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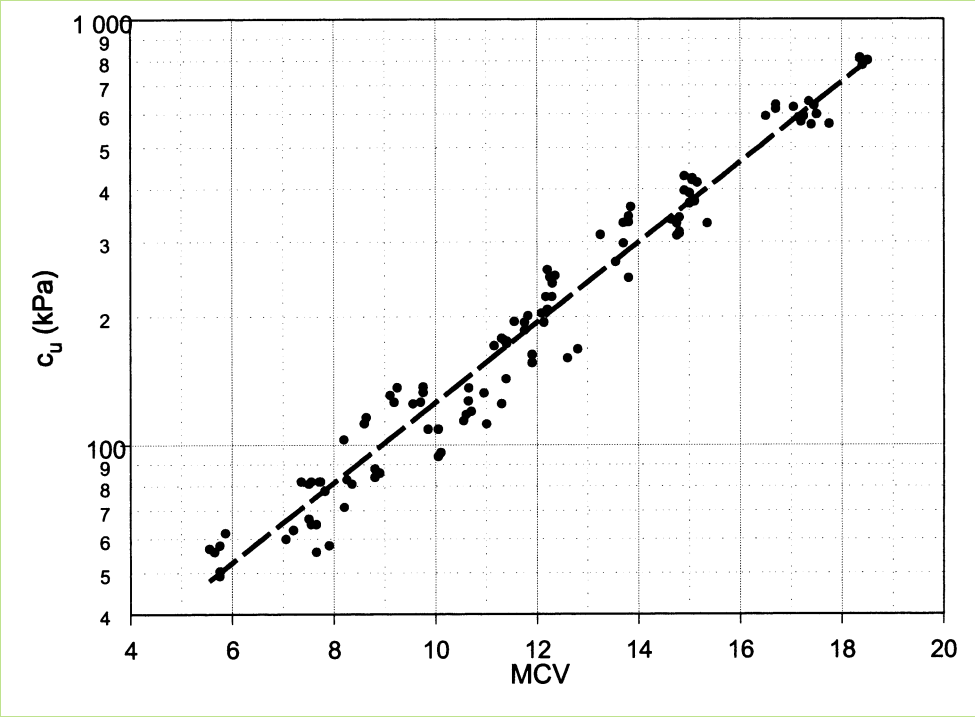

Compaction- and strength properties of stabilised and unstabilised fine-grained tills

PER LINDH

Rapport 66

LINKÖPING 2004

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Compaction- and strength properties of stabilised and unstabilised fine-grained tills

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22	Figure 3.2	Draft	-
82	Figure 5.11	(after Little 1997)	(after Little 1987)
186	EQ:7.9	$5.01*e^{(0.248*MCV)}$	$5.01*e^{(0.242*MCV)}$
187	Figure7.15	$5.01*e^{(0.248*MCV)}$	$5.01*e^{(0.242*MCV)}$
188	Table 7.10	$R^2 = 0.746$	$R^2 = 0.97$
188	Last line	over 74%	over 97%
204	Figure 7.23	Average coarse-grained	Average medium-grained
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Summary

Background

Fine-grained soils are often regarded as problematic soils in earthworks because of their water- and frost sensitivity. Only limited amounts of fine-grained tills are used as earthwork material today. The main objective of this thesis is to improve the knowledge of how to use and also treat a fine-grained till, so that a greater amount can be utilised as earthwork material. To achieve an increasing use of fine-grained tills, they must be handled in a certain way or treated/modified to achieve the desired properties. This thesis is focused on the compaction- and strength- properties of tills, both untreated and treated with a stabilising agents.

Test program

The results in this study are based on both laboratory investigations and field studies. The laboratory results are based on tests performed on 13 different soils. The main type has been clay till but coarser soils such as

clayey sand tills have also been studied. The field tests have been performed at two different sites.

Compaction properties of unstabilised fine-grained tills

Compaction is essential to achieve a good base for foundations for roads, railways and other constructions. To achieve a good result, the water content of the soil to be compacted must be within a certain range. Densification of fine-grained soils at low water contents is about to overcome the soils' "cohesion". The soils' apparent cohesion is the sum of cohesion and matrix suction. The density increase during compaction is related to the applied compaction energy and the water content of the soils.

