stabilised Ballydermot peat

4.5.6.1 Consolidated undrained triaxial compression tests (CUTC)

Isotropically consolidated undrained triaxial compression with local strain measurements were carried out on laboratory-mixed cement-stabilised Ballydermot peat samples at 90 days curing time and at confining pressures ranging from 50 to 350 kPa. The samples were sheared at a constant rate of strain of 4.5% per day. The deviatoric stress-strain curves of the CUTC tests carried out are plotted in Figure 4.27.

Figure 4.27 shows that cement-stabilised peat samples reach failure at low strains in undrained triaxial conditions. Peak strength was reached within 1.5 to 2% axial strain. The excess pore pressure response generated during shearing in the CUTC tests is plotted in Figure 4.28 in terms of the normalised pore pressure (excess pore pressure/effective consolidation pressure). High build-up of pore pressure can be observed for the specimen consolidated at 50 kPa. For this specimen, the maximum excess pore pressure was reached

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**Figure 4.26.** Coefficient of permeability $k$ versus void ratio $e$. Cement-stabilised peat sample cured at 240 days. Amount of binder 200 kg/m$^3$. 

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**Figure 4.27** shows a graph of $k$ (m/s) versus void ratio $e$ with a peak at yield.
at an axial strain of 0.25% and remained constant thereafter. For higher confining pressures, the excess pore pressure is shown to reach a maximum at the peak strength.

![Figure 4.27. Stress-strain curves. CUTC tests on cement-stabilised Ballydermot peat. Amount of binder 200 kg/m³.](image)

Figure 4.27. Stress-strain curves. CUTC tests on cement-stabilised Ballydermot peat. Amount of binder 200 kg/m³.

![Figure 4.28. Normalised pore pressure versus axial strain. CUTC tests on cement-stabilised Ballydermot peat. Amount of binder 200 kg/m³.](image)

Figure 4.28. Normalised pore pressure versus axial strain. CUTC tests on cement-stabilised Ballydermot peat. Amount of binder 200 kg/m³.

The effective stress paths for the CUTC tests carried out are shown in Figure 4.29. The tension cut-off line (i.e. $\sigma'_3 = 0$) is also plotted in the same graph. It is shown that the effective radial stress at failure approaches zero for the specimen consolidated at 50 kPa.
Figure 4.29. Effective stress paths for CUTC tests on cement-stabilised Ballydermot peat. Amount of binder 200 kg/m$^3$.

The effective stress paths obtained from CUTC tests are shown in Figure 4.30 for different contours of shear strains.

Figure 4.30. Effective stress paths for CUTC tests on cement-stabilised Ballydermot peat. Amount of binder 200 kg/m$^3$.

The Mohr circle for CUTC tests is shown in Figure 4.31. It can be seen that for confining stresses of 200 and 300 kPa there is no increase in the deviator stress. No effective shear strength parameters could be derived from the Mohr circles plotted in Figure 4.31.
Figure 4.31. Mohr circles for CUTC tests on cement-stabilised Ballydermot peat. Amount of binder 200 kg/m$^3$.

The secant undrained Young’s modulus $E_u$ normalised by the undrained shear strength $c_u$ is plotted in Figure 4.32. It can be shown that the stiffness of the cement-stabilised peat is strongly non-linear and is dependent upon both strain and confining stress level. The specimen consolidated at 50 kPa seems to yield the highest ratio $E_u/c_u$ among all the specimens tested. For axial strains higher than 1%, all the $E_u/c_u$ ratios for the different specimens seem to converge towards a same value of $\approx 100$.

Figure 4.32. $E_u/c_u$ versus axial strain. CUTC on cement-stabilised Ballydermot peat. Amount of binder 200 kg/m$^3$. 

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4.5.6.2 Consolidated drained triaxial compression tests (CDTC)

Isotropically consolidated drained triaxial compression tests were carried out on cement-stabilised Ballydermot peat at a curing age of 90 days. The stress-strain curves obtained from CDTC tests on cement-stabilised peat are plotted in Figure 4.33. The drained tests were not continued to large strains due to the set-up used during the tests (i.e. the GDS volume controller reached full saturation after an axial strain of about 6%). The specimen consolidated at 50 kPa reached its peak strength within the range of axial strain applied whereas for the rest of the specimens, the deviator stress was still increasing with increasing axial strain.

![Stress-strain curves](image)

**Figure 4.33. Stress-strain curves. CDTC tests on cement-stabilised Ballydermot peat. Amount of binder 200 kg/m³.**

The behaviour of the cement-stabilised peat was investigated at large strains using a different set-up. Two tests were carried out at confining pressures of 50 and 400 kPa. The confining pressures were selected to examine the post-peak behaviour at both low and high pressures. The stress-strain curves obtained for both tests are plotted in Figure 4.34. From this figure, it can be seen that for the specimen consolidated at 400 kPa, the first peak strength was reached at an axial strain of about 12.5%, the deviator stress remained constant and then experienced a sudden increase before reaching a second peak. For the specimen consolidated at 50 kPa, peak strength was reached at a quiet low strain and the stress-strain curve levelled off thereafter.
Figure 4.34. Stress-strain curve. CDTC tests on cement-stabilised Ballydermot peat. Amount of binder 200 kg/m$^3$.

The volumetric strain $\varepsilon_v (=\varepsilon_a + 2. \varepsilon_t)$ is plotted against the axial strain in Figure 4.35. The slope $\mu = (d\varepsilon_v/d\varepsilon_a) = 1.13$ measured for axial strains less than 1% was found to be independent of the confining pressure. Poisson’s ratio estimated from the relationship: $\nu = (\mu - 1)/2$, was about 0.07. The volumetric strain versus axial strain plots, for the specimens consolidated at 200 and 300 kPa, seem to fall within the one-dimensional consolidation line for the range of axial trains applied. Drained triaxial compression tests carried out on cement-stabilised peat specimens recovered from the calibration chamber showed the specimens tested remained essentially cylindrical within the range of axial strains applied (12 to 15%).

The variation of the secant shear and bulk modulus $G$ and $K$, normalised by the mean effective stress $p'$, with the shear strain are shown in Figures 4.36 and 4.37. The behaviour of the cement-stabilised peat is shown to be stiff and non-linear. The specimen consolidated at 50 kPa displayed the highest stiffness among the specimens tested.
Figure 4.35. Volumetric strain versus axial strain. CDTC on cement-stabilised Ballydermot peat.

Figure 4.36. Normalised secant shear modulus $G'_{sec}/p'$ versus shear strain. CDTC on cement-stabilised Ballydermot peat.
Figure 4.37. Normalised Bulk modulus $K_{sec}/p'$ versus volumetric strain. CDTC tests on cement-stabilised Ballydermot peat. Amount of binder 200 kg/m$^3$.

The overall effective stress paths obtained from both CUTC and CDTC tests are plotted in Figure 4.38.

Figure 4.38. Overall effective stress paths for CUTC and CDTC tests. Cement-stabilised Ballydermot peat.
4.5.6.3 Undrained direct simple shear tests (DSS)

Undrained direct simple shear tests were carried out on cement-stabilised Ballydermot peat specimens mixed in the laboratory and cured for a period of 90 days. The specimens tested were 50mm diameter and 19.05mm high. No back pressure could be applied in the direct simple shear therefore the specimens were not saturated during the test. The results obtained from the DSS tests are analysed in terms of total stresses.

The shear strength parameters (i.e. undrained cohesion and the angle of shearing resistance) were determined from the plot of the shear stress against the normal total stress. Figure 4.39 shows the undrained shear strength parameters of the cement-stabilised Ballydermot peat.

The undrained cohesion and the angle of shearing resistance were 26.3 kPa and 23.7° (assuming $\phi = \tan (\frac{t_{\text{max}}}{\sigma_n})$).

![Figure 4.39. Shear stress versus normal stress. DSS tests on cement-stabilised Ballydermot peat. Amout of binder 200 kg/m³.](image)

4.6 SUMMARY

This chapter details the procedures used for the preparation and testing of the laboratory-mixed stabilised peat specimens. Details of the tests carried out are given and the results of these tests presented.
Five different binders were used: cement, pfa, lime, blast furnace slag and gypsum, to investigate potential binders for the stabilisation of peat. In total seven different mixtures were designed and mixed with Rahennmore peat, only two mixtures were mixed with Ballydermot peat, at an amount of binder of 150, 200 and 250 kg/m$^3$. The strength achieved for each mixture was assessed by a means of unconfined compression tests. Cement and blast furnace slag binders performed best for Raheenmore peat whereas for Ballydermot peat the blast furnace slag binder performed poorly. The highest strength of the stabilised Ballydermot peat was obtained with cement as binder. For the CPL mixtures, the strength achieved decreased as the pfa content increased in the mixture. A relationship $E_{50} \approx 100 c_u$ was obtained using external strain measurement, however a secant stiffness modulus $E_{50}$, three-fold higher was measured using local strain measurements. The failure strain measured from the unconfined compression tests generally decreased as the strength achieved increased. Very brittle behaviour was observed for specimens that achieved high strengths.

The electron scanning microscope pictures of the microstructure of the cement-stabilised peat at 1 day curing time showed the cement particles to start hydrating. Examination of the microstructure of the Raheenmore peat stabilised with blast furnace slag mixture at 1 day curing time showed formation of ettringite rods-like shaped growing in the peat voids. The microstructure of the cement-stabilised peat at 28 days showed a texture covered with hexagonal platelets with average sizes over 10 $\mu$m.

Consolidation tests carried out on cement-stabilised Ballydermot peat at different curing times showed a stiff behaviour pre-yield. The yield stress increased with curing time, from 210 kPa at 28 days to about 510 kPa at 240 days curing time. The normally consolidated part of the $e$-$\log \sigma'$, did not exhibit any collapse type feature of cemented soils. Values of the coefficient of consolidation $c_v$ were about 5-6 m$^2$/year but increased quickly as yield stress approached and then decreased with increasing load. The coefficient of secondary consolidation $C_a$ values were initially low, started increasing as yield stress was approached and then increased sharply after yielding. A ratio $C_a/C_c = 0.036$ was obtained for specimens cured at 90 and 240 days.

The permeability of the cement-stabilised Ballydermot peat was of the order of $4$ to $5.10^{-10}$ m/s for the specimens preloaded immediately after mixing. For the specimens...
not preloaded, the permeability was of the same order as that of the original peat. The relationship between the void ratio and the coefficient of permeability $k$ (in a semi-log scale) was linear.

The shear strength behaviour of the cement-stabilised Ballydermot peat was investigated at a curing age of 90 days. Peak strength was reached within 1.5 to 2% axial strain in CUTC tests. High build-up of pore pressure was observed at low confining pressures. The lateral effective stress at low confining pressures approached zero at failure. The small strain stiffness was found to be strongly non-linear and was dependent on both strain and stress level. The post-peak behaviour in CDTC tests was different depending on the confining pressure applied. For low confining pressures, the stress-strain curve seemed to level off post-peak whereas at high confining pressures, the stress-strain curve reached a second peak strength post-peak. The deformation in CDTC tests was mainly accompanied by a decrease in height, the cement-stabilised peat is characterised by a low Poisson's ratio value.
CHAPTER 5

DESIGN AND DEVELOPMENT OF THE LARGE-SCALE TESTING CHAMBER TEST
5. DESIGN AND DEVELOPMENT OF THE LARGE-SCALE TESTING CHAMBER TEST

5.1 INTRODUCTION

This chapter gives details of the preparation, construction and testing procedures adopted for the testing of a stabilised soil structure (i.e. stabilised peat column and stabilised peat surface layer mass) in a large-scale testing chamber. Details of the instrumentation used for the monitoring of the test are presented as well as results of a parametric finite element study of the chamber test.

5.2 CONFIGURATION OF THE LARGE-SCALE TESTING CHAMBER TEST

5.2.1 Details of the large-scale testing chamber

The Trinity College large-scale testing chamber used was originally designed by Gavin (1998) for an experimental investigation of open and closed ended piles in sand. The testing chamber consists of a precast reinforced concrete pipe of 2.3m height, 1.68m diameter and wall thickness of 150mm (total internal volume of pipe is $5\text{m}^3$). The pipe was placed in a 2.4m square and 5m high structural steel frame capable of supplying a reaction force of 300kN. Figure 5.1a shows a picture of the testing chamber.

The axial load was applied by means of airbags acting on the top of the chamber to allow uniform pressure distribution. As there was a difference of 0.3m between the surface of the stabilised mass and the reaction frame, a steel plate of 10mm thickness was placed on the top of the airbag and the reaction was transmitted to the reaction frame through timber spacers as shown in Figure 5.1b. The structural design of all the components of the chamber are described in detail by Gavin (1998). That author designed the structural components for a maximum working pressure of 300 kPa, which is well beyond the maximum pressure intended to be used in this project (i.e. 60 kPa).
5.2.2 Peat block sampling and filling of the large-scale testing chamber

5.2.2.1 Peat block sampling

Undisturbed block samples were recovered from Ballydermot bog. The samples were taken from a vertical face 2.5m high, as described in § 3.2.3. The samples were cubic in shape, $1 \times 1 \times 1$ m in dimension, and were obtained using a steel box which consisted of three faces having sharp edges. The steel box was attached to a Hy-Mac lifting arm specially designed by Bord Na Mona (semi-state body responsible for mining peat in Ireland) for block sampling- Figure 5.2 (a).

The top 0.5 m soil layer was cleared off before sampling. Peat blocks were cut at the back and the sides before pushing the steel box into the vertical face. The peat blocks were wrapped in polythene to keep the moisture within the samples and were put into wooden boxes, assembled on site, tight enough to prevent the block samples from moving during transportation to the laboratory. Four peat blocks were recovered from a depth of 0.5 m - 1.5 m and four others from a depth of 1.5 m - 2.5 m. Photographs of the sampling process are shown in Figure 5.2.
Figure 5.2. Peat block sampling process at Ballydermot bog.

5.2.2.2 Filling of the large-scale testing chamber

Four block samples recovered from the same depth were positioned on top of a steel plate and trimmed to the same diameter as the steel plate. The trimmed block samples were then surrounded with wallboard to keep the shape while lifting. This operation was repeated for the remaining blocks, keeping the same block position as in the field. Once the two layers were formed, the trimmed blocks were lifted using a steel plate, placed at the bottom, attached to the gantry crane by a means of steel chains which were assembled to a cruciform to prevent the steel chains from cutting through the peat blocks—see Figure 5.3. As the peat blocks were lowered down into the chamber, the wallboard was stripped off. To keep the moisture in the peat, water obtained from a drainage ditch in the same sampling area was added to the peat and the water table was consequently kept at the surface of the material in the chamber throughout the test.
Cruciform preventing the steel chains from cutting through the peat.

Figure 5.3. Process of filling the large-scale testing chamber. (a) First layer formed and lined with cardboard, (b) wooden platform assembled for forming the second layer, (c) both layers lifted and lowered down into the chamber.

5.2.3 Instrumentation of the large-scale testing chamber

The large-scale testing chamber was instrumented to allow monitoring of the following:

(i) pore pressure,
(ii) total earth pressure,
(iii) settlements at different depths,
(iv) temperature.

Because of the high compressibility and low shear strength of the peat, the instrumentation used was carefully selected in order to prevent or minimise any interaction with the peat.
Some of the instrumentation proposed for installation in the stabilised material could be affected by high temperatures. Therefore, a preliminary test was carried out in order to estimate the expected temperature rise by enclosing a sample of cement-stabilised peat in a 0.5m cube insulated steel box. Thermocouples for measuring temperature were embedded in the stabilised peat at different depths and the results obtained showed a maximum temperature of 13.5°C. See Figure 5.4.

![Figure 5.4. Increase in temperature with time for peat-cement mixture.](image)

### 5.2.3.1 Pore pressure transducers

The pore pressure transducers used were miniature Druck PDCR 81 pore pressure transducers. These were 11.9mm long and 6.4mm diameter. The operating range of the transducers was between 0 to 350 kPa. The hysteresis was of ± 0.2% B.S.L, thermal sensitivity shift of ± 0.2%/°C and a sensitivity of 0.055mV/V/kPa. The operating temperature range was -20°C to +120°C. A detail of the pore pressure transducers used is shown in Figure 5.5. The pore pressure transducers were excited by a 10V power supply. Even though, the pore pressure transducers were supplied with calibration certificate by the manufacturer, these were checked again by immersing the transducers in a water cylinder at various known depths and recording the output voltage. The calibration factor was found to be similar to that supplied by the manufacturer. The pore pressure transducers were pushed into the soil and sealed with a mixture of bentonite-cement.
5.2.3.2 Soil pressure cells

The soil pressure cells used were the same as those previously used by Gavin (1998). Altogether, three pressure cells were used during the test. The soil pressure cells had an operating range of 0 to 500 kPa and were oil filled. The pressure cells were calibrated before being placed in the testing chamber by applying various known masses and recording the change in output.

5.2.3.3 Settlement plates

The settlement plates used were designed in-house. These were polycarbonate rings of 51 mm internal and 100.6 mm external diameter. Circular magnets of standard ferrite constitution were attached to the rings. The settlement plates were centred on a 38 mm diameter collapsible hose. The location of the settlement plates was measured by a means of a sensor, which was designed using a Hall effect transistor with characteristics sensitive to the magnet situated on the polycarbonate plates. The sensor was attached and sealed carefully, to prevent any contact with pore water, to a metal rod. When lowered into the collapsible hose, the output voltage displayed by the sensor increased as it approached the magnet attached to the settlement plate reaching a peak output at the level of the magnet. A simulation of the conditions prevailing in the testing chamber was performed by placing a settlement plate at a known depth in a large container filled with peat. The sensor was then
lowered down into the collapsible hose and the output voltage showed that the location of
the plate can be determined within ± 1mm as shown in Figure 5.6.

![Graph showing output voltage versus distance from target.](image)

Figure 5.6. Output voltage versus distance from target for settlement sensor.

A laboratory test showed that the compressibility of the collapsible hose under axial load
was considerably less than that of an equivalent area of peat, hence its introduction does
not increase the stiffness of the material in the chamber. A detail of the settlement plates
used is shown in Figure 5.7.

![Settlement plate image with ruler.](image)

Figure 5.7. Settlement plate used for settlement measurements.

### 5.2.3.4 Temperature thermocouples

The temperature developing in the testing chamber was measured by a means of
thermocouples made of two wires (nickel and cadmium). Both wires are exposed to
temperature and the resistance of the thermocouple varies as the temperature changes (i.e.
resistance of the thermocouple decreases as the temperature increases).
5.2.3.5 Data acquisition

All the instruments placed in the testing chamber were monitored continuously during the test. All the wires were carefully protected in small plastic pipes to avoid any damage during loading. The instruments were connected to an RS recorder system unit, which recorded the output voltage except for the thermocouples, which displayed the current temperature. The data obtained were stored in the computer hard disk and backed up in a floppy disk in a regular basis. The RS recorder system could be programmed to scan readings at regular frequency.

5.3 FORMATION AND DESIGN OF THE STABILISED COLUMN AND STABILISED MASS

5.3.1 Stabilisation design

The design concept for the testing chamber was to have a central stabilised column through the entire height of the peat and also have a stabilised surface layer to distribute the load to the column.

5.3.2 Mixing procedure for the stabilised mass and column

The “dough-mixer” described in § 4.1 was only suitable for mixing small quantities of peat, therefore it was necessary to use a larger laboratory mixer for forming the stabilised peat to be used in the testing chamber.

The peat was mixed in a large-pan laboratory mixer of 0.08m³ capacity -see Figure 5.8. Because of the large volume of peat to be mixed, it was necessary to assess the strength achieved by cement-stabilised peat samples mixed in the pan mixer and compare the strength figures obtained with those obtained when using the dough-mixer, which was the reference mixing procedure used.

The stabilised peat was formed in the pan-mixer as described in § 4.2 except that two mixing times of 5 and 10 minutes were selected for mixing the peat with the cement. The
unconfined compression test results obtained for specimens mixed in the pan-mixer showed more scatter than those mixed in the dough-mixer—see Figure 5.9.

Figure 5.8. Laboratory pan-mixer used for formation of the stabilised mass and column.

The strength achieved for specimens mixed in the pan-mixer for 10 minutes was about 77% that achieved for specimens mixed in the dough-mixer, whereas for specimens mixed for 5 minutes the strength achieved was about 65% of that achieved when using the dough-mixer. Except for the increase in scatter observed, the strength achieved for specimens mixed in the pan-mixer was comparable to that achieved when using the dough-mixer. It was therefore decided to select a mixing time of 10 minutes when using the pan-mixer and an amount of cement of 200 kg/m$^3$ for the stabilised mass and 250 kg/m$^3$ for the stabilised column.

![Graph showing results of unconfined compression tests](image)

Figure 5.9. Results of unconfined compression tests on specimens mixed in the dough-mixer and the pan-mixer. 28 days curing time. Amount of binder 250 kg/m$^3$. 

D.M: dough-mixer
P.M. 5: pan-mixer, mixing for 5 minutes.
P.M. 10: pan-mixer, mixing for 10 minutes.
5.3.3 Finite element parametric study of the large-scale testing chamber test

5.3.3.3 Modelling of the large-scale testing chamber

The large-scale testing chamber test was modelled using the Plaxis finite element model version 6.0. A parametric finite element study was carried out in order to investigate the effect of the following parameters: column diameter (the height of the stabilised mass being fixed to 0.5m), stiffness of the stabilised material and the height of the stabilised mass. Because of the symmetry of the problem, only half of the testing chamber required modelling. Plaxis calculations were made with an axisymmetric model using 15-noded triangle elements. Figure 5.10 shows the finite element mesh used for the calculations. The left boundary represents the axis of symmetry of the model.

![Finite element mesh](image)

Figure 5.10. Large-scale testing chamber mesh used with boundary conditions. (to scale)

At the vertical mesh boundaries horizontal movement is prevented. The bottom boundary is fixed in both horizontal and vertical direction and the upper horizontal boundary is free to move in the vertical direction. The material in the chamber is free to drain at both ends and radially; however, because of the axisymmetric conditions, no drainage is allowed at the vertical left boundary (i.e. axis of symmetry).

The design parameters used for the stabilised peat were those determined from the results of the unconfined compression tests carried out at a curing age of 90 days. This curing time was selected because of the increase in strength with time observed for the cement-
stabilised peat. Due to the lower strength yielded in the laboratory pan-mixer and also the possible delay in applying the initial surcharge, which is more likely to occur considering the scale of the project, it was decided to select a design target shear strength of 150 kPa for the stabilised column and 100 kPa for the stabilised mass.

For the initial analysis, a linear-elastic perfectly plastic model with Mohr-Coulomb failure criterion was used for all the materials. The calculation sequence was as follows:

(i) Initial stresses generation using the $k_0$ procedure (a lateral earth pressure coefficient of 0.35 was assumed for peat and 0.5 for stabilised peat),

(ii) Load increments of 20, 40 and 60 kPa (plastic calculation), with each stage followed by a consolidation analysis.