Figure S.1 shows the curves based on Proctor compaction and on MCA compaction. These tests have different purposes, as presented in the thesis. However, as can be seen from Figure S.1, they give the same contribution to the compaction curve. The figure also shows that the soil type influences the optimum water content and resulting maximum dry density.

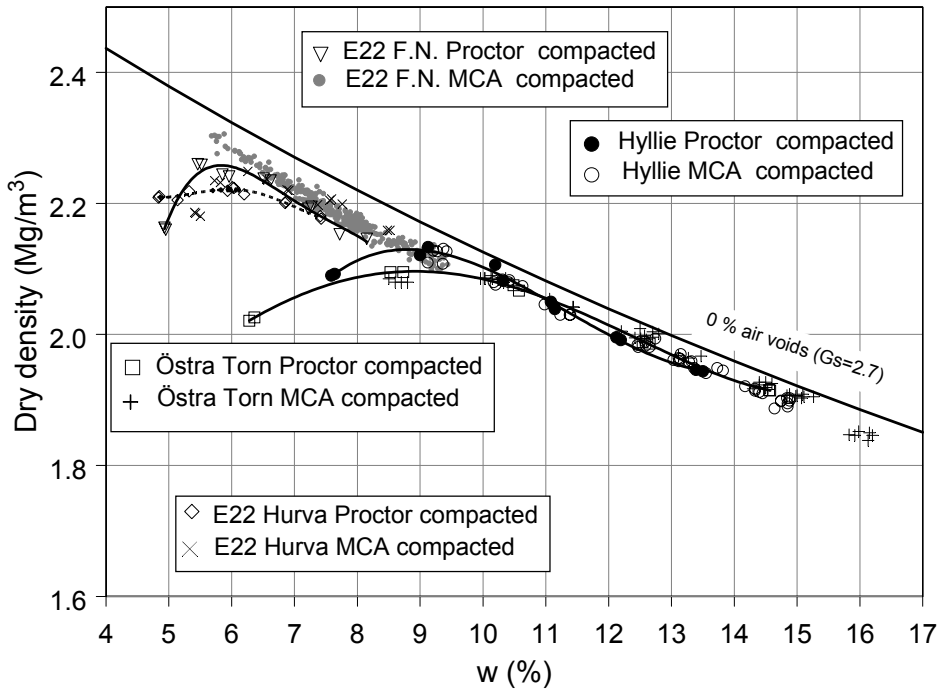


FIGURE S.1 *Dry density as a function of water content for modified Proctor and MCV-compacted specimens.*

The figure shows that the MCV test gives proper data on the achieved dry density of natural fine-grained tills at different water contents.

Strength properties of unstabilised fine-grained tills

The shear-strength tests in this study have been performed mainly on MCA compacted specimens. The MCA compacted specimen had a diameter of 100 mm and the height varied between approximately 75 and

90 mm. To fulfill a required height diameter relationship in the compression test the MCA specimens were tested in pairs placed on top of each other. This increases the slenderness ratio from approximately 0.75:1 to 1.5:1.

Some undrained shear-strength results in this study are presented in Figure S.2, and the regression line determined over 74% of the variation in the data.

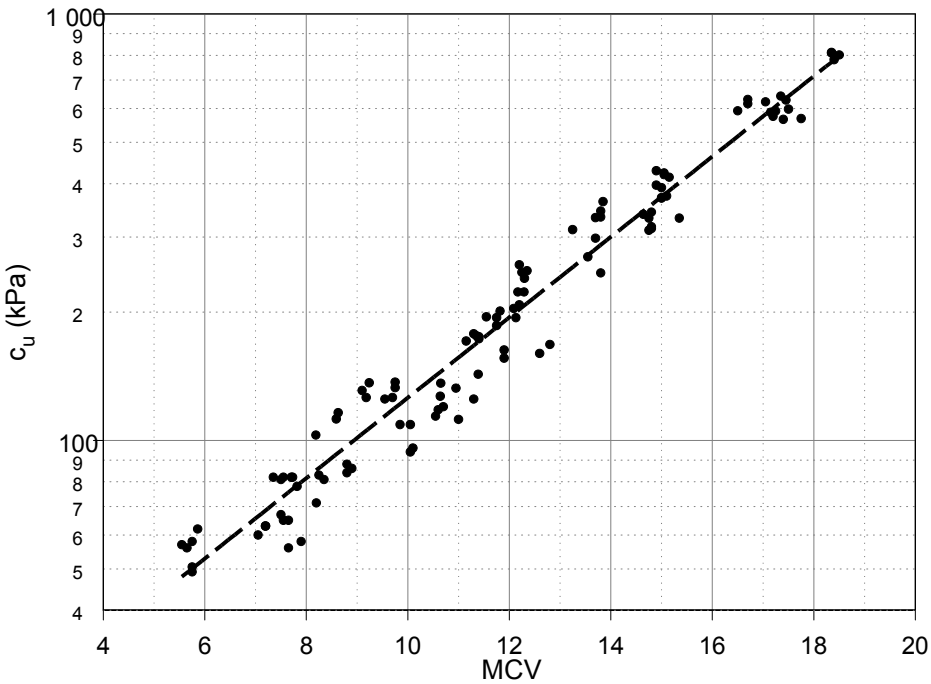


FIGURE S.2 *Shear strength (c_u) as a function of MCV for the tested soils. The result is based on 106 tests on six different tills.*

The relation between the undrained shear strength, c_u and MCV is expressed by the following equation;

$$c_u = 14.1 \cdot e^{0.22 \cdot MCV} \qquad \textbf{(EQ : S.1)}$$

where c_u is the undrained shear strength in kPa.

The results of the investigation of one of the fine-grained tills are compiled in Figure S.3. On the basis of water content, the dry density, the MCV and the undrained shear strength can be evaluated. In a planned earthwork the pre-investigation should preferrably contain a graph similar to Figure S.3. This data should be sufficient for a contractor to decide the type of earth-moving plant and capacities at different conditions of the fill material. It could also create a basis for decisions when soil modification/stabilisation is to be considered.

The MCV method is a very useful method of predicting the soils water sensitivity, the workable range regarding water contents and also predicting the soil's undrained shear strength after compaction. The MCV method is not a form of compaction control, but rather a method to determine if a soil can be sufficiently compacted at its present water content.