The drained soil parameters used for the peat and the stabilised peat are given in Table 5.1.

Table 5.1 Preliminary parameters used for Plaxis calculations.

<table>
<thead>
<tr>
<th></th>
<th>$c'$ (kPa)</th>
<th>$\phi'$ (deg)</th>
<th>$E'$ (kPa)</th>
<th>$v'$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peat</td>
<td>1</td>
<td>30</td>
<td>333</td>
<td>0.11</td>
</tr>
<tr>
<td>Stabilised mass</td>
<td>20</td>
<td>30</td>
<td>10000</td>
<td>0.25</td>
</tr>
<tr>
<td>Stabilised column</td>
<td>30</td>
<td>30</td>
<td>15000</td>
<td>0.25</td>
</tr>
</tbody>
</table>

Notation:

$E'$ = drained secant stiffness modulus

$c'$ = effective cohesion

$\phi'$ = effective angle of shearing resistance

$v'$ = drained Poisson’s ratio

The following correlations were used:

$E' = 100.c_u$ and $c' = 0.2.c_u$. Peat materials do not have cohesion, however the Plaxis finite element model requires the input of a minimal value of 1 for the cohesion.

(a) Varying the column diameter

Column diameters of 30, 40, 50, 60 and 70 cm were selected using the same input parameters as in Table 5.1. Figure 5.11 shows the computed settlement ratios (ratio of the settlement of the treated soil to the that of the untreated soil) for three load increments applied. The column spacing is defined as the ratio (chamber diameter/column diameter).
Figure 5.11. Settlement ratio versus column spacing. Parametric finite element study.

As it can be seen from Figure 5.11, the settlement ratio decreases with decreasing column spacing. This decrease becomes greater for column spacing of less than four times the diameter of the column. However, the effect of column spacing is somewhat overshadowed by the interaction soil/concrete wall.

Figure 5.12 shows the distribution of average vertical stresses (i.e. the average stress was calculated by dividing the force acting on the column by the area of the column) in the column and the stabilised mass for three column diameters of 30, 50 and 60cm. For an applied load of 20 kPa, the stress increases in the column as the column diameter decreases; however, for a load of 60 kPa, the stress distribution in the column does not seem to be affected by the column diameter. At this stage the column behaviour is no longer elastic and the range of stresses carried by the column is depending upon the strength characteristics of the stabilised material.

Figure 5.12. Variation of stress with depth along the column and the stabilised mass. (a) Applied load of 20 kPa, (b) applied load of 60 kPa.
(b) Varying the stiffness of the stabilised material

Three values of the stiffness modulus $E$ were selected for the stabilised material: $E$, $0.7E$ and $0.5E$. The column diameter was fixed at 60 cm. Figure 5.13 shows the effect of varying the stiffness of the stabilised material on the computed final settlements.

![Stress Settlement Graph](image)

Figure 5.13. Effect of varying stiffness of the stabilised material on the maximum calculated final settlements.

Figure 5.13 shows the difference in settlements increased with increasing applied load. For lower stiffness values, a larger part of the load applied was taken by the surrounding soil thus, larger settlements occurred.

Figure 5.14 shows the stress distribution with depth for different stiffness values of the stabilised material. For an applied load of 20 kPa, the stress distribution is almost identical and is not affected by the stiffness of the stabilised material. As the applied load increases, the stress distribution differs. The stiffer the material, the higher is the stress carried by the column.

From both results obtained when varying the column diameter and the stiffness of the stabilised material, the distribution of stress, in the elastic range, with depth in the column is more sensitive to a variation in column diameter than to a variation in the stiffness of the stabilised material.
Figure 5.14. Stress distribution with depth in the column for different stiffness values of the stabilised material. (a) Applied load of 20 kPa, (b) applied load of 40 kPa.

(c) Varying the height of the stabilised mass (i.e. stabilised peat surface layer)

A comparison was made between the case where the peat is reinforced with a stabilised column only and a case where the peat is reinforced with both a stabilised column and a stabilised mass. Two layer heights of 0.25 and 0.50m were selected. Figure 5.15 shows the computed final settlements for a variation of the stabilised mass height.

Figure 5.15. Computed maximal settlements, at the surface, at different load increments for various stabilised mass heights.

From Figure 5.15, it can be seen that there is a slight difference in the maximum computed final settlements for a stabilised mass height of 0.25 and 0.5m. However, there is a drastic increase in computed settlements when the peat is only reinforced by a stabilised column.
Examination of the distribution of stress concentration, defined as the ratio of computed stress in the column to the computed stress in the peat at a radial distance of 0.45m from the centreline, along the depth of the system stabilised soil/original soil shows clearly that arching occurred between the column and the peat when a stabilised mass was formed on the top layer of the peat- see Figure 5.16.

![Stress concentration ratio vs depth](image)

Figure 5.16. Variation of the stress concentration ratio with depth at a load of 20 kPa.

The parametric finite element study of the testing chamber showed that:

(i) Larger decrease in the settlement occurs at a column spacing of less than four times the column diameter. The final computed settlements are affected by a variation of the stiffness of the stabilised material. The difference in settlement becomes higher as the load applied increases,

(ii) The stabilised mass formed at the upper layer of the soil plays a great role in allowing arching of the soil between the columns. Reinforcement of peat soils with stabilised columns only is not efficient.

Following these conclusions it was decided to form a stabilised column of 60cm diameter over a height of 1.5m and a stabilised mass on the top 0.5m layer over the whole circumference of the testing chamber.
3.4 Formation of the stabilised column and stabilised mass

The stabilised column was formed by pushing in a 600mm-diameter pipe (see Figure 5.17) into the peat within the chamber. The peat extracted from the pipe was mixed with cement in a large laboratory pan-mixer, in accordance with the procedure described in § 5.3.2, and then replaced in the chamber. The peat used for forming the stabilised mass was extracted from the chamber, mixed with cement in the same way as for the stabilised column and then replaced in the chamber. Due to unforeseen technical problems, the stabilised mass was formed in two layers interrupted in time. The instrumentation used was placed at the same time as the stabilised column and mass were being formed in the testing chamber. The stabilised material was compacted in several layers as it was replaced in the chamber.

Two months after formation of the stabilised peat in the chamber, two surface load increments of 20 kPa and 40 kPa were applied. The chamber was thereafter unloaded successively to 30 and 20 kPa and reloaded to 30, 40 and 50 kPa. The axial stress was applied by a means of airbags as explained in § 5.2.1. The air pressure was generated by a portable compressor and controlled by a means of a pressure gauge having a sensitivity of ± 1 kPa. See Figure 5.18 for details of the instrumentation layout used in the testing chamber.

Figure 5.17. Pipe used for extracting peat used for formation of the stabilised column.
Figure 5.18. Instrumentation of the testing chamber.
5.4 CPTU TEST FOR IN-SITU TESTING

5.4.1 CPTU test set-up

The same assembly system as that used by Gavin (1998) for his model pile experiment was used to push a 35.7 mm diameter cone into the chamber. The assembly system consisted of two cross beams bolted to the four uprights and a H reaction frame bolted in turn to the two cross beams. The H reaction frame consisted of two I-beams connected by a double channel section. Cone rods could therefore be passed through the space created by the double channel- see Figure 5.20a.
A 250 kN capacity hydraulic jack was bolted to the H reaction frame and a load cell was attached to it and secured by an adjustable screw nut—see Figure 5.20b. The hydraulic jack was set to a maximum stroke of 150 mm. The cone head penetration was measured by means of an encoder firmly secured to the H reaction frame and tied to the driving clamp. As the driving clamp was pushed down, this movement induced the encoder to rotate and the number of revolutions observed were converted to axial displacement by a calibration factor determined previously. The hydraulic jack was lowered at a constant rate of displacement of 12 mm/min using a driving system based on oil pressure. After reaching the maximum stroke (i.e. 150 mm), the driving clamp was loosened off, the ram of the jack was retracted and the driving clamp was then resecured in its new position further back up to allow for another penetration of 150 mm.

The cone resistance, pore pressure and sleeve friction were directly read, signals transmitted through a cable pre-threaded down the push rods, using a datalogger. The cone head displacement and the load cell output were read using the 5000 datascan system. One feature of the cone truck datalogger is that it scans the cone resistance, sleeve friction and pore pressure at constant depth intervals. As the cone was pushed into the soil with penetration increments of 150 mm, knowing the elapsed time between the start and the finishing of the penetration, the position of the cone could be determined for each scanning time and the cone resistance profile with depth was obtained.

Cone penetration tests were carried out at two different locations. One at the centre of the column and the other one mid-way between the centreline and the concrete wall.
Figure 5.20. (a) Assembly system used to perform the CPT test.

Figure 5.20. (b) Assembly system used to perform the CPT test.
5.5. SUMMARY

This chapter reviewed the design and development of a large-scale laboratory test, which was performed in an instrumented testing chamber of 1.68m diameter and 2.3m high.

Undisturbed peat block samples, 1x1x1m in volume, were recovered from Ballydermot bog. The peat blocks were transported to the laboratory, trimmed to fit into the chamber and eventually placed to a height of 2m. To simulate construction methods in the field for stabilisation of organic soils, it was decided to form a stabilised column and a stabilised surface layer (i.e. stabilised mass) in the peat. A parametric finite element study, using Plaxis finite element model, was performed in order to design the dimensions of the stabilised column and the stabilised mass.

The peat used for forming the stabilised material was extracted from the chamber, mixed with the cement in a large pan-mixer and replaced in the chamber. A stabilised column of 600mm diameter and 1.5m high and a stabilised mass of 0.5m high were formed. The amount of binder used was 200 kg/m³ for the stabilised mass and 250 kg/m³ for the stabilised column. The large-scale testing chamber was instrumented in order to monitor settlements, pore pressure, earth pressure and temperature variation throughout the test.

A CPTU test set-up was designed in order to assess the variation of strength in the stabilised mass and column with depth. Details of the particular set-up used were presented.
CHAPTER 6

EXPERIMENTAL RESULTS OF THE LARGE-SCALE TESTING CHAMBER TEST
6. EXPERIMENTAL RESULTS OF THE LARGE TESTING CHAMBER TEST

6.1 INTRODUCTION

This chapter presents results of the large testing chamber test described in chapter 5. The material in the chamber was subjected to loading and unloading sequences. After the completion of the tests, in-situ measurement of the strength of the stabilised material was performed. Samples of the stabilised material were also recovered for laboratory testing. The parameters obtained from these tests were then used for a back analysis of the testing chamber test using a finite element model.

6.2 LARGE TESTING CHAMBER TEST

6.2.1 Loading sequence

The loading sequence adopted for the testing chamber test is shown in Table 6.1.

Table 6.1. Loading sequence of the testing chamber test.

<table>
<thead>
<tr>
<th>Date</th>
<th>Load, (kPa)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>26/05/99</td>
<td>/</td>
<td>Formation of the stabilised column and the first layer of the stabilised mass.</td>
</tr>
<tr>
<td>11/06/99</td>
<td>/</td>
<td>Formation of the second stabilised mass layer.</td>
</tr>
<tr>
<td>11/08/99</td>
<td>20</td>
<td>Loading stage</td>
</tr>
<tr>
<td>30/09/99</td>
<td>40</td>
<td>Loading stage</td>
</tr>
<tr>
<td>25/11/99</td>
<td>30</td>
<td>Unloading stage</td>
</tr>
<tr>
<td>10/01/00</td>
<td>20</td>
<td>Unloading stage</td>
</tr>
<tr>
<td>17/01/00</td>
<td>30</td>
<td>Reloading stage</td>
</tr>
<tr>
<td>24/01/00</td>
<td>40</td>
<td>Reloading stage</td>
</tr>
<tr>
<td>14/02/00</td>
<td>50</td>
<td>Loading stage</td>
</tr>
<tr>
<td>06/04/00</td>
<td>50</td>
<td>Test completed</td>
</tr>
</tbody>
</table>
6.2.2 Monitoring of the test and performance of the instrumentation used

The testing chamber was instrumented in order to monitor the following items:

(i) Surface deformation versus time in the stabilised mass,
(ii) Deformation versus time in the stabilised column and in the peat layer at different depths,
(iii) Pore water pressure in the peat, stabilised mass and stabilised column,
(iv) Total earth pressure on top and bottom of the stabilised column and top of the peat section,
(v) Temperature inside and outside the stabilised material.

The system used for measuring settlements performed quite well throughout the test except in cases where anomalies were observed at some settlement plates. Heave was recorded by some settlement plates (S3, S6 and S1) during the early loading stages and as this does not represent realistically the settlement profile of the soil in the chamber. It was necessary to discount these when interpreting the results.

The earth pressure cells placed at the top and bottom of the stabilised column performed poorly as they did not record any change in pressure throughout the test. Therefore, the records of these have been excluded. Only the total earth pressure at the top of the unstabilised peat section was recorded.

The pore pressure gauges recorded changes in pore pressure during loading, although the changes were somewhat erratic. Thermocouples performed very well during the entire duration of the test.

6.2.3 Settlements

The settlements measured at different locations in the testing chamber are presented in this section. The results are presented in terms of settlement versus time for the loading and unloading-reloading stages- see Figures 6.1a to g. (z represents the depth below the surface level and r the horizontal distance from the chamber centreline).
Figure 6.1a. Settlements versus time for settlement plate S1 (z = 1.00m, r = 0.60m).

Figure 6.1b. Settlements versus time for settlement plate S2 (z = 0.50m, r = 0.60m).

Figure 6.1c. Settlements versus time for settlement plate S3 (z = 1.50m, r = 0.00m).
Figure 6.1d. Settlements versus time for settlement plate S4 (z = 1.00m, r = 0.00m).

Figure 6.1e. Settlements versus time for settlement plate S5 (z = 0.50m, r = 0.00m).

Figure 6.1f. Settlements versus time for settlement plate S6 (z = 0.00m, r = 0.00m).
Figure 6.1g. Settlements versus time for settlement plate S7 (z = 0.00m, r = 0.60m).

The measured final settlements at the different loading stages are shown below in Table 6.2.

Table 6.2. Final measured settlements at different loading stages.

<table>
<thead>
<tr>
<th>Settlement plate</th>
<th>Location</th>
<th>Settlement, (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>20 kPa</td>
</tr>
<tr>
<td>S1 (z = 1.00m)</td>
<td>peat</td>
<td>40</td>
</tr>
<tr>
<td>S2 (z = 0.50m)</td>
<td>peat</td>
<td>20</td>
</tr>
<tr>
<td>S3 (z = 1.50m)</td>
<td>column</td>
<td>-3</td>
</tr>
<tr>
<td>S4 (z = 1.00m)</td>
<td>column</td>
<td>37</td>
</tr>
<tr>
<td>S5 (z = 0.50m)</td>
<td>column</td>
<td>34</td>
</tr>
<tr>
<td>S6 (z = 0.00m)</td>
<td>S.mass</td>
<td>-8</td>
</tr>
<tr>
<td>S7 (z = 0.00m)</td>
<td>S.mass</td>
<td>42</td>
</tr>
</tbody>
</table>

The measured settlements with time are plotted in Figure 6.2a and b for the settlement plates S1, S2, S4 and S5. The figure shows the variation in settlement at two different horizontal sections; one at a depth of 0.5m from the surface and the other at 0.5m below this. Only loading stages of 20 and 40 kPa are shown.
Figure 6.2a. Settlement versus time for settlement plates S2 and S5 (z = 0.50m).

Figure 6.2b. Settlement versus time for settlement plates S1 and S4 (z = 1.00m).

A plot of the measured average strain (i.e. change in distance between the settlement plates over the original distance) distribution in the chamber is shown in Figures 6.3a and b for the two applied loads of 40 and 50 kPa. The settlement readings at S6 (z = 0.00m, r = 0.00m) were not consistent with the observed behaviour at the surface of the stabilised mass, which settled essentially uniformly. This discrepancy possibly arose due to deflection of the settlement ring. Therefore, the settlement at S6 was taken equal to that of S7 in subsequent analyses.
From the measured settlements data obtained, it can be observed that:

(i) For the first load increment of 20 kPa, the measured settlement at S1 was nearly double that measured at S2 suggesting that the peat layer has expanded and this is not realistic. Settlements measured at S3 and S6 showed the soil to heave and this is also inconsistent with the overall behaviour observed in the chamber. After applying the second load of 40 kPa, S1 heaved before it started settling again whereas, both settlement plates S3 and S6 settled with the soil- see Figures 6.1a, b, c and f.
(ii) A differential settlement of 54mm was measured between S6 and S7 for the last applied load of 50 kPa. Direct measurement of the location of both plates, relative to a fixed reference beam laid on the top of the chamber, after removal of the loading set-up, showed a difference of only 13mm between the two settlement plates. This could be accounted to the non-uniformity of the surface after the formation of the stabilised mass,

(iii) The upper third layer of the stabilised column nearly compresses by the same amount for all the load increments applied. As shown in Figure 6.3a and b, little compression takes place in the upper part of the column, whereas the remaining part experiences much higher compression. The average measured compression of the upper layer of the column between settlement plates S4 and S5 was 0.4% and 0% for applied loads of 40 and 50 kPa. The measured average compression for the layer between S4 and the bottom of the testing chamber (fixed bottom) was of the order of 7.5% and 10.5% for the same load increments. The average measured compression in the stabilised mass was 1.6 and 1.4% for applied loads of 40 and 50 kPa,

(iv) In the peat layer, the compression was higher in the upper peat layer than in the bottom layer. The compression of the upper layer measured between settlement plates S1 and S2 was of the order of 7.6% and 8.8% for applied loads of 40 and 50 kPa, whereas the average measured compression in the lower peat layer measured between the settlement plate S1 and the bottom of the chamber was 3.3% and 4.10% for the same applied loads,

(v) The top of the column section was found to settle more than the adjacent peat section at the same level. The difference was 5mm for an applied load of 40 kPa and 20mm for a load of 50 kPa. For the horizontal section located at 1m below the surface level, the settlement measured in the column was higher than that measured in the peat layer. The difference was 42 and 64mm for applied loads of 40 and 50 kPa,

Figure 6.4 shows the level of the settlement plates during the load/unload and reload stages carried out throughout the test.
Figure 6.4. Level of the settlement plates at different load stages.

It can be seen that the slope of the unload/reload parts is the same. During the reloading stage, the final measured settlements were slightly smaller than those measured during the unloading stage for a load of 30 kPa. However, for the applied load of 40 kPa, the measured settlements were almost identical to those measured during the loading stage except for settlement plate S2 where a difference of 3mm was observed.

6.2.4 Pore pressures

The pore pressure readings taken throughout the test are presented in this section. The pore pressure was measured immediately after application of the first load increment. Figures 6.5a, b and c show the variation of excess pore pressure with time measured at the different pore pressure gauges locations. Figures 6.6a and b show the variation of excess pore pressure with time for the same horizontal section at \( z = 1.00 \text{m} \) and \( 1.50 \text{m} \).
Figure 6.5a. Excess pore pressures versus time. P1 (z = 1.50m, r = 0.60m), P2 (z = 1.00m, r = 0.60m).
Figure 6.5b. Excess pore pressure versus time. P3 \((z = 1.50\text{m}, r = 0.00\text{m})\), P4 \((z = 1.00\text{m}, r = 0.00\text{m})\).
Figure 6.5c. Excess pore pressure versus time. P5 (\(z = 0.50\)m, \(r = 0.00\)m), P6 (\(z = 0.00\)m, 
\(r = 0.00\)m)
Figure 6.6a. Excess pore pressure changes versus time at $z = 1.50$m.

Figure 6.6b. Excess pore pressure changes versus time at $z = 1.00$m.

From the results obtained it can be observed that:

(i) The excess pore pressure measured during the first applied load of 20 kPa increased quickly, reaching a maximum value within 24 hours of applying the load, then started dissipating over a period of 10 days. Thereafter, all excess pore pressures measured showed an increase even though the load applied remained constant,

(ii) For the applied load of 40 kPa, no noticeable dissipation of pore pressures was observed throughout the duration of the load increment. Excess pore pressures measured in the stabilised mass showed an increase with time (more pronounced for P6),
(iii) Excess pore pressure measured in the column showed that more pore pressure develops in the top layer. For a load of 40 kPa, the excess pore pressure measured at a depth of 0.5m in the column was of the order of 60 kPa and about 20 kPa at a depth of 1m in the column,

(iv) Excess pore pressures measured at a depth of 1.5m below the surface level were higher in the peat than in the column whereas at a depth of 1m below the surface level, excess pore pressures were higher in the column than in the peat,

(v) Inconsistent excess pore pressure measurements were observed in the stabilised mass. Measurements obtained from pore pressure gauges P5 and P6 showed quite different patterns,

6.2.4.1 Rate of consolidation

The observed pattern of excess pore pressure changes with time for the different loading stages did not enable the determination of the rate of consolidation in both the stabilised material and the peat. Therefore, it was decided to use a graphical method described by Asaoka (1978), which enables determination of the final primary settlement (S) as well as the vertical coefficient of consolidation $c_v$. This graphical method was only applied to the stabilised column. Magnan & Deroy (1980) showed for a case where drainage is one-dimensional that $c_v$ can be derived from the plot of $S(i)$ versus $S(i-1)$ using the following expression:

$$c_v = \frac{-4 H^2 \ln \beta}{\pi^2 \Delta t}$$  \hspace{1cm} (6.1.)

Where:

H: height of the drainage path.

S(i): settlement at time i.

f: slope of the $S(i) - S(i-1)$ line obtained from the graphical construction.

$\Delta t$: time step used for plotting $S(i) - S(i-1)$ points.

The derived expression of $c_v$ is only applicable if the soil layer is homogeneous and the consolidation of the soil is one-dimensional, however this method is not applicable for soils exhibiting high creep. The one-dimensional consolidation of the column could be
justified by the fact that higher \( c_v \) values are expected in the column due to much lower values of \( m_v \) of the stabilised material compared with those of the peat. The theoretical justification of this graphical method can be found in Magnan & Deroy (1980).

Figure 6.7 shows the graphical construction used for the stabilised column. The settlement results plotted are those observed at S5 \((z = 0.50\text{m}, r = 0.00\text{m})\).

![Graph of Asaoka's graphical construction method applied to settlement observations at S5](image)

Figure 6.7. Asaoka’s graphical construction method applied to settlement observations at S5 (top of the stabilised column).

For the applied loads of 40 and 50 kPa, the slope of the line \( S_i = f(S_{i-1}) \) was found to be 0.36 and 0.67 which, respectively, yield values of \( c_v \) of \( 2.15 \times 10^{-6} \text{ m}^2/\text{s} \) and \( 1.69 \times 10^{-6} \text{ m}^2/\text{s} \).

### 6.2.5 Total earth pressure

Out of the three total earth pressure cells used in this project only one pressure cell has given continual reading (i.e. PC3). Only the total pressure reported onto the peat has been recorded during the test. Therefore, no comparison can be made of the ratio of load taken by the column to that taken by the peat. Variation of the total earth pressure recorded at PC3 with time is shown in Figure 6.8. It can be observed that:

(i) For the first two loads applied, the total pressure measured on the top of the peat layer was found to increase with time. The maximum measured total pressure was reached during the unloading stage and was of the order of 20 kPa. During the reloading stage, the measured total pressure was found to decrease with time,
(ii) Some inconsistent results can be observed in the reload stage. The total earth pressure reported onto the peat seems to decrease as the external applied load increases.