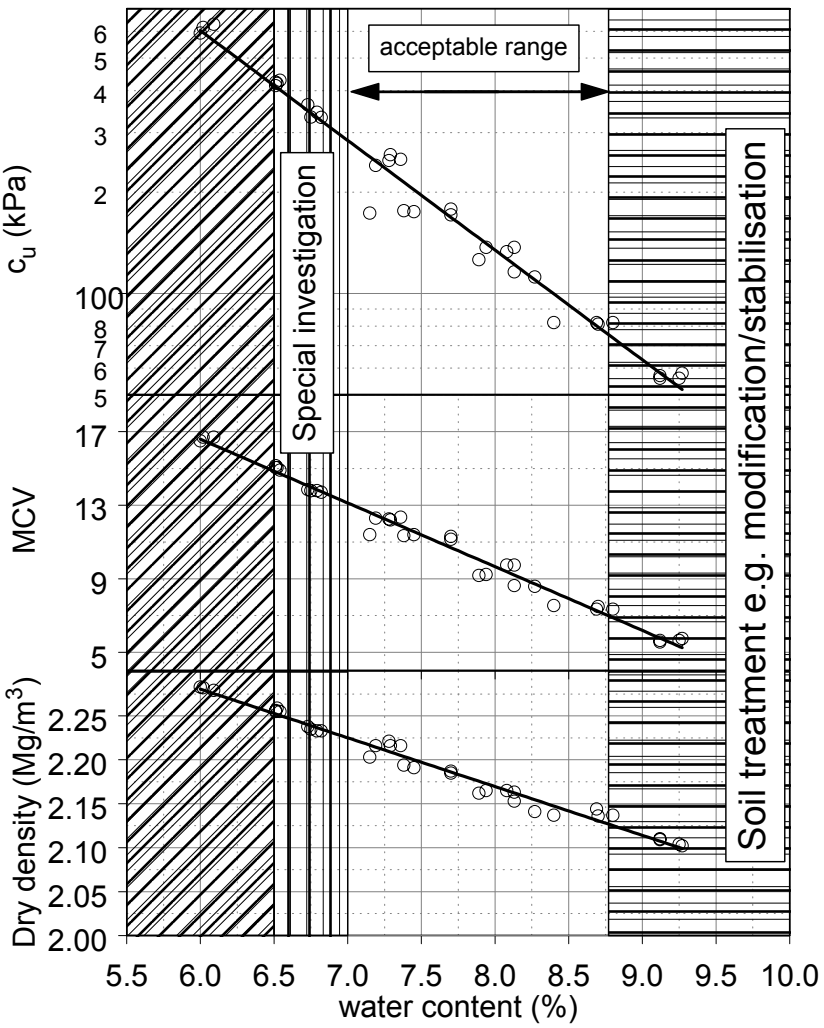


FIGURE S.3 *Relationship between water content; dry density, MCV and shear strength for the E22FN material.*

Compaction properties of stabilised fine-grained tills

Soil stabilisation and soil modification change the structure of the soil and thereby change the engineering properties. The compaction properties of a soil-binder mixture are different from those of the unstabilised material. There are different binders to be considered. In this thesis, cement, lime and steel slag have been tested induvidually and combined. The effect of stabilisation is a small reduction in water content in addition to the more important changes in structure.

Figure S.4 indicates a significant interaction effect between cement and lime. This interaction results in an even higher MCV than if cement is used alone. Without this interaction, a blend of cement and lime would only give an MCV somewhere in between the MCV obtained with cement or that obtianed with lime.

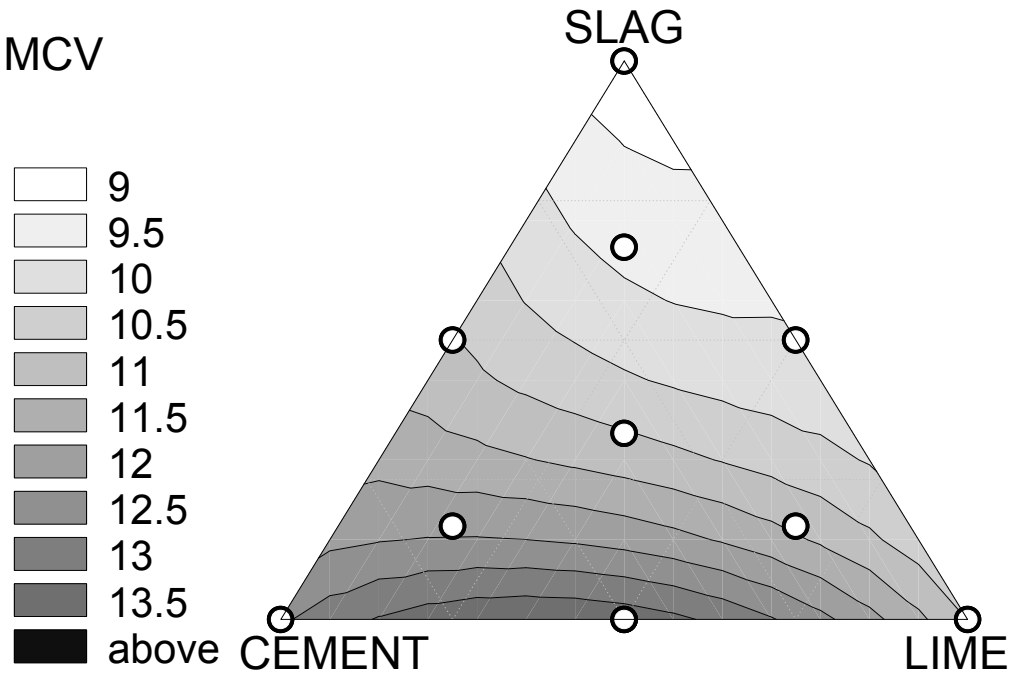


FIGURE S.4 *MCV response surface for stabilised Petersborg material. Binder content 2.5%. The delay time between mixing and compaction was one hour.*

Another test series is shown in Figure S.5. The effect of delay time on density was tested. This effect is more pronounced for a high binder content, i.e. for a binder content of 5%. The reason for the decrease in dry density after some delay time is the strength increase in the stabilised soil, owing to the cement reactions. After the reaction has occured some of the compaction energy has to be used to break the bonds created by the cement reactions and thereby less compaction energy is available for the densification. This leads to a lower dry density. However, for a binder content of 1% the delay time gives a small increase of the dry density, i.e. an increased delay time is beneficial for compaction.

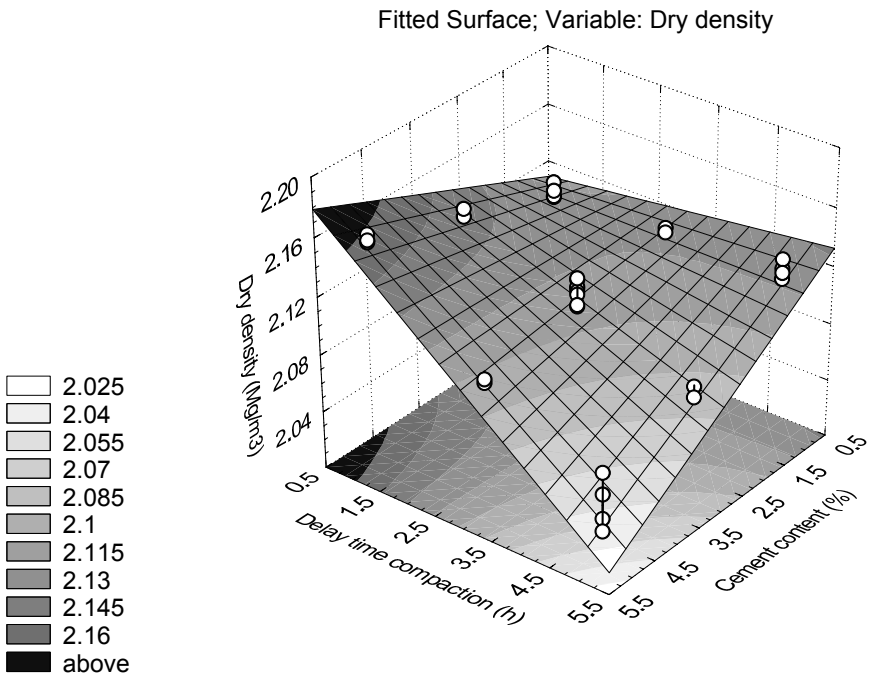


FIGURE S.5 *Dry density as a function of cement content and delay time between mixing the soil with cement and compaction with modified Proctor.*

Strength properties of stabilised fine-grained tills

In stabilised soils, the strength is dependent on binder type, total amount of binder and applied compaction effect. Since different binders have different reaction times, the strengths were studied during different curing periods.

Figure S.6 shows that cement gives the highest strength and slag the lowest and it clearly indicates the interaction effect between lime and slag when combined. Without such interaction the unconfined compression

strength (UCS) should result in a mean value of the UCS for lime and the UCS for slag. In fine-grained till, the mix of lime and slag is thus beneficial for the development of strength.