![Loading sequence](image)

Figure 6.8. Total pressure change with time. Total earth pressure cell PC3 ($z = 0.50\text{m}$, $r = 0.60\text{m}$).
6.2.6 Temperature measurements

Figure 6.9 shows the variation in temperature change (i.e. difference between temperature in the column and in the adjacent peat layer) with time. The temperature readings in the column were recorded from the time of its formation.

Figure 6.9. Temperature change with time in the stabilised column.

The maximum increase in temperature occurred in the top layer of the column and was of the order of +9°C. This was reached within two days of formation of the stabilised column and then reduced to be equal to that of the peat after 40 days.

6.3 ASSESSMENT OF THE STRENGTH OF THE STABILISED PEAT

The strength of the stabilised peat in the testing chamber was assessed by in-situ vane test, CPTU test and by unconfined compression tests carried out on specimens recovered from the chamber. Some specimens of the stabilised material, recovered from the batches used for forming the stabilised column and mass, were stored in the laboratory and tested for unconfined compression.
The objective of these tests was to: (a) measure the geotechnical parameters of the stabilised peat specimens recovered from the chamber; (b) determine the homogeneity of the stabilised column. The programme of testing performed is shown in Table 6.3.

Table 6.3. Programme of tests performed.

<table>
<thead>
<tr>
<th>Test performed</th>
<th>Type</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>CPTU</td>
<td>Penetration</td>
<td>Location: 380 and 750mm from concrete wall</td>
</tr>
<tr>
<td>Consolidation</td>
<td>Oedometer</td>
<td>Stabilised mass: surface</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Stabilised column: surface, 0.85m</td>
</tr>
<tr>
<td>Permeability</td>
<td>Triaxial cell // Rowe cell</td>
<td>Stabilised mass: surface</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Stabilised column: surface</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Stabilised column: 0.7m</td>
</tr>
<tr>
<td>Triaxial test</td>
<td>CDTC</td>
<td>Stabilised column: 0 - 0.20m, 0.40 - 0.60m, 1 - 1.20m Laboratory-stored specimen (column)</td>
</tr>
<tr>
<td>Unconfined compression</td>
<td>UC</td>
<td>Stabilised mass: surface, 0.20m</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Stabilised column: surface, 1m</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Laboratory-stored specimens (column and stabilised mass)</td>
</tr>
<tr>
<td>Vane shear test</td>
<td>Hand Vane</td>
<td>Stabilised mass: surface, 0.20m</td>
</tr>
</tbody>
</table>

Notations:
CPTU: cone penetration test with pore pressure measurement
CDTC: consolidated drained triaxial compression
UC: unconfined compression test

6.3.1 CPTU test results

The CPTU test results for both locations are shown in Figure 6.10a and b. The cone resistance, pore pressure and sleeve friction that developed during the test were measured.
Figure 6.10a. CPTU profile of the stabilised mass and peat layer.

Figure 6.10b. CPTU profile of the stabilised mass and stabilised column.

The undrained shear strength profile was estimated from the cone resistance $q_c$ by assuming the following expression: $c_u = q_c/17$ which was calibrated against values of the undrained shear strength obtained from the vane shear test.
The undrained shear strength profile obtained is shown in Figure 6.11. Results obtained from vane shear tests are plotted in the same graph (open circles).

![Undrained shear strength profile](image)

Figure 6.11. Undrained shear strength profile along the testing chamber.

From Figure 6.11 it can be observed that:

(i) For the first penetration point, which was put down outside the stabilised column, the top of the peat layer is clearly discernible at a depth of 0.40m. The average undrained shear strength of the peat is of the order of 11 kPa,

(ii) The undrained shear strength of the stabilised mass was found to vary between 50 kPa and 175 kPa with average undrained shear strength of 110 kPa, which was of the same order of the design strength of 100 kPa. The vane shear tests yielded an undrained shear strength of about 50 kPa at the surface of the stabilised mass and 100 kPa at a depth of 0.2m,

(iii) The strength profile of the stabilised mass was found to be similar at both test locations except for the maximum value of the undrained shear strength, which was higher at the centreline of the calibration chamber,

(iv) The undrained shear strength profile of the stabilised column was found to be heterogeneous. The undrained shear strength was found to decrease with depth.
Values of undrained shear strength derived from the CPTU test were found to lie between 25 kPa to 150 kPa. Overall, the undrained shear strength design value (i.e. 150 kPa) for the stabilised column had not been reached.

(v) On the basis of the CPTU results, the stabilised column can be divided into three main sections with respect to the undrained shear strength: (1) 0.40m–0.80m, the undrained shear strength varies between 100 and 150 kPa, (2) 0.80m–1.00m, the undrained shear strength varies between 50 and 100 kPa, (3) 1.00m–bottom of the column, the undrained shear strength is below 50 kPa except for a peak of 70 kPa recorded at a depth of 1.25m.

### 6.3.2 Unconfined compression tests

Unconfined compression tests were carried out on both specimens recovered from the testing chamber (i.e. U100mm and 65mm diameter samples) after completion of the test and from specimens cured in the laboratory from the same batches of stabilised material used for the testing chamber. These latter specimens were loaded in the same manner as the soil in the chamber. The unconfined compression tests were carried out at a curing age of 7, 28 and 90 days for the laboratory-stored specimens. Results of the unconfined compression tests are presented in Table 6.4.

#### Table 6.4. Results of the unconfined compression tests.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Age, (days)</th>
<th>Depth, (m)</th>
<th>$q_u$, (kPa)</th>
<th>$E_{50}$, (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stabilised mass</td>
<td>7</td>
<td>/</td>
<td>22.5</td>
<td>1340</td>
</tr>
<tr>
<td>Stabilised mass</td>
<td>28</td>
<td>/</td>
<td>16.2</td>
<td>770</td>
</tr>
<tr>
<td>Stabilised mass</td>
<td>90</td>
<td>/</td>
<td>25</td>
<td>1980</td>
</tr>
<tr>
<td>Stabilised mass</td>
<td>/</td>
<td>surface</td>
<td>107</td>
<td>3147</td>
</tr>
<tr>
<td>Stabilised mass</td>
<td>/</td>
<td>0.2</td>
<td>113</td>
<td>3445.12</td>
</tr>
<tr>
<td>Stabilised column</td>
<td>7</td>
<td>/</td>
<td>123.4</td>
<td>7250</td>
</tr>
<tr>
<td>Stabilised column</td>
<td>28</td>
<td>/</td>
<td>67.72</td>
<td>3600</td>
</tr>
<tr>
<td>Stabilised column</td>
<td>90</td>
<td>/</td>
<td>127.56</td>
<td>12750</td>
</tr>
<tr>
<td>Stabilised column</td>
<td>/</td>
<td>surface</td>
<td>141.2</td>
<td>2854</td>
</tr>
<tr>
<td>Stabilised column</td>
<td>/</td>
<td>1.0</td>
<td>125.1</td>
<td>2746</td>
</tr>
</tbody>
</table>
Figure 6.12 shows a plot of $E_{50}$ versus undrained shear strength for both laboratory-stored specimens and testing chamber specimens.

![Figure 6.12. $E_{50}$ versus undrained shear strength for laboratory-stored and testing chamber specimens.](image)

Results of the unconfined compression tests showed that a correlation $E_{50} \approx 152. c_u$ can be obtained for the laboratory-stored specimens. The unconfined compression test results for the stabilised column showed a good agreement with laboratory-stored specimens, cured at 90 days, and testing chamber specimens whereas, for the stabilised mass, $q_{uc}$ values obtained from laboratory-stored specimens, cured at 90 days, were on average 4 to 5 times lower than those of the testing chamber specimens. The secant stiffness modulus $E_{50}$ values obtained from testing chamber specimens were much lower than those obtained from laboratory-stored specimens.

### 6.3.3 Consolidation test results

Figure 6.13 shows results of the consolidation tests carried out on testing chamber specimens recovered from both the stabilised mass and column. Results are plotted in terms of void ratio versus effective consolidation stress in a semi-log scale.
Figure 6.13. $e - \log \sigma'_v$ for testing chamber specimens.

Table 6.5 shows the different parameters obtained from the consolidation tests.

Table 6.5. Consolidation parameters for the testing chamber specimens.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$\sigma'_p$, (kPa)</th>
<th>$C_c$</th>
<th>$c_v$, (m²/year)</th>
<th>$C_\alpha (= \Delta e/\Delta logt)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stabilised column (surface)</td>
<td>140</td>
<td>2.02</td>
<td>1.36 - 7.9</td>
<td>0.01 - 0.08</td>
</tr>
<tr>
<td>Stabilised column (0.85m)</td>
<td>120</td>
<td>2.47</td>
<td>1.93 - 7.5</td>
<td>0.01 - 0.065</td>
</tr>
<tr>
<td>Stabilised mass (surface)</td>
<td>110</td>
<td>2.55</td>
<td>3.23 - 7.1</td>
<td>0.045 - 0.1</td>
</tr>
</tbody>
</table>

Notation:

- $\sigma'_p$: preconsolidation pressure or yield pressure.

From Tables 6.4 and 6.5, it can be observed that the ratio $\sigma'_p / q_u$ ($q_u$ being the unconfined compressive strength) ranges between 0.96 – 0.99 for the testing chamber specimens.

6.3.4 Permeability test results

The results of constant head permeability tests carried out on both specimens recovered from the stabilised mass and stabilised column are shown in Table 6.6.
Table 6.6. Coefficient of permeability of testing chamber specimens.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Depth, (m)</th>
<th>Hydraulic gradient $i$</th>
<th>Coefficient of permeability $k_v$, (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stabilised column</td>
<td>Surface</td>
<td>20</td>
<td>$1.07 \times 10^{-8}$</td>
</tr>
<tr>
<td>Stabilised column</td>
<td>0.7</td>
<td>25</td>
<td>$4.40 \times 10^{-8}$</td>
</tr>
<tr>
<td>Stabilised mass</td>
<td>Surface</td>
<td>20</td>
<td>$1.60 \times 10^{-8}$</td>
</tr>
</tbody>
</table>

It is important to note that the permeability of the stabilised material is of the same order of magnitude as the original peat. Variation of the permeability with void ratio for a stabilised column specimen is shown in Figure 4.25.

6.3.5 Triaxial test results

Consolidated drained triaxial compression tests (CDTC) were carried out on saturated specimens of stabilised material recovered from the stabilised column and also on one specimen recovered from the stabilised material batch used for the stabilised column. The CDTC tests carried out were all consolidated to a pressure of 50 kPa prior to shearing. Results obtained are shown in Figures 6.14a, b and c.

Figure 6.14a. Stress-strain curves for the stabilised column specimens.
From results of the CDTC tests on the stabilised column specimens it can be seen that:

(i) Failure of the specimens recovered from the stabilised column had not been reached within the range of axial strains applied (12 to 15%). For the laboratory-stored specimen, failure was reached at an axial strain of about 10%,

(ii) It can be seen that the small strains stiffness of the stabilised material is non-linear. The specimen from a depth of 0.20 - 0.40m yielded the highest stiffness. The
laboratory-stored specimen displayed the highest stiffness among all the specimens tested,

(iii) The plot of the volumetric strain \( \varepsilon_v = \varepsilon_a + 2.\varepsilon_r \) versus axial strain shows that the compression of the stabilised peat in CDTC tests is almost one-dimensional.

6.4 FINITE ELEMENT BACK-ANALYSIS OF THE TESTING CHAMBER TEST

6.4.1 Material properties

A back-analysis of the testing chamber test was performed using the Plaxis finite element model as described in § 5.3.3.3. The parameters were derived from the laboratory tests for the peat and from both laboratory and CPTU tests for the stabilised material. The undrained shear strength profile used in the back analysis was derived from the CPTU test results. As the strength profile in the column was non-homogeneous, the column was divided into two layers having average undrained shear strength of 100 kPa and 50 kPa. The stabilised mass was modelled with average undrained shear strength of 100 kPa. The original peat was modelled using the soft soil model (SSM), the hard soil model was used for the stabilised column and the Mohr-Coulomb soil model for the stabilised mass.

The drained material properties used for the peat are shown in Table 6.7.

<table>
<thead>
<tr>
<th>( \gamma ) (kN/m(^3))</th>
<th>( \lambda^* )</th>
<th>( \kappa^* )</th>
<th>( c' ) (kPa)</th>
<th>( \phi^* ) (deg)</th>
<th>( p_c' ) (kPa)</th>
<th>( v' )</th>
<th>( k_v = k_h ) (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>11</td>
<td>0.3</td>
<td>0.06</td>
<td>1</td>
<td>30</td>
<td>15</td>
<td>0.2</td>
<td>10(^*)</td>
</tr>
</tbody>
</table>

Notations:

\( \lambda^* \): Modified compression index.
\( \kappa^* \): Modified swelling index.
\( p_c' \): Preconsolidation pressure.

From the soil models available in Plaxis, the soft soil model is the more appropriate for soils, which exhibit significant volume changes under load. It is assumed in this model that
there is a logarithmic relationship between the volumetric strain $\varepsilon_v$ and the mean effective stress $p'$ and is formulated by:

$$\varepsilon_v - \varepsilon_v^0 = -\lambda^* \ln \left( \frac{p'}{p_o^0} \right)$$  \hspace{1cm} \text{(Virgin compression)}$$

and

$$\varepsilon_v^e - \varepsilon_v^0 = -\kappa^* \ln \left( \frac{p'}{p_o^0} \right)$$  \hspace{1cm} \text{(Unloading/reloading)}$$

One of the features of this model is that it has a memory for the preconsolidation pressure. The soft soil model parameters $\lambda^*$ and $\kappa^*$ are related to conventional oedometer parameters ($C_c$ and $C_s$) as follows:

$$\lambda^* = \frac{C_c}{2.3(1+e)} \quad \text{and} \quad \kappa^* \approx 1.3 \frac{1-v_{ur}}{1+v_{ur}} \frac{C_s}{1+e}$$

The material properties used for the peat were derived from laboratory tests carried out on Ballydermot peat samples – see § 3. The parameters $\lambda^*$ and $\kappa^*$ are related to $C_c$ and $C_s$ as shown above and could be easily derived from consolidation tests, $p'_c$ is the preconsolidation as derived from the $e$-$\log\sigma'_v$ curves obtained from consolidation tests. The value of the cohesion intercept of the peat was set at unity. Consolidated drained triaxial tests performed on Ballydermot peat samples showed that there was no cohesion intercept (i.e. $c' = 0$), however it is a requirement of the Plaxis finite element model that $c' \geq 1$ kPa. The value of permeability used was derived from constant head permeability tests carried out on Ballydermot peat samples.

It should be noted that the soft soil model used in this version does not account for secondary compression of the soil. The soil parameters and the soil model used were found to realistically represent the recorded behaviour of Ballydermot peat in a standard oedometer test as can be seen from Figure 6.15.
Figure 6.15. Modelling of an oedometer test on Ballydermot peat specimen.

The drained material properties used for the stabilised material are shown in Table 6.8.

Table 6.8. Drained material parameters of the stabilised mass. Mohr-Coulomb soil model.

<table>
<thead>
<tr>
<th></th>
<th>$\gamma$ (kN/m$^3$)</th>
<th>$E'_{50}$ (kPa)</th>
<th>$c'$ (kPa)</th>
<th>$\phi'$ (deg)</th>
<th>$\nu'$</th>
<th>$k_v = k_h$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stabilised mass</td>
<td>12</td>
<td>2000</td>
<td>40</td>
<td>35</td>
<td>0.15</td>
<td>$10^{-8}$</td>
</tr>
</tbody>
</table>

The stabilised column was modelled using the hard soil model. This soil model assumes a hyperbolic relationship between the vertical strain and the deviatoric stress in primary triaxial loading. The drained material properties used for modelling the stabilised material in the column are shown in Table 6.9.

Table 6.9. Drained material parameters of the stabilised column. Hard soil model.

<table>
<thead>
<tr>
<th></th>
<th>$\gamma$ (kN/m$^3$)</th>
<th>$E'_{50}^{ref}$ (kPa)</th>
<th>$E'_{ur}^{ref}$ (kPa)</th>
<th>$m$</th>
<th>$\nu'$</th>
<th>$\phi'$ (deg)</th>
<th>$c'$ (kPa)</th>
<th>$K_v = k_h$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer I*</td>
<td>12</td>
<td>2000</td>
<td>4000</td>
<td>0.8</td>
<td>0.15</td>
<td>35</td>
<td>40</td>
<td>$10^{-8}$</td>
</tr>
<tr>
<td>Layer II**</td>
<td>12</td>
<td>1000</td>
<td>2000</td>
<td>0.8</td>
<td>0.15</td>
<td>35</td>
<td>20</td>
<td>$10^{-8}$</td>
</tr>
</tbody>
</table>

*: $c_u = 100$ kPa

**: $c_u = 50$ kPa

$E'_{50}^{ref}$ : Primary loading stiffness
$E_{ur}^{ref}$: Unloading/reloading stiffness

$m$: power in stiffness law

The stiffness modulus $E'_{50}$ used for the stabilised material was derived from consolidation tests carried out on stabilised peat samples recovered from the testing chamber after the completion of the test. By similarity to the confining conditions (i.e. 1-D compression) prevailing in the testing chamber, it seems more appropriate to derive $E'$ from consolidation tests. The variation of $E' = 1/mv$ with axial strain showed that $E' \approx 20.c_u$ ($c_u$ being the undrained shear strength) - see Figure 6.16. This relationship was used in the finite element analysis.

![Figure 6.16. Variation of $E' = 1/mv$ with axial strain for stabilised peat samples recovered from the testing chamber.](image)

From results reported within the EuroSoilStab project (Ahnberg et al. 2000), it was shown from consolidated drained triaxial compression tests carried out in Sweden that the angle of friction of the stabilised peat varied between 31° (measured after 1 month curing time) to 39° (measured after 1 year curing time). The cohesion intercepts evaluated from the same tests carried out in both Finland and Sweden were about 0.2 to 0.3$q_{uc}$ ($q_{uc}$ being the unconfined compressive strength). Subsequently, an effective angle of friction $\phi' = 35^\circ$ and a cohesion intercept $c' = 0.2q_{uc}$ were assumed for the stabilised Ballydermot peat in the testing chamber. The power factor $m$ used was set at 0.8. The permeability value used was derived from constant head tests carried out on stabilised peat samples recovered from the testing chamber.
Figure 6.17 compares the computed behaviour of the stabilised peat in a standard oedometer test, using the derived soil parameters, with the actual laboratory test result.

![Graph comparing measured and calculated effective consolidation stress](image)

**Figure 6.17.** Modelling of a standard oedometer test on a cement-stabilised peat specimen recovered from the testing chamber.

### 6.4.2 Calculation procedures

The calculation procedure used was as follows:

1. Initial stresses generation using the $K_0$ procedure (a lateral at-rest earth pressure of 0.35 was assumed for peat and 0.5 for the stabilised material),

2. Loading and unloading increments as described in § 6.2.1. Each increment was followed by a consolidation sequence for the entire duration of time specified in the loading sequence.

The boundary conditions used in the calculations are the same as those assumed in § 5.3.3.3. The effect of the outer boundary (i.e. concrete wall) was modelled by using an interface element, along the wall, where a strength reduction factor $R$ was varied to give the best match with the experimental results. The strength reduction factor $R$ is a parameter unique to Plaxis finite element model whereby the shear modulus of the interface is related to that of the soil by the following relationship: $G_{\text{interface}} = R^2 \cdot G_{\text{soil}}$. The interface was simulated by relating the shear modulus to that of the concrete wall and by selecting a very low value of the $R$. 

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6.4.3 Calculation results

6.4.3.1 Settlement results

Figure 6.18 shows the observed and calculated settlement results versus depth for applied loads of 20, 40 and 50 kPa. The settlement results reported are those observed in the centreline of the testing chamber. Settlements observed at settlement plate 3 were disregarded.

Figure 6.18. Vertical settlement versus depth at chamber centreline for both experimental and calculated results. (a) 20 kPa, (b) 40 kPa, (c) 50 kPa.
The calculated settlements for the applied loads of 20, 40 and 50 kPa showed very good agreement with the measured settlements along the chamber centreline. The settlements at the top surface and top of the column were estimated within a margin of ± 10mm. Calculated settlements for the applied load of 40 kPa almost coincided with those measured. However, the calculated settlement in the column underestimated the measured one (i.e. at \( z = 1.00 \)m). The difference was of the order of 10–20mm for the loads applied.

Figure 6.19 shows the distribution of calculated strains in the testing chamber for different loads applied.

![Calculated strain distribution in the testing chamber for different loads applied.](image)

From Figure 6.19, it can be seen that:

(i) The calculated strains in the stabilised mass were very low, compared to the stabilised column and the peat layer, for the entire loads applied. It was of the order of 2% for the last applied load of 50 kPa,

(ii) The calculated strain distribution in the stabilised column was found to be lower at the upper third layer (\( z = 0.50 \)m - 1.00m) than in the remaining bottom layer. For the first applied load of 20 kPa, the computed results showed the column to compress by about 1.26% and 2.04% in the upper and bottom layer and for the second applied load of 40 kPa by about 3.05% and 5.7% respectively and for the last applied load of 50 kPa by about 4.3% and 8.6%,
(iii) The calculated strain in the peat was found to be higher in the upper layer ($z = 0.50$-$1.00m$) than in the bottom layer. For the first load increment of 20 kPa, the calculated strain in the peat layer was of the order of 3.2% and 1% in the upper and bottom layer respectively. The final calculated strain in the peat layer was 11.8% and 5.3% for the upper and bottom layer.

(iv) The calculated strain distribution in the chamber shows that the stabilised mass experiences very little compression and that most of the compression occurs in the column and the adjacent peat layer. For the layer comprised between ($z = 0.50m$-$1.00m$) the compression is higher in the peat layer than in the column whereas in the bottom layer ($z = 1.00m$-$2.00m$) the compression is higher in the column than in the adjacent peat layer.

The back-analysis of the consolidation stages, which followed the application of the load increments, is mainly dependent on the permeability values used and the assumed drainage boundary conditions. The permeability values used for the stabilised material were measured on specimens recovered from the testing chamber after completion of the test. Figure 6.20 shows the measured and calculated settlement-time curves at the top surface, top of the column and the top of the peat layer. Results are only shown for the loading stages.

The shape of the calculated settlement-time curves provides a very good estimation of the settlement-time behaviour observed for settlement plates S7 ($z , r = 0.00m , 0.60m$), S5 ($z , r = 0.00m , 0.50m$) and S2 ($z , r = 0.50m , 0.60m$). The shape of the experimental settlement-time curve was very well captured.
Figure 6.20. Calculated and measured settlement-time curves. (a) S7, (b) S5, (c) S2.