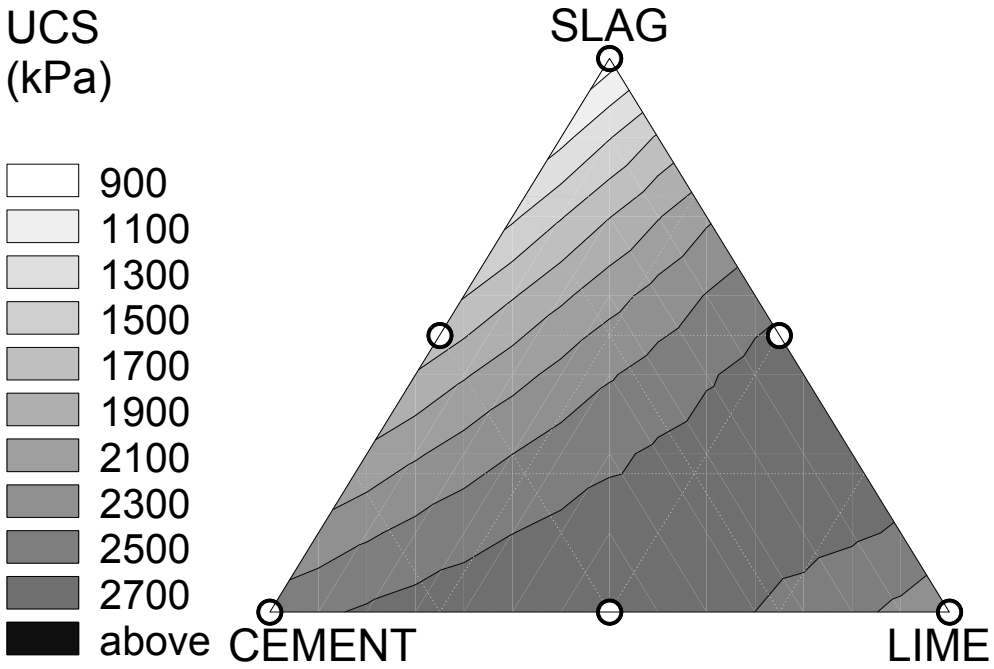


FIGURE S.6 *UCS for Petersborg material after 90-days of curing at 20°C. All specimens were vibrator-compacted one hour after mixing.*

For a cement-stabilised soil, most of the strength is reached within 28-days. Figure S.7 shows an increasing UCS with increased cement content for all tested curing times. The effect is more pronounced for a curing period of 29 days compared to 1 day curing. Further the figure shows that the strength increase with curing time is marginal for 1% cement. For a cement content of 5% the strength increase with curing time is very pronounced. With a high cement content more bondings are created and they can be assumed to be more evenly distributed.

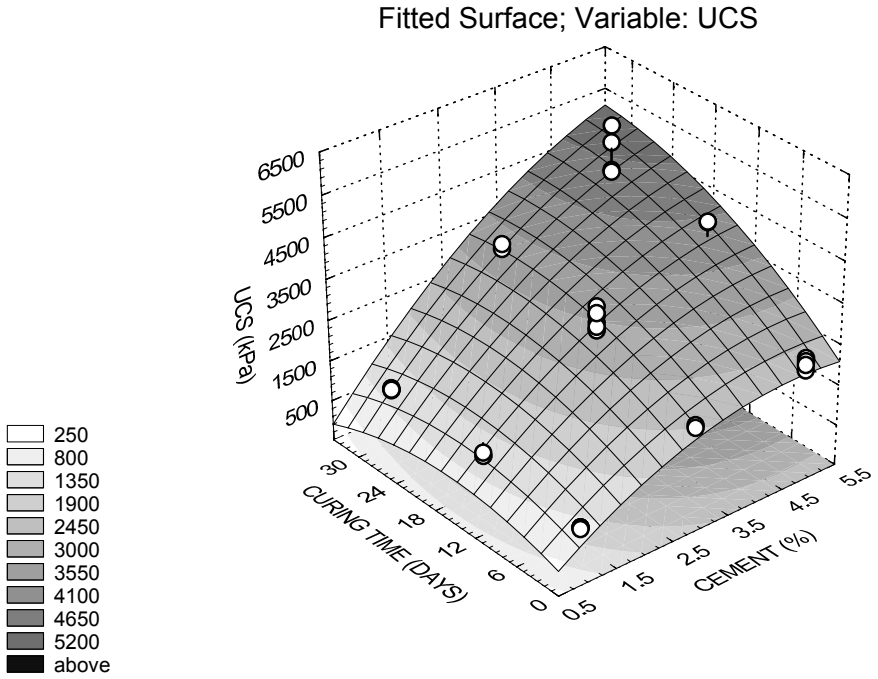


FIGURE S.7 *Unconfined compression strength as a function of cement content and curing time between compaction and UCS test for the E22FN material.*

Final remarks

To conclude: the thesis work shows that fine-grained tills can be used as qualified fills in earthworks, and that the MCA has been proven to be a very efficient and accurate tool to predict the properties of a fine-grained till.

The thesis also shows that on the the basis of moisture condition value (MCV) it is possible to predict the soil's compaction and shear strength properties at different water contents, even for a modified soil.

MCA in combination with vane tests can be used as an acceptability tool and as a control tool.

The soil treatment could include stockpiling and dewatering of the soil by aereation. However, a faster and more precise soil treatment method is soil modification or soil stabilisation, which is also less weather dependent.

The tests presented show that Swedish fine-grained tills can be treated by single or combined additives.

Lime is the main soil-stabilising agent. However, cement may be preferred in cold weather conditions since to the chemical reactions also occur at low temperature.

Blended binders have proven to be very efficient and could well compete with single binders and in many cases give a considerably better effect than a single binder. Blended binders have many advantages regarding the binders' working period.