From the back analysis results, the coefficient of consolidation of the stabilised column was derived from the plot of the degree of consolidation U versus time (in a semi-log scale). Since it is assumed that there is no radial flow from the peat to the column, the $T_v$ value determined for 50% consolidation yielded a value of $c_v$ equal to $1.27 \times 10^{-6} \text{ m}^2/\text{sec}$.

The simulation of the unload/reload stages at the top of the column can be seen in Figure 6.21. The unload/reload part is modelled by elasticity for which a stiffness value is used. For the soft soil model used for modelling peat, the unload/reload stage is simulated by $\kappa^*$ which is derived from standard oedometer tests, whereas for the hard soil model unload/reload is simulated by using a value of stiffness $E_{ur}^{ref}$ usually set equal to $2.E_{50}$. The computed unload/reload settlements seem to agree very well with those measured.

Figure 6.21. Calculated and measured settlement-time curve for S5 ($z = 0.50\text{m}, r = 0.00\text{m}$).
Figure 6.22 shows contours of vertical displacements occurring in the testing chamber. This figure shows clearly the effect of the concrete wall boundary on the vertical displacements of the peat layer and the column (the stabilised mass is surrounded by PVC sheeting therefore free to move). The settlement profile in the peat layer is not uniform, less settlement occurs close to the concrete wall. For the first load increment of 20 kPa, the settlements in the column where fairly uniform, however as the load is increased, the settlements become less uniform over the same cross section.

The calculated incremental displacement fields are shown in Figure 6.23. It can be clearly seen that a shear plane is forming within the column as the applied load is increased (i.e. this can be seen by the arrows of the displacement field of the stabilised material in the column). Also, some slippage along the concrete wall starts taking place at the transitional layer between the stabilised mass and the peat as the load applied is increased. This can be clearly observed for load increments of 40 and 50 kPa by the displacement field of the peat along the concrete wall.
Figure 6.22. Calculated contours of vertical displacements for different applied loads. (a) 20 kPa, (b) 40 kPa, (c) 50 kPa.
Figure 6.23. Calculated incremental displacement field for different applied loads. (a) 20 kPa, (b) 40 kPa, (c) 50 kPa.
6.4.3.2 Load distribution between the stabilised column and the peat

The computed distribution of effective stresses with depth (average effective stress for the column), after full consolidation, in the stabilised mass, in the stabilised column and in the adjacent peat section for the applied loads of 20, 40 and 50 kPa is shown in Figure 6.24.

![Effective stress distribution](image)

Figure 6.24 Variation of effective stresses with depth for different load increments. (a) Stabilised mass and column, (b) stabilised mass and peat. (after full consolidation).

The distribution of normalised stress in the peat layer with radial distance is shown in Figure 6.25. The distribution is shown for the load increment of 20 kPa and is illustrative of the rest of the load increments applied. The results are plotted in terms of the normalised stress (i.e. ratio of the effective stress in the peat to the external applied stress) versus r/a (i.e. ratio of the radial distance from the chamber centreline to the radius of the column) for different normalised depths z/H, z being the depth and H the total depth of the chamber.

![Normalised stress distribution](image)

Figure 6.25. Variation of normalised stress distribution in the peat with radial distance.
Figure 6.26 shows the variation of the stress concentration with depth. The stress concentration is defined as the ratio of the effective stress in the column to the effective stress in the peat layer.

![Stress concentration ratio graph](image)

**Figure 6.26.** Variation of the stress concentration ratio with depth for different load increments.

From Figures 6.24, 6.25 and 6.26 and it can be seen that:

(i) For all the load increments applied, the arching developing at the top of the column can be clearly seen in Figure 6.24a. This arching is more significant as the load is increased from 20 to 40 kPa and to a lesser extent when it is increased from 40 to 50 kPa,

(ii) The effective stresses in the adjacent peat section were of the order of 5-9 kPa for the first load increment of 20 kPa and remained fairly constant over the entire depth of the peat layer. As the load was increased, the effective stress distribution in the adjacent peat section was found to decrease with depth. For the last applied load of 50 kPa, effective stresses in the peat were comprised between 14 kPa and 27 kPa,

(iii) From Figure 6.25, it can be seen that the distribution of effective stresses in the peat layer are fairly constant within a radial distance of 0.60m from the chamber centreline. The effective stress then increases significantly close to the concrete boundary,
The distribution of the stress concentration ratio, as shown in Figure 6.26, shows a maximum at the top of the column and then decreases with depth. For the first applied load of 20 kPa, the stress concentration ratio was found to increase over the upper third layer of the column, whereas for the applied loads of 40 and 50 kPa the stress transfer zone was smaller. The stress concentration ratio between the stabilised column and the adjacent peat layer was found to vary between 2.5 to a maximum of 3.6 for the first applied load of 20 kPa.

The arching effect developing at the top of the column could be better seen in Figure 6.27. Some arching can also be observed along the interface peat/concrete wall at the transitional layer between the stabilised mass and the peat. The arching developing at the top of the stabilised column and along the interface peat/concrete wall is shown to increase in intensity as the load applied is increased.

6.4.4 Efficiency of the stabilisation technique

A comparison between the case where the peat in the testing chamber is treated and not treated was carried out in order to assess the efficiency of the stabilisation technique in reducing the overall amount of settlements. The behaviour of the non-treated peat was simulated using the same soil model and parameters as in § 6.4.1.

Figure 6.28 shows a comparison of the maximal calculated settlements at the top surface of the calibration chamber for both cases. Summary of the calculated settlements is also shown in Table 6.10.
Figure 6.27. Principal effective stresses in the testing chamber.
Figure 6.28. Computed final settlements versus applied load for treated and non-treated peat.

Table 6.10. Summary of the maximal calculated settlements for treated and untreated peat.

<table>
<thead>
<tr>
<th>Load, (kPa)</th>
<th>20</th>
<th>40</th>
<th>50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Settlement $S_u$, (mm)</td>
<td>118</td>
<td>288</td>
<td>361</td>
</tr>
<tr>
<td>Settlement $S_t$, (mm)</td>
<td>30</td>
<td>82.7</td>
<td>122.6</td>
</tr>
<tr>
<td>Ratio $S_t/S_u$</td>
<td>0.25</td>
<td>0.28</td>
<td>0.33</td>
</tr>
</tbody>
</table>

$S_u$: maximal calculated settlement for the case of untreated peat.
$S_t$: maximal calculated settlement for the case of treated peat.

These calculations show that the stabilisation technique had a very beneficial effect in reducing the final settlement of the peat by an average ratio of 0.28.

6.5 SUMMARY

The experimental results of the calibration chamber test were presented. The material in the chamber was subjected to loading and unloading stages. The recorded settlement, pore pressure, earth pressure and temperature data throughout the test was presented.

The CPTU test carried out after the completion of the test showed that the strength of the stabilised material in the column varied with depth. Stabilised peat specimens were recovered from the chamber and tested in the laboratory. Visual inspection of these specimens revealed that the mixing of the original peat was not uniform and that some unstabilised peat lumps...
still subsist in the cement column and that this accounts for the non-uniformity in the column strength.

A back analysis of the calibration chamber test was performed using a finite element method. The soil parameters used in this numerical model were derived from both in-situ and laboratory tests. The results showed good agreement between the overall experimental behaviour and the simulated one.

The analysis of the experimental and back-analysed results is presented in chapter 7.
CHAPTER 7

DISCUSSION OF THE EXPERIMENTAL RESULTS
7. DISCUSSION OF THE EXPERIMENTAL RESULTS

7.1 INTRODUCTION

This chapter interprets the findings of the experimental results obtained in the laboratory and relates these to the expected behaviour and to the findings of other researchers. The discussion covers both results obtained on laboratory mixed stabilised peat and those obtained from tests on samples recovered from the large-scale testing chamber. The results of the back-analysis of the testing chamber test will also be discussed.

7.2 DESIGN AND STRENGTH OF BINDERS FOR ORGANIC SOILS

The results of unconfined compression tests carried out on laboratory-mixed Raheenmore peat specimens stabilised with different binders showed that the highest strength achieved was by an (85:15) mixture of blast furnace slag-gypsum. The addition of the gypsum to the slag resulted in approximately a threefold strength increase compared with the strength obtained with the blast furnace slag alone. However, this mixture performed very poorly when mixed with Ballydermot peat. The highest strength achieved for stabilised Ballydermot peat was obtained when using cement as a binder, although the strength achieved was lower than that achieved for cement-stabilised Raheenmore peat.

This variation in strength gain was also recorded by other researchers (Ahnberg & Holm 1999; Lahtinen et al. 1999; den Haan 1998). Ahnberg & Holm (1999) have investigated the strength improvement of two peats from Sweden stabilised with different types of stabilisers. The results of the unconfined compression tests showed that a combination of cement-slag stabiliser in a proportion of 50:50 yielded the highest strength for both peats. A combination of cement-slag-gypsum was found to be effective but to a lesser extent than cement-slag mixture. Lahtinen et al. (1999) tested two peats from Kivikko and Soderhamn (Finland). Both were quite similar in terms of organic and water content although, the results obtained from each were quite different. The highest strength for Kivikko peat was obtained using cement whereas a combination of cement and slag yielded the highest strength for the Soderhamn peat. With this combination, the difference in strength achieved for both peats was almost threefold. Even though no explanations were given by the
authors for this disparity in behaviour, these results together with the tests carried out on the Ballydermot and Raheenmore peat, indicate that the unconfined compressive strength obtained for a particular mixture of binders can be very different for peats having similar water and organic contents. Therefore, the chemical properties of the peat are more likely to be the determinant factor for the reactions between the binder and the peat. The determination of the type of peat tested (i.e. sphagnum, carex, reed, wood and cotton grass) is important for a stabilisation work as the chemical properties of peat vary with the type of plant. Huttunen & Kujala (1996) showed the sphagnum peat achieved the highest strength when stabilised and the carex peat the lowest. Mathur & Farnham (1985) found that the humic, fulvic acids and bitumen content varied with the type of peat, with the carex peat having much higher concentration of humic acids than the sphagnum peat.

Huttunen & Kujala (1996) reported that strength gain achieved by stabilisation is less significant as the degree of decomposition increases. This is in agreement with the results obtained on Irish peats, as the strength achieved for the stabilised Ballydermot peat, which is at a nearly complete stage of decomposition (H8-H9 on the Von Post scale), was lower than that of the stabilised Raheenmore peat, which is slightly decomposed (H2). The degree of decomposition has a great influence on the chemical properties of the peat. Tsutsuki & Kuwatsuka (1978) reported that the COOH functional group found in the humic acids increases during humification of the organic matter and according to Stevenson (1994), the maximum binding capacity of humic acids for any given metal ion is approximately equal to the content of acidic functional groups, primarily COOH. This could possibly explain the decrease in strength achieved with the advanced degree of decomposition of the peat stabilised.

The addition of gypsum to the blast furnace slag was, in most cases reported in the literature, found to be satisfactory in improving the strength of stabilised organic soils (den Haan 1998; Ahnberg & Holm 1999). The addition of gypsum would favour the formation of ettringite in the stabilised soil. The ettringite has a needle-shaped like crystals and is known to have a capacity to expand to about 4 to 5 times its volume. If one considers the microstructure of the peat as shown in Figure 3.9, it is characterised by a very open cellular microfabric, which suggests that the efficient binder should have both functions of strengthening and filling the voids. The ettringite needle-shaped like crystals will then grow freely in the large voids if they do not encounter any obstacle (i.e. the larger the
voids, the more space is available for the ettringite to expand and the more beneficial is the
effect on reducing the volume of voids of the stabilised soil) thus, forming a tight mesh of
rods, reaching lengths of a couple of μm.

The addition of gypsum to the blast furnace slag did not enhance the strength of
Ballydermot peat. Similar results were obtained by Ahnberg & Holm (1999) on the
stabilisation of two types of gyttja from Domle and Holma (Sweden). The addition of
gypsum to the cement-slag proved to be effective in stabilising Domle gyttja but unsuitable
for the Holma gyttja. The difference between the two gyttja was in the clay content, which
was about 80% for Holma gyttja and 36% for Domle, and in the pH, 8.5 for Homla gyttja
and 5.4 for Domle gyttja. Huang (1997) showed that the beneficial effect of adding
gypsum depended upon the degree of saturation of the pore water with Ca(OH)₂. If the soil
absorbs a little amount of Ca(OH)₂, cement silicate hydrates CSH will grow normally but
ettringite in high concentrations of Ca(OH)₂ will destroy the structure formed in the
stabilised peat. As found in § 3.3.3, the capacity of Ballydermot peat to absorb calcium
from a saturated solution of Ca(OH)₂ is half than that of Raheenmore peat. This is
consistent with findings of Huang (1997).

The capacity of a soil to absorb calcium ions from a saturated solution of Ca(OH)₂ would
be an important indicator in assessing the possible effect of adding the gypsum to the slag.

Combinations of pfa-cement-lime binders gave poor unconfined compressive strengths.
The strength achieved was found to decrease with a decrease in the proportion of cement in
the mixture. A combination of pfa:cement in a proportion of (80:20) did not give any
improvement in the strength of the stabilised peat compared to the strength of the original
peat. This trend was somewhat expected when dealing with organic soils with little or no
minerals present. In order for the pfa and lime to react with the soil, a minimum amount of
clay particles must be present in the soil. Results reported in the literature (Cortellazo &
Cola 1999; Lahtinen et al. 1999) support this argument.

The strengths achieved for both stabilised Raheenmore and Ballydermot peats were found
to increase with curing time. The gain in strength with time was monitored for 28 days for
Raheenmore peat and for 1 year for Ballydermot peat. Cement stabilised peat was found to
perform best at 7 days curing time. The blast furnace slag-based binders achieved very low
strength at 7 days curing time and this could be due to the slow reaction time of the blast furnace slag itself. However, the increase in strength after 28 days was very significant. The strength of the cement-stabilised Ballydermot specimens showed an increase over 1 year curing period. The strength achieved doubled between 7 and 28 days and was still increasing after 28 days, almost doubling after 1 year. This increase in strength suggests that the cement reactions were still in progress.

Locat et al. (1990) have presented a mechanical conceptual model for the development of the shear strength of sensitive clays stabilised with lime. This model could also be applied to the cement-stabilised Ballydermot peat as it can be seen in Figure 7.1.

The strength development over time for the cement-stabilised peat can be divided into two main phases. The first phase is characterised by a rapid increase in strength, which accounts for the greatest portion of the final strength. The second phase is characterised by a gradual increase in strength with time, which is much slower than in the first phase. From a practical point of view, it is more beneficial to design stabilisation work in the field for a curing age of 90 days, as 70% of the strength gain, recorded over a period of 365 days, occurred within this period of time.

If the increase in strength with time is plotted starting from the 7th day of curing time, it can be shown that the variation of the strength with time can be approximated by a

Figure 7.1. Mechanical conceptual model of shear strength development with time. (After Locat et al. 1990). LWC: low water content, HWC: high water content.
logarithmic relationship of the same form as that proposed by Brandl (1983):
\[ q_{u,t} = A \ln\left(\frac{t}{7}\right) + q_{u,7}, \]
where \( q_{u,t} \) is the unconfined compressive strength at a curing time \( t \) and \( A \) is a correlation factor. Figure 7.2 is illustrative of this approximation.

![Figure 7.2. Shear strength versus time factor (t/7).](image)

A relationship \( q_{u,t} = 132.9 \ln \left(\frac{t}{7}\right) + q_{u,7} \) was obtained for the cement-stabilised Ballydermot peat specimens. Similar trends were obtained by Ahnberg & Holm (1999) and Lathinen et al. (1999) on Domle peat stabilised with a mixture of slag-cement and on Kivikko clay stabilised with a mixture of gypsum-lime-cement.

### 7.3 MECHANISM OF INTERACTION BINDER/ORGANIC SOIL

The interaction of the binder and the organic soil was investigated using the scanning electron microscope. The products formed shortly after mixing and after 28 days curing time were examined. The original peat microstructure exhibited a very open cellular microfabric with large voids, which explains its high compressibility and also its high capacity for holding pore water. After dry-mixing the peat with a binder some of the pore water is bound to the cement grains and is therefore used for the hydration of the binder. Water content measurements after mixing showed a reduction of the water content from an average of 800%–900% to about 250%–300%.
From Figure 4.9, which shows scanning electron microscope pictures of cement-stabilised specimen after 1 day of mixing, the hydration of cement particles can be clearly seen by the formation of a honeycomb-like structure, which consists of microscopic fibre-like outgrowths at the surface of the cement grains—see Figure 4.9a and b. This structure creates very tight meshes if the cement grains are very close to each other, and therefore, start filling the large voids around the peat material. The large volume of voids observed in the original peat could explain somewhat the need for high amounts of binders, a minimum of 150 kg/m³ found for both Raheenmore and Ballydermot peat, in order to achieve a significant improvement in the strength of the stabilised peat.

In all the scanning electron microscope pictures examined, no evidence of interaction between the organic matter and the binder could be found. The presence of the hydrated cement particles could be seen in the peat pores but no direct contact with the organic matter was observed. Theoretically, this interaction is only possible if some clay minerals are present in the peat material in order to interact with the calcium hydroxide, released during hydration of cement, to form additional cementation compounds. Therefore, no binding occurs between the organic matter and the cement grains. This would suggest the use of the term "stabiliser" instead of "binder" for the cement that is mixed with the peat.

Based on a model for the hydration of cement proposed by Birchall et al. (1978) and also on the observations of the microstructure of the cement-stabilised peat, a mechanism of cement stabilisation for organic soils can be proposed as shown in Figure 7.3.

The proposed mechanism of the stabilisation of peat suggests that the closer the cement particles are, the tighter the mesh formed and also the more efficient is the stabilisation effect (i.e. more volume of voids is filled by the cementation components). This suggestion seems to support the beneficial effect of preloading the stabilised peat during curing. Preloading of the stabilised peat will compress the stabilised peat structure, thus reducing the volume of voids and bring a tighter packing of the cement particles which results in a higher concentration of cement particles per unit of volume. The experimental results obtained in this project showed that the strength achieved for preloaded stabilised peat specimens were almost five times higher than that of specimens not loaded—see Figure 7.10. However, Ahnberg (1999) stressed the fact that a delay in loading the
stabilised peat by as little as 4 hours results in a significant decrease in the strength achieved.

Figure 7.3. Proposed mechanism for cement stabilisation of peat.

Figure 4.11 shows a scanning electron microscope picture of the microstructure of a cement-stabilised specimen loaded immediately after mixing and cured for 28 days. It can be clearly seen that the peat pores are no longer discernible and a very tight mesh of CSH platelets is covering the surface of the peat.

The examination of the increase in temperature during hydration of cement mixed with both laboratory and peat pore water showed the retarding effect of the peat pore water. A delay of two hours was observed for the second main temperature peak for the cement-peat pore water paste - see Figure 4.12. The X-ray diffraction patterns obtained on both pastes were similar. The higher peak of Ca(OH)$_2$ observed for the cement-laboratory water paste indicates that more calcium hydroxide was released during the course of hydration of this paste compared to the cement-peat pore water paste. A similar trend was observed for the CSH (most important compound for strength production), but to a lesser extent. It is most likely that the deficit in calcium ions observed for the cement-peat pore water paste is due to the humic acids present in the pore water, which create complexes with calcium ions. Results shown in Figure 3.8 seem to support this view. The peat pore water exhibits a high
buffering capacity compared with laboratory water and this indicates that more calcium hydroxide will be neutralised by the peat pore water.

The XRD patterns for both pastes showed no changes in the phases formed in the cement-peat pore water paste. This confirms that the retarding effect of the organic matter on the hydration of cement is mainly due to the consumption of calcium ions and this reduces the number of these ions available for the pozzolanic reaction.

The chemical composition of the cement-stabilised Ballydermot peat pore water showed an increase in concentrations of (Na^+, Ca^{2+}, K^+ and Cl^-) and a decrease in the concentration of Mg^{2+} and SO_{4}^{2-}, relative to chemical composition of Ballydermot peat pore water. The increase in concentration of Na^+ and K^+ ions measured is typical of the hydration of cement (Odler 1998). The measured increase in chloride may suggest the presence of some dissolved salts such as calcium salts formed by the reaction of the humic acids with the compounds present in the cement (i.e. CSH and Ca(OH)_2). This reaction follows the basic reaction between an acid and a base and according to Lees (1991), for weak acids such as humic acids, the formation of salts continues until either the acid or the base is used up.

This raises some questions regarding the long-term strength of stabilised peat columns formed in an aggressive groundwater environment, most likely to be the case in organic soils. The attack of humic acids on the hydrated cement compounds leads to the disappearance of solids if the salts formed are soluble, leaving them free to be washed away.

7.4 MECHANICAL BEHAVIOUR OF LABORATORY-MIXED STABILISED PEAT

7.4.1 Behaviour in unconfined compression tests

The stabilised peat was found to become more brittle as the strength achieved increased (i.e. it failed at a lower strain and experienced a significant reduction after failure). The secant stiffness modulus E_{50} (i.e. secant stiffness modulus at 50% unconfined compressive strength) was derived for all tests and a correlation E_{50} \approx 100 \cdot c_u was obtained- see Figure 4.8. This correlation agrees well with reported data in the literature. Ekstrom (1994)
reported a ratio $E_{50}/c_u$ of about 50 to 150 times for specimens recovered from lime-cement columns and also for specimens mixed in the laboratory. However, results obtained from an unconfined compression test carried out with local strain measurements showed the stiffness modulus to be nearly threefold higher at low strains when compared with external strain measurements—see Figure 4.7. Consolidated undrained triaxial compression tests, with local strain measurements, carried out on similar specimens show clearly that $E_u$ derived from the $E_u/c_u$ relationship does not represent the recorded behaviour—see Figure 7.4. Therefore, it can be suggested that the relationship $E_{50}/c_u \approx 100$, frequently used in design, is a conservative representation of the stress-strain behaviour of stabilised peat.

![Figure 7.4. Normalised deviator stress $q/q_f$ obtained from CUTC tests versus axial strain.](image)

Failure strains in unconfined compression tests were found to vary depending on the strength achieved. For high undrained shear strengths, the failure strain was between 2.5% - 4%. For low undrained shear strengths, the failure strain was as high as 15%. Similar findings were reported by Ahnberg & Holm (1999) from results of unconfined compression tests carried out on a Swedish peat stabilised with different stabilisers.

Broms (1999) proposed the use of a failure strain of 5% in the design of columns with an undrained shear strength of less than 150 kPa, 3% for an undrained shear strength between 150 kPa and 300 kPa and 1% for undrained shear strengths exceeding 300 kPa. This proposition is in agreement with the results obtained on stabilised peat. However, for the range of undrained shear strength proposed by this author, slightly higher failure strains can be assumed for the stabilised peat tested in this project. Even at high undrained shear...
strengths, the failure strain was found to exceed 2.5%. The assumption that the peak strength of the column is mobilised at the same strain as the peak strength of the original soil is not valid for peat soils, as failure strain reported in the literature for peat are generally high.