The evaluations of blended binders are preferable performed with response surface methodology (RSM) a statistical evaluation technique that evaluates the interactions between the different agents. Different RSM techniques should be used depending on which type of parameter is to be evaluated.

Sammanfattning

Bakgrund

Finkorniga moräner betraktas ofta som problemjordar inom anläggningsbranschen beroende på jordarnas känslighet för förändringar i vatteninnehåll och för deras tjälegenskaper. I dagsläget används endast en begränsad mängd finkorniga moräner i anläggningsbyggande. Huvudmålet med denna avhandling har varit att öka kunskapen om dessa jordars egenskaper och att finna/utveckla möjligheter att behandla dessa så att de kan få ökad användning i anläggningsbyggandet.

För att uppnå en ökad användning av finkorniga moräner gäller det att de behandlas på ett korrekt sätt eller att deras egenskaper förändras så att de uppfyller satta kriterier. Denna avhandling är fokuserad på jordarnas packnings- och hållfasthetsegenskaper både med och utan bindemedel.

Försöksprogram

Resultaten i denna avhandling är till största delen baserade på laboratorieförsök. Laboratorieförsöken har omfattat 13 olika jordar varav huvuddelen är lermoräner men också jordar som leriga sandmoräner har studerats. Fältförsök har utförts på två platser.

Packningsegenskaper hos ostabiliserade finkorniga moräner

Packning är mycket viktig för att uppnå en bra grundläggning av vägar, järnvägar och övriga konstruktioner. För att uppnå ett bra resultat måste vattenkvoten i jorden vara inom ett visst spann. Packning av en finkornig morän med låg vattenkvot handlar om att övervinna dess kohesion. Jordens skenbara kohesion är summan av porundertryck och äkta kohesion. Ökningen av jordens densitet vid packning är relaterad till använd packningsenergi och jordens vattenkvot.

Figur S.1 (Se engelsk sammanfattning) visar packningskurvor baserade på modifierad Proctor och MCA-packade prover. Proctor och MCV-försök har olika syften som framgår av denna avhandlingen. Emellertid ger försöken liknande resultat, se Figur S.1. Vidare visar figuren att jordtyper har inverkan på den optimala vattenkvoten.

Hållfasthetsegenskaper hos ostabiliserad finkornig morän

Skjuvhållfastheten har företrädelsevis baserats på MCA-packade provkroppar med en diameter på 100 mm och medan höjd som varierat mellan 75 och 90 mm. För att få ett bättre höjd/diameter förhållande så har två provkroppar satts ihop på höjden. Detta medför att slankhetstalet ökar från ca 0.75:1 till ca 1.5:1. Några enaxiella tryckförsök på olika jordar har sammanställts i Figur S.2. Regressionsmodellen förklarar 74% av variationen i hållfastheten.

Relationen mellan odränerad skjuvhållfasthet, c_u, och MCV kan uttryckas enligt ekvation EQ:S.1.

$$c_u = 14.1 \cdot e^{0.22 \cdot MCV} \qquad \textbf{(EQ : S.1)}$$

där c_u är uttryckt i kPa.

Resultaten från en av de undersökta jordarna är sammanställd i Figur S.3. Från jordens vattenkvot kan torrdensiteten, MCV, och den odränerade skjuvhållfastheten bestämmas. I en förundersökning för ett anläggningsarbete bör en sammanställning enligt Figur S.3 göras. Med dessa uppgifter kan en entreprenör välja typ av schaktningsutrustning och kapaciteter vid olika vattenkvoter hos materialet. Figuren skapar också ett underlag för om modifiering/stabilisering skall övervägas.

MCV-metoden är en mycket användbar metod för att prediktera jordens vattenkänslighet, bearbetbarhet och hållfasthet efter packning vid olika vattenkvoter. MCV-metoden är ingen packningskontroll utan snarare en metod för att bestämma om jorden överhuvudtaget kan packas vid aktuell vattenkvot.

Packningsegenskaper hos en stabiliserad finkornig morän

Stabilisering och modifiering förändrar jordens struktur och därmed också dess egenskaper. Packningsegenskaperna hos en stabiliserad jord skiljer sig från egenskaperna hos en ostabiliserad jord. Flera olika bindemedel kan komma i fråga. Denna avhandling behandlar cement, kalk och stålslagg antingen var för sig eller som blandning. Effekten av stabilisering är en liten reduktion av vattenkvoten förutom den mer betydelsefulla strukturförändringen.