7.4.2 Compressibility behaviour

Few studies were reported in the literature on this subject, particularly on stabilised peat. The addition of cement to the peat would indicate that the stabilised peat is more likely to behave as a cemented soil or at least exhibit some of the features unique to this type of soil. However, the examination of the e-logσ'v curves indicates the contrary.

The curved part defining the yield stress was well defined for all the tests carried out and the normally consolidated line did not exhibit any brittle behaviour comparable to cemented soils. The gain in strength was evident with time. The yield stress increased from 18 kPa (i.e. curing load applied) to about 210 kPa at 28 days and 520 kPa at 240 days curing time. This increase in yield stress is expected as it is in agreement with the increase in strength with curing time observed from unconfined compression tests.

The pre-yield behaviour displayed was typical of stiff soils, little deformation occurred in the pre-yield part as the applied stress was increased. The post-yield behaviour was the same for all the specimens tested regardless of the age of the specimen. All the normally consolidated lines of the specimens tested had similar slopes, suggesting that Cc is independent of the curing time of the specimen. This result indicates that the value of Cc obtained at an early curing time, for instance 28 days, can be used for different curing times. The ratio of $C_c/(1+e_0)$ was found to be about 0.35 which is comparable to that obtained for Ballydermot peat $C_c/(1+e_0) \approx 0.45$.

When considering the behaviour of cement-stabilised peat in compression, it could be deduced that the addition of cement to the peat increases the resistance of the stabilised peat to deformation up to the yield stress. The yield stress increases with longer curing time, however, all the consolidation curves, irrespective of the age of the specimens, seem to plot on the same normally consolidated line after yielding. This would
indicate that after yielding, the stabilised soil specimens, regardless of the curing age and the yield pressure, reach a same state.

Nagaraj et al. (1998), in a study on the compressibility behaviour of soft cemented soils, suggested that for stress increments below the yield stress, the cementation bonds would carry the additional stress entirely, with negligible deformation. At yield stress, some of the bonds, being subjected to stresses greater than the bond strength, will transfer the load carried by them to the frictional bonds. This model was proposed mainly for cemented fine-grained soils, where the soil grains are cemented to each other. However in the case of organic soils very little interaction, depending upon the amount of clay minerals present in the organic soil, occurs between the organic matter and the cement. From the observation of the microstructure of stabilised peat, as shown in Figures 4.9 to 4.11, the mechanism of strength gain is mainly by cement particles filling the volume of void space in the peat.

For a better understanding of the mechanism of compression of the cement-stabilised peat, it is helpful to compare the behaviour of the soil in its intact and remoulded state. Results plotted in Figure 4.18 show both normally consolidated lines of the intact and remoulded specimens to be almost identical after yielding. Remoulding of the specimen, at nearly the same water content, had no significant effect on the e-log $\sigma'_v$ curve suggesting that the intact structure has virtually no influence on the compressibility characteristics of the cement-stabilised peat post-yield. Diffley (2000) compared the behaviour of the cement-stabilised Ballydermot peat in both its intact and remoulded state. However, the specimen was initially remoulded to a state of slurry at water content near the liquid limit. Figure 7.5 shows a plot of the e-log $\sigma'_v$ curves obtained on preloaded, non preloaded and remoulded cement-stabilised peat specimens.

Remoulding of the soil at a water content near the liquid limit resulted in the elimination of the yield pressure. However, this effect was less obvious for the specimen remoulded at the same water content. The reduction of the yield stress for this specimen suggests that some of the cementation components were destroyed during remoulding. It appears that for all the intact specimens tested, the normally consolidated lines plot on a same line after yielding.
Figure 7.5. e-log $\sigma^*$, curves on cement-stabilised Ballydermot peat.

Comparison of the intact and remoulded (from a slurry) behaviour of the cement-stabilised peat suggests that after yielding, some of the cementation components are still contributing to the overall resistance to deformation of the soil. However, if the cementation components had an effect on the compressibility behaviour of the cement-stabilised peat post-yield, this would be reflected by a different “ncl” for the intact specimens having achieved different strengths, but, as shown in Figure 7.5, all the “ncl” for the intact specimens plot on the same line. Furthermore, the same slope observed in the unload-reload stages seemed to support the fact that the cementation components do not contribute to the resistance to deformation post-yield.

By analogy to the compression model proposed by Nagaraj et al. (1998), the stress component for a given void ratio could be considered to be the sum of the resistance of the original peat and a stress component resulting from the addition of cement to the peat (i.e. formation of a new structure having different void ratio, water content and specific gravity)- see Figure 7.6.
The values of the coefficient of consolidation $c_v$ for cement-stabilised Ballydermot peat were found to be about 5-6 m²/year at the start of the compression and then increased to about 15-20 m²/year as the load applied increased near the yield stress. This trend was different from that observed for the original peat, where $c_v$ was found to decrease with increasing the applied load. The range of $c_v$ values measured for the original peat was between 0.18-8.8 m²/year. It can be suggested that the addition of cement to the peat results in an increase in $c_v$ values. $m_v$ values were found to decrease with increasing load as would be expected. Average values measured ranged between 0.1 to 0.2 m²/MN- see Figure 4.20. These measured values are much lower than those measured on the original peat (5-8 m²/MN). Figure 7.7 shows the variation of $c_v$ with the applied load for both cement-stabilised peat and peat.

The permeability value of the cement-stabilised peat was found to be lower than that of the original peat. Nevertheless, $c_v$ values were higher for the cement-stabilised peat than in the original peat. This could be explained by the much lower values of $m_v$ measured on the cement-stabilised peat as $c_v = \frac{k_v}{m_v \gamma}$.
Values of the coefficient of secondary compression $C_{\alpha}$ obtained on the cement-stabilised peat were found to be about 10 times lower than those obtained on the original peat. The variation of $C_{\alpha}$, as shown in Figure 4.22, suggests that the creep behaviour of the cement-stabilised peat could be associated with a structural breakdown of the soil (i.e. significant increase in $C_{\alpha}$ occurs post-yield). As explained previously, after yielding, the cementation components do not contribute to the resistance to deformation of the cement-stabilised soil. Therefore more load is carried by the peat which is known to have high creep characteristics. This could possibly explain the significant increase in $C_{\alpha}$ after yielding.

The long-term consolidation test carried out at a curing age of 90 days showed the settlements to be a non-linear function of time. Factors such as hydration of cement with time and temperature changes may produce some physico-chemical changes in the soil sample that will in turn affect the secondary compression of the cement-stabilised peat. Settlements measured after 24 hours of applying a load increment accounted for about 70% of the total settlement measured within one month. However, further long-term testing should be carried out at constant temperature and at curing age where the soil has reached chemical stabilisation. The unload stages following the last load increment showed no revert to creep within the time increment applied (i.e.1 month).

The $C_{\alpha}/C_c$ ratio was higher at 28 days than at 90 and 240 days and this could be explained by the fact that the soil is still undergoing chemical changes between 28 days and 90 days.
curing time. The $C_\alpha/C_e$ ratio derived at 90 days curing time was similar to values reported by Mesri & Godlewski (1977) for a wide variety of natural soils.

### 7.4.3 Permeability

The permeability of cement-stabilised Ballydermot peat, preloaded immediately after mixing, was found to be lower than that of the original peat. Similar findings were reported by Ahnberg & Holm (1999). These authors showed the permeability of the stabilised peat to be about $10^{-8}$ m/s to $10^{-7}$ m/s compared with $10^{-6}$ m/s for the original peat. This decrease in permeability was ascribed by the authors to the compression of the samples caused by the initial loading (i.e. surcharge of 18 kPa applied immediately after mixing).

The curing procedure used in this project is believed to cause the observed decrease in the permeability of the cement-stabilised peat compared to the original peat. The initial curing load applied caused the stabilised peat to compress by an amount varying between 14.7% to 35% depending on the amount and type of stabiliser used. The permeability measured on a cement-stabilised sample, which had not been subjected to any loading during curing, was of the order of $7 \times 10^{-8}$ m/s, which is of the same order as that of the original peat. Furthermore, permeability values measured on the specimens recovered from the chamber were of the same order of magnitude as that of the original peat (chamber specimens were loaded after 2 months). This confirms the suggestion that the decrease in permeability is more likely to be due to the curing procedure applied to the stabilised peat.

Considering the previous observation, the permeability of the laboratory-mixed cement-stabilised peat cannot be considered to be higher than that of the original peat if a curing load is applied immediately after mixing. However, the extension of this observation to field conditions should be used with great caution. The energy of mixing applied in the field is lower than that applied in the laboratory and this is more likely to result in a less homogeneous mixing. The permeability measured on laboratory-mixed specimens is not representative of the structural features imposed on the columns by the mixing procedure used in the field (i.e. mixing tool and the rate of rotation). Data reported by den Haan et al. (2000) is illustrative of this. The authors found two main permeability regimes of $10^{-5}$ m/s and $10^{-8}$ m/s on the specimens extracted from stabilised organic soil columns whereas, permeability measurements on laboratory-mixed specimens yielded a value of about
The permeability of the cores extracted from the columns depended upon the bulk density of the soil samples, which is a good indicator of the efficiency of mixing.

According to the Nordic countries experience, where the dry mix method is used, the permeability of the stabilised columns is higher than that of the original soil and is often assumed 100-1000 times higher (Broms 1984). However, the Japanese experience on cement deep mixing commonly called the “wet method” showed the permeability of the cement columns to be lower than that of the original soil (Okumura 1996). This raises the issue of the effect of the mixing procedure used on the permeability of the stabilised soil. In the “dry method” the stabiliser is injected with air pressure as the mixing tool is withdrawn, whereas in the “wet method” the stabiliser is injected as a slurry both during insertion and during withdrawal of the mixing tool. The injection of the binder with air pressure is more likely to result in a less saturated soil. The presence of air bubbles entrapped in the soil makes the flow of water difficult and this should result in a reduction of the permeability of the stabilised soil. However, in the author's opinion, the higher values of permeability measured in the field are more likely to be due to the heterogeneity of the stabilised soil; less homogeneity is achieved with the "dry method". Therefore, drainage of the columns in the field occurs in the weak portions, which have not been successfully mixed. Data reported by den Haan (2000) seem to support this suggestion as the non-homogeneity of the stabilised material yielded two permeability regimes within the same stabilised material.

With reference to Broms (1998), lime columns are assumed to act as vertical drains and this therefore reduces the time required for the dissipation of the excess pore water pressures in the natural soil when loaded. From the results obtained on the cement-stabilised Ballydermot peat, the acceleration in the rate of consolidation does not imply a higher permeability of the stabilised peat than that of the original peat but is more likely to be due to higher $c_v$ values of the stabilised peat.

There is a sharp fall in the permeability of the stabilised peat after yielding. This was verified on both preloaded (i.e. loaded immediately after mixing) and non-preloaded cement-stabilised specimens. However, this fall in the permeability is not followed by a drastic decrease in void ratio. This could be due to a possible rearrangement of the micro fabric after yielding or possibly to a disappearance of the macro pores. Further
investigation using mercury micro porosimetry for measuring the volume of the pores at different stages of compression could give a better insight on this matter.

7.4.4 Triaxial tests

Examination of the results of the CUTC tests carried out on cement-stabilised peat showed a high build up of pore pressure at low stresses. For the specimen consolidated at 50 kPa, the maximum excess pore pressure occurred at a very low axial strain of 0.25%. An effective lateral stress of 3 kPa was recorded at failure. den Haan (1998) in a suite of CUTC tests carried out on a Dutch peat stabilised with a mixture of slag-cement and anhydrite found considerable dilatancy to develop at low confining pressures and failure occurred when the lateral effective stress reduced to zero. If one considers the stress paths obtained from the CUTC tests, the specimen consolidated at 50 kPa displayed a high dilatancy and its stress path lay very close to the tension cut-off line (i.e. $\sigma_3 = 0$). This behaviour should receive attention when carrying out stabilisation work in the field, for if the stabilised columns are loaded fast enough to assume undrained conditions, then little axial displacement is required for the lateral effective stress to reduce to zero.

Peak strength in CUTC tests was reached within 1.5% to 2% axial strain. These are slightly lower than those obtained from the unconfined compression tests. As reported by Tatsuoka et al. (1996) failure strains in cement-stabilised soil are as low as 0.1% to 1%. These values of failure strain raise the question whether full interaction could be expected between the cement columns and the adjacent original soil. In field conditions, this low failure strain is not sufficient to fully mobilise the undrained shear strength of the adjacent peat layer, as the peat is characterised by high failure strains.

The failure envelope obtained from CUTC tests on cement-stabilised peat was found to be curved. There seems to be an intermediate stress region where the maximum deviator stress at failure is not affected by the confining pressure. Considering the specimens consolidated at 200 and 300 kPa, both fail at the same deviator stress despite the difference of 100 kPa in the confining pressure. This suggests that the failure envelope is horizontal within this stress region. For the specimen consolidated at 350 kPa, the deviator stress at failure was found to increase.
Mitchell (1970), in a study of the yielding and mechanical strength of a naturally cemented Leda clay of Eastern Canada, found that the mode of failure of Leda clay in triaxial compression tests was dependent on the mean normal stress at failure. Three approximate regions could be defined, dividing thus the failure region into three regions of mean effective stress $p'$.

Similar work carried out by Uddin et al. (1999) on cement-treated Bangkok soft clay showed that the failure envelopes, for different amounts of cement, were curved and the overall curvature depended upon the range of $p'_0$ applied. The authors distinguished three main $(p,q)$ zones for the cement-treated clay. There was no report of an intermediate zone where peak strength is independent of the confining pressure prior shearing. However, after reaching peak strength, the residual points of the cement-treated clay plotted in a manner that they constituted an envelope referred to as destructured envelope. It was postulated by the authors that if complete destructuration of the treated clay were achieved, and neglecting any permanent change in structure, a destructured envelope would be developed where the behaviour of the treated clay is purely frictional.

The progressive destructuration that occurs in the cementation components as the confining stress is increased, within the yield stress, could possibly explain the curved failure envelope obtained for the cement-stabilised peat. After yielding, the cementation components are destroyed and as shown in the oedometer tests, these components do not contribute to the resistance to deformation of the cement-stabilised peat after yielding therefore, a frictional behaviour of the structure (i.e. peat and the destroyed cementation components) develops.

Following this, the examination of the failure envelope obtained from CUTC tests would suggest the presence of three main stress regions for the cement-stabilised peat, although no tests at higher confining stress than 350 kPa were carried out to confirm the behaviour of the soil at high stresses. These main stages are: (1) low stress region characterised by a high dilatancy. This is expected to yield a high effective angle of friction, (2) an intermediate region where the peak strength is not affected by the confining pressure. The failure envelope is horizontal and the behaviour of the cement-stabilised peat is believed to be governed by the cementation components, (3) a normally consolidated stress region.
characterised by a breakdown of the cementation components where, the cement-stabilised peat develops a frictional behaviour.

Figure 7.8 shows the Mohr circle of total stresses plotted from the CUTC test results. As expected for saturated samples, $\phi_u = 0$. The difference observed for the specimen consolidated at 50 kPa is probably due to the fact that more saturation is achieved as the confining pressure is increased. The failure criterion proposed by Kivelo (1998), see § 2.7.1, for lime-cement columns, based on a total stress approach, shows the stabilised soil to have both a cohesion and friction component at low stresses and then the shear stress becomes constant as the normal stress is increased. However, the results obtained by that author were on partially saturated samples.

![Figure 7.8. Mohr circle of total stresses.](image)

The direct simple shear test results showed the total angle of friction of the cement-stabilised peat to decrease as the vertical applied pressure increases (it must be noted that the specimens tested were not saturated). At low stresses, the total angle of friction was found to be about 37° compared to 23.7° yielded by the best-fit line for all the test results—see Figure 4.39. These results suggest that the total angle of friction of the partially saturated cement-stabilised peat specimens decreases with increasing vertical applied pressure. This is in conformity with findings of Kivelo (1998) on lime-cement stabilised organic clay. The results obtained on partially saturated specimens are more likely to simulate the behaviour of the stabilised soil in the field.

The stiffness of the cement-stabilised peat was found to be strongly non-linear and was dependent on both strain and confining stress level. The highest $E_u/c_u$ ratio was obtained
for the specimen consolidated at 50 kPa. The increase in confining pressure resulted in a progressive disruption of the cementation components, which in turn resulted in a decrease in the stiffness of the soil. The stiffness moduli inferred from the CUTC tests with internal strain measurements were significantly higher than those obtained from empirical methods relating the stiffness modulus to the unconfined compressive strength (i.e. $E_{50} = 100c_u$). This significant difference is believed to be due to the method used for measuring strains as explained in § 7.4.1.

An attempt to define yielding for the cement-stabilised peat was performed using the definition given by Malandarki (1994). This author suggested that yielding of bonded soils is associated with a loss in tangential stiffness. Figure 7.9 shows a plot of the tangential stiffness versus axial strain in a log-log scale. It can be seen that two yield points can be identified for the cement-stabilised peat. The first yield point occurring at an axial strain of 0.02% showing the change in stiffness of the soil from stiff to less stiff behaviour and the second yield point, occurring at about 0.3% showing clearly the initiation of a major drop in the stiffness of the soil sample. For axial strains higher than 1%, a third yield point can be seen and could be associated with a final yield as described by Malandarki & Toll (1996).

![Figure 7.9. $E_{\text{tan}}$ versus axial strain. CUTC tests on cement-stabilised peat. (log-log scale).](image)

Failure in the CDTC tests carried out on cement-stabilised peat was not reached within the axial strain applied. The use of GDS controllers was found to limit the margin of axial displacement that can be applied (i.e. the volume controller was found to reach saturation at an axial strain of about 6%). The deviator stress was still increasing when the tests were stopped.
The stress-strain curves obtained from CDTC tests were in the main mostly independent of the confining pressure. The curves were almost identical for the range of confining pressures applied—see Figure 4.33. One important aspect of the stress-strain behaviour of the cement-stabilised peat is the effect of the confining pressure on the post-peak strength behaviour.

Two tests carried out at large strains showed the post-yield behaviour of the cement-stabilised peat to be different depending on the confining pressure applied. For the specimen consolidated at low confining pressure, the deviator stress remained constant post-peak whereas for the specimen consolidated at 400 kPa, the deviator remained constant after the first peak strength and then experienced a sudden increase in strength with increasing axial strain, reaching a second peak strength at larger strain. Similar behaviour was reported by Sangrey (1972) for naturally cemented soils. The author suggested that at low stresses, the cementation dominates and little friction is left when the bonds are destroyed, whereas high confining pressures enable friction to build up after the cementation bonds are destroyed so that the stress-strain curve continues to increase beyond the first peak strength. This could possibly explain the behaviour observed at large strains for the cement-stabilised peat.

The cement-stabilised peat was found to be characterised by a very low Poisson’s ratio ($v' = 0.065$). As the confining pressure was increased, the specimens were found to remain mainly cylindrical with little lateral deformation. At confining pressure of 200 and 300 kPa, the volumetric strain-axial strain plot fell along the one-dimensional compression line suggesting that compression of the cement-stabilised peat in drained triaxial conditions is mainly accompanied by a decrease in height. Chapuis et al (1984) on a suite of triaxial drained tests carried out on artificially bonded clay showed that the specimens remained cylindrical during the test and their diameter was practically unchanged.

**7.4.6 Effect of preloading**

The preloading of the stabilised peat immediately after mixing appears to be a critical factor in terms of the mechanical behaviour achieved. As shown in this project and also reported in Appendix I, the strength achieved for the stabilised peat is significantly boosted when the soil is preloaded. This has been confirmed throughout this project by comparing
the mechanical behaviour of the preloaded cement-stabilised peat specimen, the laboratory-stored chamber specimen and those recovered from the chamber after completion of the test. The main points observed are:

(i) No significant increase in strength occurs for the specimens not preloaded, after 7 days curing time. The strength achieved at 90 days was approximately equal to that achieved at 7 days, whereas, for the preloaded specimens, the strength achieved at 90 days was almost fivefold higher,

(ii) The yield stress determined from oedometer tests is higher for the specimens preloaded, however, the behaviour post-yield was similar for both preloaded and non-preloaded specimens- see Figure 7.5. The coefficient of secondary compression is higher for the specimens not preloaded,

(iii) Preloaded specimens yield lower permeability than those not preloaded,

(iv) Lower stiffness is achieved for the specimens non-preloaded.

Figure 7.10 is illustrative of the effect of preloading on the strength achieved.

Figure 7.10. Unconfined compression test on both preloaded and non-loaded cement-stabilised peat specimens cured at 90 days.

These results suggest that the stress state acting during the curing process of the cement-stabilised peat plays a fundamental role in the mechanical behaviour of the stabilised peat.
It is therefore recommended for stabilisation work in the field to preload the stabilised peat immediately after mixing.

7.5 DISCUSSION OF THE TESTING CHAMBER TEST RESULTS

7.5.1 Performance of the instrumentation used

As stated previously, the performance of the settlement plates and thermocouples used was generally satisfactory. Some heave was measured at settlement plates S3, S6 and S1 during loading stages. A possible explanation for this behaviour is the fact that the magnet used for locating the settlement plate is only attached to one side of the settlement plate as shown in Figure 5.8. Therefore, any tilting of the plate during loading would result in measuring heave instead of settlement.

The performance of the pore pressure gauges used was very poor. Some possible reasons for this behaviour could be due to factors such as electrical noise, temperature and also the partial saturation of the stabilised material.

The ambient temperature prevailing in the testing chamber was monitored throughout the test as shown in Figure 7.11.

![Figure 7.11. Variation of temperature in the testing chamber with time.](image)

The temperature in the testing chamber is shown to fluctuate between +15° and +20°. According to the specifications provided by the manufacturer, the variation in the output reading due to temperature changes is of the order of ± 0.2%/°C, which in this case would
give a maximum variation of 1%. Therefore, fluctuation of the temperature in the testing chamber is unlikely to affect output readings to the extent found experimentally.

Another possibility could be due to the partial saturation of the stabilised material. However, similar behaviour was recorded in the peat, which was expected to be fully saturated. Therefore, the partial saturation of the stabilised material could have possibly affected the observed trend of excess pore pressure during the test but the similar trend observed in the peat would suggest that this is not a determinant factor.

The electrical noise in the laboratory could also have a possible effect on the output readings. The laboratory where the testing chamber test has been carried out possesses many electrical equipment and as reported by Gavin (1998), this can induce voltages to the system which will in turn affect the output readings. This factor was not investigated in this project.

Regarding the earth pressure cells, only one earth pressure cell out of the three used performed during the test. However, readings obtained from this earth pressure cell were not consistent during the unload-reload stages.

7.5.2 Homogeneity of the stabilised material in the testing chamber

From the strength profile obtained from the CPTU test, the strength of the column was found to be higher at the top third layer (i.e. $c_u = 100$ kPa) than at the bottom third layer which had average undrained shear strength of 50 kPa. The strength of the stabilised mass was also found to be non-homogeneous. The strength measured by means of a hand vane at the top surface was half that measured at a depth of 0.20m. As reported in chapter 6, the stabilised mass was formed into 2 layers of 30 cm and 20 cm and this may have resulted in a different strength achieved for both layers. The design strength for the stabilised mass (i.e. 100 kPa) was reached whereas that for the stabilised column (i.e. 150 kPa) was not reached. This is believed to be due to the delay in preloading the stabilised material.

The strength with depth of the stabilised column is expected to be fairly constant, particularly if the stabilised material was formed under laboratory conditions. The quantity of binder mixed in this case is the same throughout the depth in contrast with field
conditions where this is not always achieved. Considering the large quantity of peat used at once in the laboratory pan-mixer for forming the stabilised peat, it is possible that some lumps of peat remain unstabilised during the course of mixing or that the distribution of the stabiliser would not have been even in the peat.

Preliminary unconfined compression tests carried out, see § 5.3.2, showed a scatter in the strength achieved for the stabilised peat specimens recovered from the same batch, a difference in the strength achieved of 70 kPa was measured. This scatter is most likely to be reflected in a variation of the strength with depth or even within the same cross section. Visual examination of a 100mm diameter core sample recovered from the bottom part of the stabilised column clearly showed some peat lumps distributed along the outer part of the cross section of the sample. Water content measurements varied from 330% at the centre of the core to about 520% at the outer part (compared with an average water content of 800-900 % in the peat layer). These observations confirm the variation in strength profile obtained in the stabilised material in the testing chamber. Despite using a laboratory device for mixing the peat, the measured strength of the stabilised peat material was found to vary along the same cross section and with depth.

The unconfined compression tests carried out on specimens recovered from different depths within the column and the stabilised mass recorded similar strength. A visual examination of the specimens tested showed very good distribution of the stabiliser over the entire cross section. The average shear strength measured was of the order of 70 kPa however, the stiffness modulus measured, $E_{50}$, was found to be much lower than that measured on laboratory-stored chamber specimens. The measured stiffness of the stabilised peat is more likely to be affected by the sampling procedure used. The disturbance caused results in measuring stiffness values, which do not truly represent the actual conditions.

The results obtained from the unconfined compression tests are in conflict with the variation in strength profile observed from CPTU test results and also from the vane shear tests carried out in the stabilised mass. Considering the small radius of the column (i.e. 300mm), any weak spot within the same cross section will in turn affect the strength measured by the CPTU as the latter is very sensitive to changes in strength. The strain distribution profile obtained in the testing chamber is more representative of the strength
profile obtained from the CPTU than that obtained from the unconfined compression tests. Therefore, it can be suggested that the CPTU is a better means for assessing the strength of the column and investigating weak layers within the column.

As shown in this project, the use of different mixers led to different strengths achieved with different scatter between the specimens tested. It appears that the strength achieved for the stabilised peat is unique to the mixing procedure used and provided that this procedure yields a satisfactory reproducibility, it is difficult to propose a general rule regarding the variation in the strength achieved on laboratory-mixed specimens.

7.5.3. Increase in temperature in the stabilised material

Compared to the increase in temperatures, following stabilisation, reported in the literature (Esrig 1999; Forsman 2000), the increase in temperature measured in the stabilised column was very small (i.e. +9°C). This maximum was reached within two days after mixing. After 40 days curing time, the temperature in the stabilised column decreased to reach that in the peat layer. This may suggest that the reactions in the binder have slowed down or probably stopped. The extent of the strength increase with time observed on laboratory-stored specimens seems to support this view. There was barely any increase in strength after 7 days curing time.

Campanella & Mitchell (1968) have shown that any increase in temperature of +1°C could increase the pore pressure by about 1.3% to 1.8% of the total overburden pressure. Therefore, it is important to monitor this factor in any stabilisation work as for instance, a temperature increase of +40°C would result in a rise of the pore water pressure by an average of 50%. The effect of the increase in temperature in the stabilised soil should be taken into account when analysing pore pressure readings.

7.5.4 Interaction between the stabilised column and the original peat

The measured settlement depth profile was in good agreement with the strength profile derived from the CPTU. Most of the compression in the column occurred in the bottom part, which is weaker than the top. The stabilised mass was found to experience very little compression compared to the stabilised column and the peat layer.
The compression profile in the testing chamber indicates that downdrag occurs at the top of the stabilised column. As the material in the testing chamber is loaded, most of the stresses applied are transferred to the stabilised column through arching in the stabilised mass as shown in Figure 6.27. However, if the load applied exceeds the creep load of the column, see § 2.7.1.1, then the adjacent peat layer will be loaded which will in turn compress more than the column owing to its much higher compressibility. The creep load of the column as determined from eq. 2.13 is about 18 kPa for an undrained shear strength of 100 kPa.

The finite element calculated settlements were found to be in good agreement with the experimental ones. Similar strain distribution with depth was obtained. The calculated settlements at the top of the column and at the top of the adjacent peat layer show that the equal strain theory assumption to be reasonable. However, the settlement is greater in the stabilised column than in the adjacent peat section. The effect of the concrete wall on the settlement profile for a given horizontal section in the peat layer can be clearly seen in Figure 6.22.

The experimental behaviour of the material in the testing chamber was successfully simulated using the strength parameters derived from both laboratory and CPTu tests. This indicates that the correlation $E'_{50} \approx 20c_u$ used for the stabilised material is a good representation of the actual stiffness of this material. Considering the range of strains measured in the column and also the variation of stiffness with axial strain obtained from standard oedometer tests performed on stabilised peat samples recovered from the testing chamber, this correlation gives a good estimation of the stiffness of the stabilised material within the working strains range recorded.

The permeability values assumed in the model along with the boundary drainage conditions successfully simulated the consolidation behaviour of both the stabilised peat and the original peat. The settlement-time curves were matched with good confidence. The estimated coefficient of consolidation $c_v$ (i.e. $c_v = 1.27 \times 10^{-6}$ m$^2$/s) almost coincided with those obtained from Asaoka’s graphical method. However, it was higher than that measured in the laboratory on recovered cement-stabilised specimens (i.e. $c_v = 2.5 \times 10^{-7}$ m$^2$/s for a load of 25 kPa). Nevertheless, the calculated value simulated the measured settlement-time curve with good confidence. It is possible that the root time method used for deriving $c_v$ is not appropriate for cement-stabilised peat.
The coincidence of the calculated $c_v$ value and that derived from Asaoka's method indicates that this graphical method could be used successfully for deriving the coefficient of consolidation as well as estimating the final settlement occurring in the stabilised column provided that the consolidation of the column can be considered to be one-dimensional. The calculated $c_v$ value is higher than that of the original peat as determined in the laboratory (i.e. $c_v = 2.79 \times 10^{-7} \text{ m}^2/\text{s}$ for a load of 10 kPa). This comparison highlights two points: (1) the drainage in the column is mainly one-dimensional. Therefore, the expression of $c_v$ used in eq. 6.1 is justified, (2) the acceleration of the rate of settlement in the stabilised column reported in case histories is more likely to be due to higher $c_v$ values in the column rather than to a higher permeability of the stabilised material; the permeability values measured on specimens recovered from the chamber and on the original peat seem to support this view.

The unload-reload loop of the material in the testing chamber is characterised by the same slope measured in unloading and reloading. The unload/reload stages show that the stabilised material behaves in accordance with the recorded unload/reload behaviour in the laboratory. As no unload/reload stages were performed in the triaxial tests for the stabilised material, the assumption that $E'_{ur} = 2.5 E_{50}$ was made, regarding the unloading/reloading stiffness. The simulation of the unload/reload stages in the testing chamber was found to be satisfactory as can be seen in Figure 6.21. Therefore, the assumption made above (i.e. $E'_{ur} = 2.5 E_{50}$) for the unload/reload stages appears to be appropriate.

The calculated effective stress distribution in the column showed a decrease with depth. This decrease in effective stresses was more pronounced as the applied load increased. The magnitude of arching occurring at the top of the column depends on the ratio $E_{col}/E_{peat}$ and also on the creep load of the column. The higher ratio $E_{col}/E_{peat}$ ($\approx 6$) at the top third of the column results in a transfer of most of the load applied, within the creep load of the column, to the top of the column.

The distribution of calculated effective stresses in the peat layer is fairly constant within a radial distance of 0.6m from the chamber centreline. However, the effective stresses increase close to the concrete wall- see Figure 6.25. This is due to wall friction and is illustrative of the significant influence of this boundary on the testing chamber test.
The magnitude of the calculated stress concentration ratio (i.e. ratio of the average effective stress in the column to the effective stress in the adjacent peat section) was found to vary between 2.5 and 3.5 for the applied loads during the test. This ratio varied between 2.5 and 3 for the applied loads of 40 and 50 kPa. This calculated stress concentration ratio is in agreement with reported concentration ratios for stone columns. Alamguir et al. (1996) quoted values of stress concentration ratio varying between 3 and 6 for different types of column-reinforced composite ground. Lin & Wong (1999) reported a stress concentration ratio of about 5.5 for cement columns formed in a soft clay layer.

The predicted variation of the stress concentration factor $m$, defined as the ratio of the average effective stress in the column to the external applied stress, with depth is shown in Figure 7.12. This figure shows the stress concentration factor to increase over the stabilised mass, reaching a maximum at the top of the column. Then the stress concentration factor decreases with the depth in the stabilised column. The maximum value for the stress concentration factor was found to be between 1.4 and 1.6 for the different applied loads.

![Stress concentration factor m vs depth](image)

Figure 7.12. Variation of the predicted stress concentration factor with depth.

Assuming the settlements of the column and the adjacent peat section to be the same throughout the length of the columns, the following equation can be derived:

$$\sigma_0 A = \sigma_{\text{col}} A_{\text{col}} + \sigma_{\text{soil}} (A - A_{\text{col}})$$

(7.1)

$\sigma$ designates the stress, $A_{\text{col}}$ the area of the stabilised column and $A$ the area of the testing chamber.
Assuming a stress concentration factor \( m = \frac{\sigma_{col}}{\sigma_0} = 1.4 \) and for a stiffness ratio \( E_{col}/E_{soil} \approx 6 \), the column is found to take about 17% of the total load applied for an area replacement of 12%. This low proportion of the load taken by the column is clearly due to the significant boundary effects.

From the finite element back analysis of the calibration chamber test, it can be seen that the use of the cement column method reduces drastically the settlement of the untreated soil and also allows faster proceeding of the settlements—see Figure 7.13. The settlement of the treated soil is reduced by an average factor of 0.28.

Figure 7.13. Calculated and experimental settlements-time curves. (Only shown for applied loads of 20 and 40 kPa).

7.6 SUMMARY

A discussion of the experimental results obtained in this project was presented.

The chemical properties of the peat tested are believed to be the determinant factor for the stabilisation of this soil. For peats having similar geotechnical properties, results of the stabilisation with the same binder can be different. The stabilisation of organic soils appears to be best achieved with cement and slag based stabilisers. The action of the cement stabiliser was shown to be by filling the voids and creating tight meshes, which in
turn reduce the volume of voids of the peat. However, no interaction between the organic matter and the hydrated cementation products was observed. A mechanism for cement stabilisation of peat was proposed from the observation of its microstructure. The gain in strength with time for the stabilised peat could be represented by a logarithmic function of the type: 

$$q_{u,t} = A \log \left( \frac{t}{7} \right) + q_{u,7}.$$ 

The stress state during the curing process of the cement-stabilised peat plays a fundamental role in the mechanical behaviour achieved. The strength, the creep behaviour and the permeability are shown to be greatly affected by the initial preload applied to the specimen.

Pre-yield, the behaviour of the cement-stabilised in compression was typical of a stiff soil. Post-yield, the cement-stabilised specimens reached the same state regardless of the curing time and strength achieved. Less creep settlement occurs in the stabilised peat compared with the original peat. From oedometer test results, a compression mechanism for cement-stabilised peat was proposed. The permeability of the cement-stabilised peat is shown to be of the same magnitude or lower, depending on the curing stress, than that of the original peat. The behaviour of the stabilised peat under triaxial conditions was similar to that of stiff soils. The stiffness was strongly non-linear and the failure envelope curved. The correlation, $E_{50} = 100.c_u$, often used in design does not represent the recorded behaviour at small strains; however, it gives a good estimation of the stiffness of the stabilised material for strains higher than 1%.

The behaviour of the stabilised soil structure in the testing chamber was successfully modelled using a finite element model and with soil parameters determined from laboratory tests. The finite element model highlighted the significant boundary effects on the performance of the stabilised soil structure. The rate of settlement of the treated peat was increased despite the fact that the permeability values used for both the original and stabilised were the same. This is believed to be due to the higher $c_v$ value of the stabilised material. The proportion of the total load taken by the column was about 17% for an area replacement ratio of 12%. The settlement of the treated peat was about 0.28 that of the original peat. Overall, the stabilisation method is shown to be applicable for soil stabilisation of organic soils and is an efficient means of reducing the settlement of the treated peat and also accelerating the rate of consolidation.
CHAPTER 8

CONCLUSIONS
8. CONCLUSIONS

The objectives of this thesis were to study the behaviour of the stabilised peat. This study included a literature review of available information relating to the chemical reactions that take place when certain binders are added to the peat, laboratory testing of a variety of binders when added to peats from two selected sites and a comprehensive laboratory investigation of the behaviour of a cement-stabilised peat from one of the selected sites. A stabilised soil structure was constructed and tested under load in a large-scale testing chamber. The performance of the stabilised soil structure was modelled using finite element methods with soil parameters based on laboratory derived parameters and the computed deformations were compared with those recorded. The following conclusions were derived from this research:

8.1. LABORATORY INVESTIGATION OF THE STABILISED PEAT

- The stabilisation of peat is best achieved with cement or blast furnace slag based stabilisers provided sufficient stabiliser is used. The laboratory tests indicate that a minimum amount of binder of 150 kg/m³ is required to achieve a significant increase in strength. The addition of gypsum to the slag resulted in a significant increase in strength for one peat but not for another. The disparity of the strength increase with the addition of gypsum with slag has been recorded by others (Huang 1997), who considered that it could be assessed by measuring the capacity of the soil to absorb calcium ions.

- Combinations of pfa and lime mixtures are not successful for the stabilisation of peat due to the low clay content of the soil.

- The chemical properties, the stage of decomposition and the type of organic matter are the fundamental factors for the stabilisation of peat. The results obtained in this project showed that the strength achieved for the cement-stabilised peat decreases with the increase in the degree of decomposition of the peat.
• The gain in strength with time for the cement-stabilised peat can be represented by a logarithmic function, similar to that reported by Brandl (1981), of the type \( q_{u,t} = A \ln \left( \frac{t}{7} \right) + q_{u,7} \). About 70% of the strength achieved after 1 year curing time occurs within 90 days curing time, therefore, it is more beneficial to design stabilisation work in the field for a curing age of 90 days.

• Inspection of the stabilised peat under electron microscope indicated that the improvement in the geotechnical properties of the stabilised peat is achieved through filling the large voids, which are characteristic of this type of soil. Hydrated cementation products form in the peat pores, creating tight meshes growing in the voids and this results in a decrease of the volume of the voids. The addition of gypsum to the blast furnace slag results in the formation of ettringite which are needle shaped-like crystals growing in the large voids, reaching lengths of the order of \( \mu \text{m} \), thus filling efficiently the volume of voids of the peat. No interaction is observed between the hydrated cementation products and the organic matter.

• The organic acids present in the peat pore water delay the hydration of cement mainly by consuming calcium ions released during the course of hydration of the cement. The X-ray diffraction patterns observed on both cement mixed with laboratory and peat pore water showed that more calcium hydroxide is released when laboratory water is used. Furthermore, the higher buffering capacity of the peat pore water that of the laboratory water indicates that more calcium ions are likely to be neutralised by the peat pore water.

• The laboratory test results indicate that preloading the stabilised peat during the curing process plays a fundamental role in the mechanical behaviour achieved. The strength of the preloaded specimens was approximately fivefold higher than those not preloaded. However, the effect of preloading is very sensitive to the time of application after mixing of the soil. It is therefore recommended to preload the peat immediately after mixing.

• Local strain measurements on stabilised peat showed the stress-strain relationship to be non-linear; the stabilised peat is very stiff at small strains. A relationship \( E_{50} = 100c_u \)
was obtained from the unconfined compression tests. This correlation does not represent the recorded behaviour observed in triaxial tests due to the method of measuring the strains (i.e. external measurements), however it provides a good estimation of the measured stiffness of the stabilised peat for axial strains higher than 1%.

- The failure strain for the stabilised peat depends on the strength achieved and the type of binder used. For undrained shear strengths of less than 100 kPa, the failure strain can be as high as 15% whereas for undrained shear strengths exceeding 200 kPa, the average failure strain is about 2.5%. This failure strain is much lower that that of the original peat.

- The yield stress of the cement-stabilised peat varies with curing time. Pre-yield, the behaviour of the stabilised peat is similar to that of stiff soils. Post-yield, all the "ncl" of the specimens tested have a similar value of $C_c$ regardless of the curing time and strength achieved. The normally consolidated part of the $e$-$\log \sigma_v$ curves was linear and the unload-reload loop was characterised by the same slope. The cementation components were shown not to contribute to the overall resistance to deformation after yield. The post-yield behaviour of the stabilised peat does not exhibit the collapse type feature of the naturally cemented soils tested in the oedometer.

- The cement-stabilised peat is characterised by higher values of the coefficient of consolidation $c_v$ than those of the peat due to lower values of $m_v$. The coefficient of secondary compression $C_o$ is reduced by a factor of 10 compared to the original peat.

- The permeability of the cement-stabilised peat is of the same order or lower than that of the original peat, depending on the stress state during curing. Preloaded specimens yield a lower permeability than that of the original peat. A significant decrease in permeability occurs post-yield. The assumption of higher permeability of the stabilised material than that of the original soil is not verified for cement-stabilised peat.

- Results obtained from triaxial tests on saturated stabilised peat specimens showed the failure envelope of the stabilised peat to be curved. Three main stress regions were distinguished: (1) low stress region characterised by a high build up of pore pressures,
(2) an intermediate stress region where the peak strength is not affected by the confining pressure. The failure envelope is horizontal and (3) a normally consolidated stress region characterised by a breakdown of the cementation bonds. Therefore, the peat develops a frictional behaviour. The stiffness of the stabilised peat is strongly nonlinear and is both affected by strain and confining stress level. The observed compression of the stabilised peat in CDTC tests is mainly one-dimensional.

- The failure criterion proposed by Kivelo (1998) for lime-cement stabilised soil was not verified as the specimens tested in the triaxial were saturated. However, results of the direct simple shear test on partially saturated cement-stabilised peat specimens indicated that the angle of friction decreased as the vertical applied stress increases and this is in conformity with the findings of that author.

8.2 DEVELOPMENT OF THE TESTING CHAMBER

- The method of peat block sampling used in this project was very satisfactory.

- A new device for measuring settlements was designed. The location of the settlement plates, embedded in the soil, can be determined within ±1mm.

- The parametric finite element calculation showed that the settlements of the treated peat were best reduced with a column spacing of less than four times the column diameter and that the stiffness of the stabilised material affected the final calculated settlements. The stabilised mass was shown to play a great role in the distribution of stresses to the column and the peat layer.

- A large laboratory mixer was used for forming the stabilised material used in the testing chamber. Comparable strengths with those obtained using the laboratory reference procedure were achieved. However, the strength achieved showed some scatter, reaching 70 kPa, among the specimens tested.
8.3. THE LARGE-SCALE TESTING CHAMBER TEST

- A stabilised soil structure composed of a stabilised column and a stabilised mass was successfully formed in the testing chamber.

- The settlement device and thermocouples used performed well throughout the test, whereas the performances of the pore pressure gauges and earth pressure cells were poor.

- The maximum increase in temperature in the column following the hydration of cement was of the order of $+9^\circ C$ and was reached within 2 days after mixing. Thereafter, the temperature in the column started cooling off reaching, eventually, that of the peat.

- The strain distribution profile with depth in the chamber was in good agreement with the strength profile derived from the CPTU results. Most of the compression occurred in the weak part of the column. The assumption of equal strain conditions between the stabilised column and the adjacent peat layer was verified at the top of the column. However, settlements in the column and in the peat were not uniform due to boundary effects.

- The experimental settlement profile showed that more compression occurred in the peat layer between (1.00m-1.50m) than in the adjacent stabilised column, whereas for the remaining depth, more compression occurs in the stabilised than in the adjacent peat layer. This indicates that downdrag occurs at the top third layer of the column. Little compression occurs in the stabilised mass.

- The highest average compression measured for load increments of 40 and 50 kPa was 7.5% and 10.5% for the column ($z = 1.00m - 2.00m$), 1.6% and 1.4% for the stabilised mass and 7.6% and 8.8% for the peat ($z = 0.50m - 1.00m$).

- The unload-reload loop carried out was characterised by the same slope in unloading and reloading. This behaviour was in agreement with the unload-reload behaviour observed on laboratory specimens.
• In-situ tests carried out on the stabilised material in the chamber showed the strength to vary with depth and over the same cross-section. Visual inspection of the stabilised material recovered from the column revealed the presence of some unstabilised peat lumps in the column. However, no general rule can be given for estimating this variation in strength.

• The strain distribution in the chamber is more representative of the strength profile obtained from the CPTU test than that obtained from unconfined compression tests. The CPTU test is more suited for assessing weak layers in the column.

• The stiffness measured on the chamber specimens tested in unconfined compression does not represent the actual stiffness of the material in the chamber because of disturbance caused by sampling.

• The finite element model has successfully simulated the behaviour of the soil in the testing chamber using soil parameters determined from laboratory and in-situ tests. The drained secant modulus derived from the correlation \( E'_{50} = 20.c_u \) yields a good estimation of the stiffness of the stabilised material in the chamber, within the working strain range observed.

• The experimental settlement-time curves were successfully matched with the values of permeability used. The increase in the rate of settlement observed is due to higher value of the coefficient of consolidation \( c_v \) in the stabilised material than in the adjacent peat layer rather than to a higher permeability of the stabilised material.

• The rate of consolidation in the stabilised column was determined using Asaoka's graphical method. The coincidence of the value of \( c_v \) determined by this means and that back calculated indicates that this method could be used as a complement or substitute to the pore pressure readings provided that the consolidation in the column is one-dimensional.

• The finite element modelling of the unload-reload behaviour of the material in the testing chamber was found to be satisfactory.
• The calculated stress concentration ratio between the column and the peat varies between 2.5 and 3.5 for the range of loads applied. The stress concentration ratio is highest at the top of the stabilised column.

• The simulation of the testing chamber test showed that the results obtained are significantly influenced by the boundary conditions. The proportion of the total load taken by the column was about 17% for an area replacement ratio of 12%.

• A comparison of the finite element modelling of the testing chamber test for both cases where the peat is stabilised and non-stabilised showed that the calculated final settlements are reduced by an average ratio of 0.28 when the peat is stabilised with a soil structure (i.e. stabilised column and mass).

8.4. PROPOSALS FOR FUTURE RESEARCH

Findings of this study showed that it is not the geotechnical properties, such as the water and organic content, of the peat that influence the extent of stabilisation of similar peats with different binders. This indicates that the chemical properties of the peat are the most important factor. Therefore, it is necessary to investigate the main chemical properties of the peat responsible for the difference in strength obtained in similar peats when using the same binder. It would be useful if these chemical properties could be related to properties such as the degree of decomposition of the peat or its type as these are easily assessed in the laboratory.

The triaxial tests carried out in this study were all on saturated specimens. As a result, the failure criterion proposed by Kivelo (1998) for lime-cement columns could not be verified, as this author carried out the tests on partially saturated specimens, which simulate better the actual conditions in the ground. Therefore, the validity of this failure criterion should be verified on partially saturated stabilised peat specimens.

It is not possible, at present, to give an indication of the field correction factor to be applied to the strength measured on laboratory-mixed specimens. It is important to assess how could the experience acquired on laboratory-mixed specimens be transferred to the field;
what is the fraction of strength measured on laboratory-mixed specimens that could be achieved in the field? and what is the effect of mixing energy and temperature on the strength achieved in the field? the use of two different laboratory mixers showed that different strengths could be achieved with different scatter. Thus, it seems necessary to develop a laboratory-mixing procedure, which simulates closely the mixing field condition and also improves the reproducibility of the stabilised specimens.

The strength of the cement-stabilised peat was found to increase with time up to 1-year curing time. In the mixing procedure used, the stabilised specimens were stored in laboratory water, however in the field, the high acidity of the groundwater is more likely to affect the long-term strength of the stabilised peat. The humic acids contained in the ground water react with the hydrated cementation products to form salts and if these latter are soluble they will wash away and this will result in a decrease in the strength. Therefore, the long-term strength should be investigated on specimens stored in acidic ground water.
APPENDIX I

REVIEW OF METHODS OF FORMING THE STABILISED SOIL
I. REVIEW OF METHODS OF FORMING THE STABILISED SOIL

The design of soil stabilisation projects is mainly based on the results obtained on laboratory-mixed specimens. It has been always sought to develop mixing methods in the laboratory that will reproduce, to the closest possible, the mixing procedure in the field. According to Pousette et al. (1999) there is no existing standard method, at present, for manufacturing stabilised peat specimens in the laboratory. Each soil has its own specific characteristics thus requires a suitable mixing procedure. den Haan (2000) claimed that there exists two main mixing procedures currently in use: (1) a simple procedure, consisting essentially of homogenising the soil, mixing it with the stabiliser, placing the mix in a mould and compacting it, curing the sample and finally preparing it for testing, (2) the advanced method which simulates as closely as possible the field mixing. To our knowledge, few studies were reported regarding the advanced method.

Edstam & Carlsten (1999) have developed a new procedure for laboratory mixing of soil samples where a new mixing tool has been specifically designed for this purpose. This latter resembles that used in the field. Results obtained were very encouraging as the scatter in the values of strength achieved was considerably reduced to ± 10%, compared to a scatter of ± 20% to 30% for the conventional methods used, making this method more reproducible and reliable.

During the course of the preparation of the stabilised soil samples in the laboratory several factors are susceptible to influence the results obtained. Pousette et al. (1999) have enumerated the main common factors to be considered in the laboratory mixing of soil samples and are mixing tool, mixing time, size of the sample, storing conditions and applied load during curing. The factors enumerated above have attracted the attention of several researchers (Ahnberg & Holm 1999; Ahnberg 1999; den Haan 2000). These latter have investigated, in particular, the effect of the mixing procedure used on the strength increase achieved of stabilised organic soil samples. Reviewing the different mixing procedures reported in the literature, there seems to be a consensus among the authors on some points such as: homogenisation of the peat before adding the stabiliser, free access to water for the stabilised peat sample during curing, application of a load immediately after mixing of the
peat sample. However, if there are agreements in the form for the different methods used there still remains some differences in their implementation.

1.1 MIXING TIME

Due to the variability of the organic soil, very high for peat, it seems necessary that the peat should be homogenised to eliminate any effect of the material heterogeneity on the results obtained before being mixed with the stabiliser. Pousette et al. (1999), in their preparation procedure, have used hand stirring for 1 minute to homogenise the peat. Other preparation procedures (Ahnberg & Holm 1999; Lahtinen et al. 1999; Cortellazo & Cola 1999) reported the use of a dough-mixer for the homogenisation of the peat. No specified mixing time was reported but the peat was mixed until visually homogeneous. den Haan (2000) recommends to limit the mixing time in the case of fibrous peat to prevent the destruction of fibres present in the peat. Therefore, there seems to be an agreement on mixing the peat mechanically until visually homogeneous.

The mixing of the soil and the binder is probably the most critical phase as far as the strength increase is concerned, as this latter depends mainly on how well is the stabiliser mixed with the soil. Pousette et al. (1999) have investigated the effect of mixing time on the strength achieved by stabilised peat samples. Mixing times of 1, 2 and 5 minutes were used. 1 and 5 minutes mixing time were found to give lower value of the unconfined compressive strength than a mixing time of 2 minutes. The authors claimed that 1min. mixing time is probably not enough to achieve a homogeneous sample whereas a mixing time of 5min. can be too much and will lead to the breakdown of peat fibres thus a lower strength is achieved. The authors suggested that the mixing time should be at least of two minutes but longer mixing times could influence the stabilisation effect. Hayashi & Nishikawa (1999), in a similar study, investigated the effect of varying mixing time on the strength achieved of a peat from the Hokkaido area, mixed with three cement-type stabilisers. Mixing times of 2 and 10 minutes were used. The findings of the authors were quiet contradictory to those of Pousette et al. (1999). The unconfined compressive strength was found to increase with increasing mixing time regardless of the type of stabiliser used. Furthermore, less scatter in the strength achieved was observed for longer mixing times. But for this latter no indication on the degree of
humification or fibre content of the peat used was given. Figure I.1 shows results obtained by Hayashi & Nishikawa (1999).

![Figure I.1. Variation of compressive strength with mixing time. Hayashi & Nishikawa (1999).](image)

### 1.2. SPECIMEN SIZE

Different sample sizes are used for the storage of the stabilised peat samples. Pousette et al. (1999) found that 50mm diameter samples give higher spreading on the unconfined compressive strength than for 68mm diameter samples. Ahnberg (1999) showed that the unconfined compressive strength decreases with increasing sample diameter. For a mixture of cement-lime (80:20), the unconfined compressive strength achieved for a 50mm diameter samples was almost double that of a 100mm diameter samples. The mixing procedure circulated within “EuroSoilStab” recommends the use of 68mm diameter samples for peat. It should be noted that in this latter mixing procedure no compaction is specified for the stabilised peat. In all procedures reviewed, stabilised peat samples are stored in tubes having free access to water at both ends.

### 1.3. PRELOADING

The construction procedure in the field shows that usually a 1m high fill layer is laid out on the top of the stabilised columns, which represents a load of about 18 kPa. To simulate this condition in the laboratory, the stabilised peat, in the laboratory, is usually cured under a
surcharge. Pousette et al. (1999) found an increase of 85% in the strength achieved of stabilised peat as the curing load increased from 10 to 40 kPa. Similar findings were reported by Ahnberg (1999). The strength achieved for stabilised peat samples cured at a load of 18 kPa almost trebled compared with samples not loaded. However, this latter author stressed the fact that stabilised samples should be loaded as soon as possible after mixing as significant decrease in the unconfined compressive strength achieved was observed when the application of the curing load was delayed by as little as 4 hours after mixing. This decrease was more pronounced as the cement content in the stabiliser was higher. This could be explained by the fact that when the load is applied the cement would have been partially set and therefore little consolidation occurs in the stabilised soil. Figure 1.2 shows results obtained by Ahnberg (1999).

![Figure 1.2. Variation of the unconfined compressive strength with the curing load. Ahnberg (1999).](image)

### 1.4 CURING TEMPERATURE

The ambient temperature is also a factor that should be considered during the curing of stabilised peat samples. The current average ground water temperature in Ireland is about 8 to 10°C. Therefore in order to simulate field conditions; the stabilised peat samples should be stored at the same temperature as in the ground water. But as pointed out by den Haan (2000), the soil is generally a good insulator and temperatures higher than the ground water
temperature are likely to occur during curing mainly due to the heat of hydration released by the stabiliser when it is mixed with the soil. Considerable increase in temperature is more likely to occur when using lime-based stabilisers. According to den Haan (2000) laboratory measurements showed temperatures up to 70°C for lime stabilisers and 25°C for cement. Ahnberg & Holm (1999) investigated the effect of the curing temperature on the strength increase of stabilised peat. The authors found that the increase in curing temperature had negative effect on the strength achieved by cement-lime stabilised peat and suggested that high curing temperatures should preferably be avoided for stabilisation of peat. Figure 1.3 shows the influence of curing temperature on the strength increase of stabilised peat as reported by Ahnberg & Holm (1999).

![Figure 1.3](image)

Figure 1.3. Variation of the unconfined compressive strength with curing temperature. Ahnberg & Holm (1999).
APPENDIX II

REVIEW ON PREVIOUS FINITE ELEMENT STUDIES ON STABILISED COLUMNS
II. REVIEW ON PREVIOUS FINITE ELEMENT STUDIES ON STABILISED COLUMNS

II.1 LITERATURE REVIEW

Finite element modelling of the interaction between stabilised columns and the adjacent original soil has been used by several researchers (Alamguir et al. 1996; Baker et al. 1997; Kivelo 1998; den Haan et al. 2000).

Alamguir et al. (1996) developed a theoretical approach, which was compared with results obtained from the CRISP finite element analysis model. The method developed was based on the “free strain” conditions (i.e. different deformation between the stabilised column and the surrounding original soil). The interaction column/original soil was modelled using the concept of “unit cell” which consists of the column and the surrounding soil within a column zone influence. The column and the soil were defined by the modulus of elasticity $E$ and the Poisson’s ratio $v$. The displacement of the columns was assumed to remain the same over its area whereas the displacement of the surrounding soil increases from the column-soil interface towards the outside boundary of the unit cell as shown in Figure II.1.

![Figure II.1. Assumed mode of deformation around a column. (Alamguir et al. 1996).](image)

Results obtained by the theoretical method proposed by the authors was found to agree very well with results obtained from the finite element analysis. Furthermore, the effect of column spacing, defined as $b/a$ – See Figure II.1, and modular ratio (i.e. $\frac{E_{column}}{E_{soil}}$) was found to be
significant on the load sharing between the column and the soil and also on the settlement of the ground. However, Poisson's ratio was found to have little influence on the interaction column/surrounding soil. The effect of column spacing and modular ratio on both stress concentration ratio $n_c$ (ratio of the stress in the column to the stress in the soil) and settlement ratio $\beta$ (ratio of the settlement of the treated ground to the settlement of the untreated ground) is shown in Figures II.2 and II.3.

Alamguir et al. (1996) observed that for very close spacing ($1 < n < 1.25$) the settlements are almost the same for the treated and untreated ground, whereas, for higher spacing ($n > 10$) the column and the surrounding soil settle independently suggesting that the column acts individually in this case. The settlement ratio $\beta$ was found to decrease with the increase of the modular ratio.

![Figure II.2](image1.png)

**Figure II.2.** Effect of column spacing and modular ratio on the stress concentration ratio. (After Alamguir et al. 1996).

![Figure II.3](image2.png)

**Figure II.3.** Effect of column spacing and modular ratio on the settlement ratio $\beta$. (After Alamguir et al. 1996).
Baker et al. (1997) used the Plaxis finite element model to carry out a parametric study to investigate the interaction between lime-cement columns and the surrounding soft clay. The numerical modelling was carried out using the principle of “unit cell” as used by Alamguir et al. (1996). The behaviour of both the column and the soft clay was assumed to be elastoplastic and the applied failure criterion was that of Mohr-coulomb with drained conditions. The main factors investigated were the roughness ratio of the interface column/clay and the column stiffness. The former factor is proper to the Plaxis finite element model. Results of the numerical modelling showed that: (1) the magnitude of stresses carried by the column is affected by the roughness ratio along the periphery of the column. The higher the roughness ratio the higher is the stress carried by the column, (2) drastic reduction of deformations in the column with increasing quality of the column. Figure II.4 shows the main findings of the numerical study.

![Figure II.4](image)

Figure II.4. (a) Effect of roughness ratio on the additional vertical stress. (b) Effect of column stiffness on the settlement. (After Baker et al. 1997).

Ilander et al. (1999) used Plaxis finite element model for modelling a test embankment carried out in Kivikko (Finland). The final height of the embankment was 5m and was constructed over a subsoil of peat, clay and silt. Calculations were made using the “unit cell” concept. The soil was modelled as linear-elastic perfectly plastic with the Mohr-Coulomb failure criterion and the parameters used for both soil and stabilised soil were obtained from standard laboratory tests. The angle of shearing resistance used for the stabilised soil was 50° for stabilised peat and 37° for the stabilised clay. The permeability of the stabilised soil was assumed to be higher than that of the original soil. A correlation $K_{y,\text{stabilised soil}} = 10. K_{y,\text{original soil}}$
seems to have been used. Results of the finite element analysis were nearly similar to those obtained from the traditional method of calculating settlements (i.e. the equal strain method) suggesting that Plaxis finite element can be used successfully, if compared to the equal strain method, for the design of soil stabilisation projects.

Edstam (1998) reported the use of Plaxis finite element model for the design of a test embankment on stabilised soil in Dömle Moss (Sweden). The original soil consisted of a layer of peat underlain by clay. The interaction between the stabilised soil and the original soil was modelled using the “unit cell” concept. The soil was modelled as a linear-elastic perfectly plastic with the Mohr-Coulomb failure criterion. The parameters used for the original soil were obtained from standard laboratory tests. Both mass stabilisation and stabilised columns were formed in the soil. For the preliminary design of the embankment test, the strength and permeability parameters used for the stabilised soil were determined from the following empirical rules: 

\[ E = 100.\text{c}_u, \quad c' = 0.2.\text{c}_u \quad \text{and} \quad k_{\text{stabilised soil}} = 100.k_{\text{soil}}. \]

The angle of shearing resistance for the stabilised soil was assumed to be 30°. The stabilised soil was modelled as linear-elastic perfectly plastic with the Mohr-Coulomb failure criterion.

II.2 SUMMARY OF THE LITERATURE REVIEW ON FINITE ELEMENT MODELLING OF SOIL STABILISATION

Review of previous experiences gained on finite element modelling of the interaction stabilised soil/original soil has showed that:

(i) The finite element modelling of the composite stabilised soil/original soil was performed using the “unit cell” concept (i.e. consists of the column and the surrounding soil within a column zone influence). The main factors affecting the interaction stabilised column/original soil are the column spacing and the quality (i.e. stiffness) of the stabilised column. A column spacing less than five times the diameter of the column is recommended for interaction to occur. Poisson’s ratio was reported to have little influence,

(ii) For preliminary design, both the original and stabilised soil are modelled as linear-elastic perfectly plastic with the Mohr-Coulomb failure criterion. The parameters used
for stabilised soil are determined from empirical correlations related to the undrained shear strength of the stabilised soil, as determined from unconfined compression tests. The main assumptions used are $E = 100c_u$, $c' = 0.2c_u$. The permeability of the stabilised soil is assumed to be higher than that of the original soil.
APPENDIX III

RESULTS OF THE UNCONFINED COMPRESSION TESTS
Raheenmore peat

Unconfined compression tests at 7 days

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Density, (t/m³)</th>
<th>( c_u ) (kPa)</th>
<th>( E_{50} ) (MPa)</th>
<th>Strain, (%)</th>
<th>Density, (t/m³)</th>
<th>( c_u ) (kPa)</th>
<th>( E_{50} ) (MPa)</th>
<th>Strain, (%)</th>
<th>Density, (t/m³)</th>
<th>( c_u ) (kPa)</th>
<th>( E_{50} ) (MPa)</th>
<th>Strain, (%)</th>
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Unconfined compression tests at 28 days

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<tr>
<th>Mixture</th>
<th>Density, (t/m³)</th>
<th>( c_u ) (kPa)</th>
<th>( E_{50} ) (MPa)</th>
<th>Strain, (%)</th>
<th>Density, (t/m³)</th>
<th>( c_u ) (kPa)</th>
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### Ballydermot peat

#### Unconfined compression tests at 7 days

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#### Unconfined compression tests at 28 days

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#### Unconfined compression tests at 90 days

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<td>Density, (t/m³)</td>
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#### Unconfined compression tests at 365 days

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<td>Density, (t/m³)</td>
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<td>Cement</td>
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<td>Blast furnace slag/gypsum</td>
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APPENDIX IV

PLAXIS FINITE ELEMENT SOIL MODELS
Plasticity is associated with the development of irreversible strains. In order to evaluate whether or not plasticity occurs in a calculation, a yield function, \( f \), is introduced as a function of stress and strain. A yield function can often be presented as a surface in principal stress space. A perfectly-plastic model is a constitutive model with a fixed yield surface, i.e. a yield surface that is fully defined by model parameters and not affected by (plastic) straining. For stress states represented by points within the yield surface, the behaviour is purely elastic and all strains are reversible.

### 3.1 ELASTIC PERFECTLY-PLASTIC BEHAVIOUR

The basic principle of elastoplasticity is that strains and strain rates are decomposed into an elastic part and a plastic part.

\[
\epsilon = \epsilon^e + \epsilon^p \\
\dot{\epsilon} = \dot{\epsilon}^e + \dot{\epsilon}^p
\]  

(3.1)

Hooke's law is used to relate the stress rates to the elastic strain rates. Substitution of Equation (3.1) into Hooke's law (2.9) leads to:

\[
\dot{\sigma}^e = D^e \dot{\epsilon}^e = D^e (\dot{\epsilon} - \dot{\epsilon}^p)
\]  

(3.2)

According to the classical theory of plasticity (Hill, 1950), plastic strain rates are proportional to the derivative of the yield function with respect to the stresses. This means that the plastic strain rates can be represented as a vector perpendicular to the yield surface. This classical form of the theory is referred to as associated plasticity. However, for Mohr-Coulomb type yield functions, the theory of associated plasticity leads to an overprediction of dilatancy. Therefore, in addition to the yield function, a plastic potential function \( g \) is introduced. The case \( g \neq f \) is denoted as non-associated plasticity.

In general, the plastic strain rates are written as:

\[
\dot{\epsilon}^p = \lambda \frac{\partial g}{\partial \sigma^i}
\]  

(3.3)

in which \( \lambda \) is the plastic multiplier. For purely elastic behaviour \( \lambda = 0 \) whereas in the case of plastic behaviour \( \lambda \) is positive.

\[
\lambda = 0 \text{ for } f < 0 \quad \text{or} \quad \frac{\partial f}{\partial \sigma^i} D^e \dot{\epsilon} \leq 0 \quad \text{(Elasticity)}
\]  

(3.4a)
\[ \lambda > 0 \quad \text{for} \quad f = 0 \quad \text{and} \quad \frac{\partial f^T}{\partial \sigma'} D^e \dot{\varepsilon} > 0 \quad \text{(Plasticity)} \quad (3.4b) \]

Figure 3.1 Basic idea of elastic perfectly plasticity model.

These equations may be used to obtain the following relationship between the effective stress rates and strain rates for elastoplasticity (Smith & Griffith, 1982; Vermeer & de Borst, 1984):

\[
\dot{\sigma}' = \left( D^e - \frac{\alpha}{d} D^e \frac{\partial g}{\partial \sigma'} \frac{\partial f^T}{\partial \sigma'} D^e \right) \dot{\varepsilon} \quad (3.5a)
\]

where:

\[
d = \frac{\partial f^T}{\partial \sigma'} D^e \frac{\partial g}{\partial \sigma'} \quad (3.5b)
\]

The parameter \( \alpha \) is used as a switch. If the material behaviour is elastic, as defined by Equation (3.4a), the value of \( \alpha \) is equal to zero, whilst for plasticity, as defined by Equation (3.4b), the value of \( \alpha \) is set to unity.

The above theory of plasticity applies straightforwardly to the Drucker-Prager model as specified in Appendix A. It also applies to the well-known Von Mises model, as described by Hill (1950). However, the theory is restricted to smooth yield surfaces and it does not incorporate an irregular yield surface as for instance involved in the well-known Tresca model and the Mohr-Coulomb model. For such irregular yield surfaces the theory of plasticity has been extended by Koiter (1960) and others to account for flow vertices:

\[
\dot{\varepsilon}^p = \lambda_1 \frac{\partial g_1}{\partial \sigma'} + \lambda_2 \frac{\partial g_2}{\partial \sigma'} + \ldots \quad (3.6)
\]
with two or more plastic potential functions. Similarly, several quasi independent yield functions \( (f_1, f_2, \ldots) \) are used to determine the magnitude of the multipliers \( (\lambda_1, \lambda_2, \ldots) \).

### 3.2 THE MOHR-COULOMB MODEL

The Mohr-Coulomb yield condition is an extension of Coulomb's friction law to general states of stress. In fact, this condition ensures that Coulomb's friction law is obeyed in any plane within a material element. The full Mohr-Coulomb yield condition can be defined by three yield functions when formulated in terms of principal stresses (see for instance Smith & Griffith, 1982):

\[
\begin{align*}
f_1 &= \frac{1}{2} |\sigma_2' - \sigma_3'| + \frac{1}{2} (\sigma_2' + \sigma_3') \sin \varphi - c \cos \varphi \geq 0 \quad (3.7a) \\
f_2 &= \frac{1}{2} |\sigma_3' - \sigma_1'| + \frac{1}{2} (\sigma_3' + \sigma_1') \sin \varphi - c \cos \varphi \geq 0 \quad (3.7b) \\
f_3 &= \frac{1}{2} |\sigma_1' - \sigma_2'| + \frac{1}{2} (\sigma_1' + \sigma_2') \sin \varphi - c \cos \varphi \geq 0 \quad (3.7c)
\end{align*}
\]

The two plastic model parameters appearing in the yield functions are the well-known friction angle \( \varphi \) and the cohesion \( c \). These yield functions together represent a hexagonal cone in principal stress space as shown in Figure 3.2. The Mohr-Coulomb model degenerates to the Tresca model when the friction angle is set to zero.

![Figure 3.2](image)

*Figure 3.2 The Mohr-Coulomb yield surface in principal stress space \( (c = 0) \).*
In addition to the yield functions, three plastic potential functions are defined for the Mohr-Coulomb model:

\[
g_1 = \frac{1}{2} |\sigma_2' - \sigma_3'| + \frac{1}{2} (\sigma_2' + \sigma_3') \sin \psi
\]

(3.8a)

\[
g_2 = \frac{1}{2} |\sigma_3' - \sigma_1'| + \frac{1}{2} (\sigma_3' + \sigma_1') \sin \psi
\]

(3.8b)

\[
g_3 = \frac{1}{2} |\sigma_1' - \sigma_2'| + \frac{1}{2} (\sigma_1' + \sigma_2') \sin \psi
\]

(3.8c)

The plastic potential functions contain a third plasticity parameter, the dilatancy angle \( \psi \). This parameter is required to model plastic volumetric strains, i.e. dilatancy as actually observed for dense soils. A discussion of all of the model parameters used in the Mohr-Coulomb model is given at the end of this section.

When implementing the Mohr-Coulomb model for general stress states, special treatment is required for the intersection of two yield surfaces. Some programs use a smooth transition from one yield surface to another, i.e. the rounding-off of the corners (see for example Smith & Griffith, 1982). In PLAXIS, however, the exact form of the full Mohr-Coulomb model is implemented, using a sharp transition from one yield surface to another. For a detailed description of the corner treatment the reader is referred to the literature (Koiter, 1960; van Langen & Vermeer, 1990).

For \( c > 0 \), the Mohr-Coulomb model allows for tension. In fact, allowable tensile stresses increase with cohesion. In reality, soil can sustain none or only very small tensile stresses. This behaviour can be included in a PLAXIS analysis by specifying a tension cut-off. In this case, Mohr circles with negative principal stresses are not allowed. The tension cut-off introduces three additional yield functions, defined as:

\[
f_4 = \sigma_1' - \sigma_t \geq 0
\]

(3.9a)

\[
f_5 = \sigma_2' - \sigma_t \geq 0
\]

(3.9b)

\[
f_6 = \sigma_3' - \sigma_t \geq 0
\]

(3.9c)

When this tension cut-off procedure is used, the allowable tensile stress, \( \sigma_t \), is taken equal to zero. For these three yield functions an associated flow rule is adopted. For stress states within the yield surface, the behaviour is elastic and obeys Hooke's law for isotropic linear elasticity, as discussed in Section 2.2. Hence, besides the plasticity parameters \( c, \varphi, \) and \( \psi \), input is required of the elastic shear modulus \( G \) and Poisson's ratio \( v \).
4 HARDENING PLASTICITY: THE SOFT SOIL MODEL

In contrast to an elastic perfectly-plastic model, the yield surface of a hardening plasticity model is not fixed in principal stress space, but it can expand due to plastic straining. In Soil Mechanics two different types of hardening exist and both of them are of interest to the users of PLAXIS, namely compression hardening and shear hardening. Compression hardening is considered in the current chapter. The next chapter describes shear hardening according to the Hard Soil model.

Compression hardening may also be referred to as compaction hardening, as here the movements of a cap-type yield surface are fully controlled by changes of density. For many engineering problems density changes of the subsoil are small, but this is not the case when normally consolidated or near-normally consolidated (lightly over-consolidated) layers of clay-type soils are being considered ($OCR < 2$). These soils are usually referred to as 'soft soils' and for this reason the former PLAXIS Cap model was renamed the Soft Soil model.

The Soft Soil model is a Cam-Clay type model, which resembles the Modified Cam-Clay model (Burland 1965, 1967). The Modified Cam-Clay model itself is also available in PLAXIS and it is described in Appendix C of this manual. For soft soils, i.e. (near-) normally consolidated silt, clay and peat, the Soft Soil model is recommended rather than the Modified Cam-Clay model, because of the mesh sensitivity of softening behaviour in the latter model and the improved behaviour in one-dimensional compression of the Soft Soil model. Some characteristics of the Soft Soil model are:

- Stress dependent stiffness (logarithmic compression behaviour).
- Distinction between primary loading and unloading / reloading.
- Memory for preconsolidation stress.
- Failure behaviour according to the Mohr-Coulomb criterion.
- No mesh-sensitivity due to softening behaviour.

4.1 ISOTROPIC STATES OF STRESS AND STRAIN ($\sigma_1' = \sigma_2' = \sigma_3'$)

In the Soft Soil model, it is assumed that there is a logarithmic relation between the volumetric strain, $\varepsilon_v$, and the mean effective stress, $p'$, which can be formulated as:

$$\varepsilon_v - \varepsilon_v^0 = - \lambda^* \ln \left( \frac{p'}{p_0} \right) \quad \text{(virgin compression)} \quad (4.1)$$
In order to maintain the validity of Expression (4.1) a minimum value of $p'$ is incorporated equal to 1 kPa. The parameter $\lambda^*$ is the modified compression index, which determines the compressibility of the material in primary loading. Note that $\lambda^*$ differs from the index $\lambda$ as used by Burland (1965). This is because Equation (4.1) involves the volumetric strain instead of the void ratio. When plotting Equation (4.1) one obtains a straight line as shown in Figure 4.1. During isotropic unloading and reloading a different line is followed, which can be formulated as:

$$\varepsilon_v^e - \varepsilon_v^{e0} = -\kappa^* \ln \left(\frac{p'}{p^0}\right)$$  \hspace{1cm} \text{(unloading and reloading)} \hspace{1cm} (4.2)

Figure 4.1 Logarithmic relation between volumetric strain and mean stress.

Again, a minimum value of $p'$ is incorporated in Equation (4.2) equal to 1 kPa. The parameter $\kappa^*$ is the modified swelling index, which determines the compressibility of the material in unloading and subsequent reloading. Note that $\kappa^*$ differs from the index $\kappa$ as used by Burland. The ratio $\lambda^*/\kappa^*$ is, however, equal to Burland’s ratio $\lambda/\kappa$. The soil response during unloading and reloading is assumed to be elastic, which explains the superscript $e$ in Equation (4.2). The elastic behaviour is described by Hooke’s law of elasticity (see Section 2.2) and Equation (4.2) implies the following linear stress dependency of the tangent bulk modulus:

$$K_{ur} = \frac{E_{ur}}{3(1 - 2v_{ur})} = \frac{p'}{\kappa^*} \hspace{1cm} (4.3)$$

The subscripts $ur$ are used to denote that the parameters relate to unloading and reloading. Note that effective parameters are considered rather than undrained soil properties, as might be suggested by the subscripts $ur$. Neither the elastic bulk
modulus, $K_{ur}$, nor the elastic Young’s modulus, $E_{ur}$, is used as an input parameter. Instead, $v_{ur}$ and $\kappa^*$ are used as input constants for the part of the model that computes the elastic strains.

An infinite number of unloading / reloading lines exist in Figure 4.1, each corresponding to a particular value of the isotropic preconsolidation stress $p_c$. The preconsolidation stress represents the largest stress level experienced by the soil. During unloading and reloading, this preconsolidation stress remains constant. In primary loading, however, the preconsolidation stress increases with the stress level, causing irreversible (plastic) volumetric strains.

4.2 YIELD FUNCTION FOR TRIAXIAL STRESS STATE ($\sigma_2' = \sigma_3'$)

For the sake of convenience, restriction is made here to triaxial loading conditions with $\sigma_2' = \sigma_3'$. For such a state of stress the yield function of the Soft Soil model is defined as:

$$ f = \bar{f} - p_c $$

(4.4)

where $\bar{f}$ is a function of the stress state $(p',q)$ and the preconsolidation stress $p_c$ is a function of plastic strain:

$$ \bar{f} = \frac{q^2}{M^2 (p' + c \cot \varphi)} + p' $$

(4.5)

$$ p_c = - p_c^0 \exp \left( \frac{\varepsilon_v^p}{\lambda^* - \kappa^*} \right) $$

(4.6)

The yield function $f$ describes an ellipse in $p'$-$q$-plane, as illustrated in Figure 4.2. The parameter $M$ in Equation (4.5) determines the height of the ellipse. The height of the ellipse is responsible for the ratio of horizontal and vertical stress in primary one-dimensional compression. As a result, the parameter $M$ determines largely the coefficient of lateral earth pressure, $K_0^{nc}$. In view of this, the value of $M$ can be chosen such that a known value of $K_0^{nc}$ is matched in primary one-dimensional compression. Such an interpretation and use of $M$ differs from the original critical state line idea, but it ensures a proper matching of $K_0^{nc}$.

The tops of all ellipses are located on a line with inclination $M$ in the $p'$-$q$-plane. In the Modified Cam-Clay model, as described in Appendix C, the $M$-line is referred to as the critical state line and represents stress states at post peak failure.
The parameter $M$ is then based on the critical state friction angle. In the Soft Soil model, however, failure is not necessarily connected to critical state. The Mohr-Coulomb failure criterion is used with strength parameters $\varphi$ and $c$, which need not correspond to the $M$-line.

The isotropic preconsolidation stress, $p_c$, determines the magnitude of the ellipse. An infinite number of ellipses therefore exist (see Figure 4.2), each one corresponding to a particular value of $p_c$. The left hand side of the ellipse is extended into the 'tension' zone of the principal stress space ($p' < 0$) by means of the term $c \cot \varphi$ in Equation (4.5). In order to make sure that the right hand side of the ellipse (i.e. the 'cap') will remain in the 'compression' zone ($p' > 0$) a minimum value of $c \cot \varphi$ is adopted for $p_c$. For $c = 0$, a minimum value of $p_c$ equal to 1 kPa is adopted. Hence, there is a 'threshold' ellipse as illustrated in Figure 4.2.

![Figure 4.2 Yield surface of the Soft Soil model in $p'$-$q$-plane.](image)

The value of $p_c$ is affected by volumetric plastic straining and follows from the hardening relation as formulated in Equation (4.6). This equation reflects the principle that the preconsolidation stress increases exponentially with decreasing volumetric plastic strain (compaction). The value $p_c^0$ can be regarded as the initial value of the preconsolidation stress. The determination of $p_c^0$ is treated in Section 4.4. According to Equation (4.6) the initial volumetric plastic strain is assumed to be zero.

In the Soft Soil model, the yield function as defined in Equation (4.4) is only active to model the irreversible volumetric straining in primary compression and is used as the cap of the yield contour. In order to model failure behaviour, a perfectly-plastic Mohr-Coulomb type yield function is introduced. This yield function
represents a straight line in \( p'-q \)-plane. The line is indicated in Figure 4.2 as the 'Mohr-Coulomb failure line'. The inclination of the failure line is smaller than the inclination of the \( M \)-line.

The total yield contour, as shown by the bold lines in Figure 4.2, is the boundary of the elastic stress area. The failure line is fixed, but the cap may increase in primary compression. Stress paths within this boundary only give elastic strain increments, whereas stress paths that tend to cross the boundary generally give both elastic and plastic strain increments.

For general states of stress, the plastic behaviour of the Soft Soil model is defined by a total of six yield functions; three compression yield functions and three Mohr-Coulomb yield functions. For a more detailed description of these functions the reader is referred to Appendix B at the end of this manual. The total yield contour in principal stress space, resulting from these six yield functions, is indicated in Figure 4.3.

![Diagram of total yield contour](image)

**Figure 4.3** Representation of total yield contour of the Soft Soil model in principal stress space.

### 4.3 MODEL PARAMETERS IN SOFT SOIL MODEL

Some parameters of the Soft Soil model coincide with those of the non-hardening Mohr-Coulomb model. These are the failure parameters \( c \), \( \phi \) and \( \psi \). Another parameter, \( v_{ur} \), agrees to some extent with a similar parameter of the Mohr-Coulomb model, although the meaning is slightly different. Finally, three other parameters are new, namely \( \kappa^* \), \( \lambda^* \) and \( M \). The latter parameter is directly related
5 HARDENING PLASTICITY: THE HARD SOIL MODEL

In contrast to soft soils, non-cohesive and heavily overconsolidated cohesive soils do not show significant volume changes under load, at least not in the usual range of engineering loads. We thus arrive at another class of soils for which plastic straining is not associated with significant volume changes giving compaction. Instead, plastic straining is dominated by shearing; associated volumetric strains are relatively small and cause dilation rather than compaction. For the sake of convenience, these soils will be referred to as 'hard soils' and the corresponding model is named the Hard Soil model.

The Hard Soil model is intended to model the behaviour of sand, gravel and heavily overconsolidated cohesive soils. When subjected to primary deviatoric loading, hard soils show a decreasing stiffness and simultaneously irreversible shear strains develop. In the case of a drained triaxial test, the observed relationship between the axial strain and the deviatoric stress can be well approximated by a hyperbola. Such a relationship was first formulated by Kondner (1963) and later used in the well-known hyperbolic model (Duncan & Chang, 1970). The latter model also includes the effect of stiffening for an increasing confining pressure. Moreover, the Duncan-Chang model distinguishes between primary loading and unloading / reloading, for which a much stiffer behaviour is obtained. However, the Duncan-Chang model is based on elasticity theory, rather than plasticity theory, which puts some major restrictions on the applicability of the model.

The Hard Soil model implemented in PLAXIS combines the merits of plasticity theory with the logic of the Duncan-Chang model; it may be conceived as an elastoplastic Duncan-Chang model. Some basic characteristics of the model are:

- Stress dependent stiffness according to a power law.
- Hyperbolic relationship between strain and deviatoric stress.
- Distinction between primary deviatoric loading and unloading / reloading.
- Failure behaviour according to the Mohr-Coulomb model.

In this chapter a brief description of the model is given. For a more complete treatment, the reader is referred to Schanz & Vermeer (1996).

5.1 HYPERBOLIC RELATIONSHIP FOR TRIAXIAL STRESS ($\sigma'_2 = \sigma'_3$)

The starting point for the formulation of the Hard Soil model is the assumption of a hyperbolic relationship between the vertical strain, $\varepsilon_1$, and the deviatoric stress, $q$, in primary triaxial loading (Duncan & Chang, 1970):
This relationship is plotted in Figure 5.1. In Equation 5.1, the parameter $E_i$ is the initial stress dependent Young's modulus for primary loading and is given by the equation:

$$E_i = E_i^\text{ref} \left( \frac{c \cot \varphi - \sigma_3'}{P^\text{ref}} \right)^m$$

(5.2)

where $E_i^\text{ref}$ is a reference Young’s modulus corresponding to the reference pressure $P^\text{ref}$. In PLAXIS, a default setting $P^\text{ref} = 100$ kPa is used. The actual stiffness depends on the minor principal stress, $\sigma_3'$, which is the confining pressure in a triaxial test. The amount of stress dependency is given by the power $m$. In order to simulate a logarithmic stress dependency, as observed for soft clays, the power should be taken as 1.0. However, for hard soils, as considered in this material model, the power is generally lower, often around 0.5. The ultimate deviatoric stress, $q_f$, and the quantity $q_a$ in Equation (5.1) are defined as:

$$q_f = (c \cot \varphi - \sigma_3') \frac{2 \sin \varphi}{1 - \sin \varphi} \quad q_a = q_f / R_f$$

(5.3)

The above relationship for $q_f$ is derived from the Mohr-Coulomb failure criterion, which involves the strength parameters $c$ and $\varphi$. As soon as $q = q_f$, the failure
criterion is satisfied and perfectly plastic yielding occurs as described by the Mohr-Coulomb model. The ratio between \( q_f \) and \( q_a \) is determined by the failure ratio \( R_f \), which should obviously be smaller than 1. In PLAXIS, \( R_f = 0.9 \) is chosen as a suitable default setting.

For unloading and reloading stress paths, another stress dependent stiffness modulus is used:

\[
E_{ur}^{ref} = E_{ur} \left( \frac{c \cot \varphi - \sigma_3'}{p^{ref}} \right)^m
\]

where \( E_{ur}^{ref} \) is the reference Young's modulus for unloading and reloading, corresponding to the reference pressure \( p^{ref} \). In many practical cases it is appropriate to set \( E_{ur}^{ref} \) equal to \( E_i^{ref} \); this is the default setting used in PLAXIS.

5.2 Yield Function for Triaxial Stress \((\sigma_2 = \sigma_3; q < q_f)\)

For the sake of convenience, restriction is again made to triaxial loading conditions with \( \sigma_2 = \sigma_3 \) and \( \sigma_1 \) being the major compressive stress. Moreover, it is assumed that \( q < q_f \). For these states of stress the yield function of the Hard Soil model is defined as:

\[
f = \overline{f} - \gamma^p
\]

where \( \overline{f} \) is a function of stress and \( \gamma^p \) is a function of plastic strains:

\[
\overline{f} = \frac{2q}{E_i (1 - q/q_a)} - \frac{2q}{E_{ur}} \quad \gamma^p = 2\epsilon^p_1 - \epsilon^p_v = 2\epsilon^p_1
\]

with \( q, q_a, E_i \) and \( E_{ur} \) as defined by Equations (5.2) and (5.3), whilst the superscript \( p \) is used to denote plastic strains. For hard soils, plastic volume changes \((\epsilon^p_v)\) tend to be relatively small and this leads to the approximation \( \gamma^p = 2\epsilon^p_1 \). The above definition of the strain-hardening parameter \( \gamma^p \) will be referred to later.

An essential feature of the above definitions for \( \overline{f} \) is that it matches the well-known hyperbolic law (5.1). For checking this statement, one has to consider primary loading, as this implies the yield condition \( f = 0 \). For primary loading, it thus yields \( \gamma^p = \overline{f} \) and it follows from Equations (5.6) that
In addition to the plastic strains, the model accounts for elastic strains. Plastic strains develop in primary loading only, but elastic strains develop both in primary loading and unloading/reloading. For drained triaxial test stress paths with $\sigma_2' = \sigma_3' = \text{constant}$, the elastic Young’s modulus $E_{ur}$ remains constant and the elastic strains are given by the equations:

$$
\varepsilon_1^e = \frac{q}{E_{ur}} \quad \varepsilon_2^e = \varepsilon_3^e = -\nu_{ur} \frac{q}{E_{ur}}
$$

where $\nu_{ur}$ is the unloading/reloading Poisson’s ratio. Here it should be realised that restriction is made to strains that develop during deviatoric loading, whilst the strains that develop during the very first stage of the test are not considered. For the first stage of isotropic compression (with consolidation) the Hard Soil model predicts fully elastic volume changes according to Hooke’s law, but these strains are not included in Equation (5.8).

For the deviatoric loading stage of the triaxial test, the axial strain is the sum of the elastic component given by Equation (5.8) and the plastic component according to Equation (5.7). Hence, it follows that:

$$
\varepsilon_1 = \varepsilon_1^e + \varepsilon_1^p = \frac{1}{E_i} \frac{q}{1 - q/q_a} - \frac{q}{E_{ur}}
$$

which corresponds approximately to the Duncan-Chang Equation (5.1). Here it should be noted that the approximation $\gamma^p = 2 \varepsilon_1^p$ becomes exact when $\varepsilon_1^p = 0$. In reality, plastic volumetric strains will never be precisely equal to zero, but for hard soils plastic volume changes tend to be small when compared with the axial strain, so that the approximation in Equation (5.9) will generally be accurate. It is thus made clear that the present Hard Soil model coincides with a hyperbolic stress-strain model, as expressed by Equation (5.1).

For a given constant value of the hardening parameter, $\gamma^p$, the yield condition $f = 0$, can be visualised in $p'$-$q$-plane by means of a yield locus. When plotting such yield loci, one has to use Equations (5.6) as well as Equations (5.2) and (5.4) for $E_i$ and $E_{ur}$ respectively. Because of the latter expressions, the shape of the yield loci depend on the exponent $m$. For $m = 1$, straight lines are obtained, but slightly curved yield loci correspond to lower values of the exponent. For a value of $m=0.5$ (which is typical for real soils), the loci in Figure 5.2 are obtained for various constant values of $\gamma^p$. 

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5.3 PLASTIC VOLUMETRIC STRAIN FOR TRIAXIAL STRESS \( (\sigma'_{2} = \sigma'_{3}) \)

Having presented a relationship for the plastic shear strain, \( \gamma^{P} \), attention is now focused on the plastic volumetric strain, \( \varepsilon^{P}_{v} \). Similar to the other plasticity models, the Hard Soil model involves a relationship between rates of plastic strain, i.e. a relationship between \( \dot{\varepsilon}^{P}_{v} \) and \( \dot{\gamma}^{P} \). This flow rule has the linear form:

\[
\dot{\varepsilon}^{P}_{v} = \sin\psi_{m} \dot{\gamma}^{P} \tag{5.10}
\]

Clearly, further detail is needed by specifying the mobilised dilatancy angle \( \psi_{m} \). For the present model, the expression

\[
\sin\psi_{m} = \frac{\sin\phi_{m} - \sin\phi_{cv}}{1 - \sin\phi_{m} \sin\phi_{cv}} \tag{5.11}
\]

is adopted, where \( \phi_{cv} \) is the critical state friction angle, being a material constant independent of density, and \( \phi_{m} \) is the mobilised friction angle:

\[
\sin\phi_{m} = \frac{\sigma'_{1} - \sigma'_{3}}{\sigma'_{1} + \sigma'_{3} - 2 \, c \, \cot \phi} \tag{5.12}
\]

The above equations correspond to the well-known stress-dilatancy theory by Rowe (1962), as explained by Schanz & Vermeer (1995). The essential property of the stress-dilatancy theory is that the material compacts for small stress ratios \( (\phi_{m} < \phi_{cv}) \), whilst dilatancy occurs for high stress ratios \( (\phi_{m} > \phi_{cv}) \). At failure, when the
mobilised friction angle equals the failure angle, $\varphi$, it is found from Equation (5.11) that:

$$\sin \psi = \frac{\sin \varphi - \sin \varphi_{cv}}{1 - \sin \varphi \sin \varphi_{cv}}$$

(5.13a)

or equivalently:

$$\sin \varphi_{cv} = \frac{\sin \varphi - \sin \psi}{1 - \sin \varphi \sin \psi}$$

(5.13b)

Hence, the critical state angle can be computed from the failure angles $\varphi$ and $\psi$. PLAXIS carries out this computation automatically and so users do not need to specify a value for $\varphi_{cv}$.

### 5.4 MODEL PARAMETERS IN HARD SOIL MODEL

Some parameters of the present hardening model coincide with those of the non-hardening Mohr-Coulomb model. These are the failure parameters $c$, $\varphi$ and $\psi$. Two other parameters, $E_{50}^{ref}$ and $\nu_{ur}$, agree to some extent with similar parameters of the Mohr-Coulomb model. Finally, three other parameters are new, namely $m$, $R_f$ and $E_{ur}^{ref}$. In conclusion, a list is given below of the material constants used by the Hard Soil model. (Suggested values are given in brackets).

**Failure parameters as in Mohr-Coulomb model:**

- $c$: Cohesion (effective)
- $\varphi$: Angle of internal friction (effective)
- $\psi$: Angle of dilatancy

**Constants that resemble Mohr-Coulomb parameters:**

- $E_{50}^{ref}$: Primary loading stiffness ($E_1^{ref} = 2 \cdot E_{50}^{ref}$)
- $\nu_{ur}$: Unloading / reloading Poisson's ratio

**New parameters:**

- $E_{ur}^{ref}$: Unloading / reloading stiffness ($E_{ur}^{ref} = 2 \cdot E_{50}^{ref}$)
- $m$: Power in stiffness laws ($m = 0.5$)
- $R_f$: Failure ratio ($R_f = 0.9$)

Further discussion is given below of the parameters $E_{50}^{ref}$ and $\nu_{ur}$, to avoid confusion between these parameters and those that are used in the conventional Mohr-Coulomb model.

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