Figur S.4 visar ett signifikant samspel mellan cement och kalk . Samspelet resulterar i ett högre MCV än om bara cement eller bara kalk skulle använts. Utan detta samspel skulle en blandning av cement och kalk ge ett resultat mellan det som erhållits med bara cement respektive bara kalk.

Ytterligare en annan testserie visas i Figur S.5. I denna serie testades effekten på torrdensiteten av olika fördröjningstider och bindemedelsinnehåll. Figuren visar att vid 5 % cement minskar den stabiliserade jordens torrdensitet markant med ökad fördröjningstid. Orsaken till reduktionen av torrdensiteten är hållfasthetstillväxten av cementen. Eftersom cementreaktionerna bygger upp en ökad hållfasthet i jorden kommer en del av packningsenergin att gå åt till att krossa cementbindningarna i stället för att öka torrdensiteten. Vid en cementhalt på 1 % ökar torrdensiteten med fördröjningstiden men endast marginellt.

Hållfasthetsegenskaper hos stabiliserad finkornig jord

I en stabiliserad jord beror hållfastheten på typ av bindemedel, mängd bindemedel och packningseffekten. Olika bindemedel har olika härdningstider och därför har hållfastheten studerats efter olika långa lagringstider.

Figur S.6 visar att cement ger den högsta hållfastheten och slagg den lägsta. Vidare visas samspelseffekten mellan kalk och slagg när de är blandade. Utan detta samspel skulle tryckhållfastheten för ett blandat bindemedel av kalk och slagg bli medelvärdet av de båda.

För en cementstabiliserad jord är större delen av hållfastheten nådd efter 28 dygn. Figur S.7 visar en ökad tryckhållfasthet med ökat cementinnehåll. Effekten är mer uttalad vid 29 dygns lagring jämfört med ett dygns lagring. Vidare visar figuren att hållfasthetsökningen är marginell vid 1 % cementinnehåll. För 5 % cementinnehåll är hållfasthetsökningen med tiden väldigt uttalad. Ett högre cementinnehåll resulterar i fler och starkare bindningar som också kan förväntas vara jämnare fördelade.

Slutkommentarer

Denna avhandling visar att finkorniga moräner kan användas som kvalificerat fyllningsmaterial och att MCA har visat sig vara ett mycket effektivt och exakt redskap för att prediktera finkorniga jordars egenskaper.

Vidare visas att med hjälp av MCV är det möjlig att prediktera en jords packnings- och hållfasthetsegenskaper även för en modifierad jord.

MCA kan tillsammans med vingförsök fungera som acceptansverktyg och kontrollverktyg.

En finkornig morän kan läggas upp för avvattning genom luftning men en mycket snabbare och mer precis behandling är modifiering som dessutom är mindre väderberoende.

Försöken visar att finkorniga svenska moräner kan behandlas med både enkla och blandade bindemedel.

Kalk är det dominerande bindemedlet men cement kan i vissa sammanhang vara att föredra eftersom cement reagerar även vid låga temperaturer.

Blandade bindemedel har visat sig väl så bra som enkla bindemedel och kan i många fall ge bättre resultat. Blandade bindemedel har oftast längre bearbetbar tid och är därför att föredra.

Vid utvärdering av blandade bindemedels effekt bör responsytemodellen användas. Detta är en statistisk utvärderingsmetod som kan visa på om där föreligger samspelseffekter mellan olika bindemedel. Beroende på vilka parametrar som skall utvärderas kan olika responsytemodeller användas.

List of symbols and abbreviations

Roman letters

A_0	initial sample area (m^2)
a	intercept between MCV calibration line and Y-axis for (MCV=0)
b	slope of the MCV calibration line
Cu	uniformity coefficient
c_u	undrained shear strength (kPa)
c_v	undrained shear strength determined by vane (kPa)
D	diameter of specimen (mm)
D_{10}, D_{60}, D_{90}	grain size corresponding to 10,60 and 90% respectively on the grain size distribution
E	compaction energy (KJ/m^3)
E_{v2}	deformation modulus determined by static plate bearing test (Mpa)
Evd	deformation modulus determined by light drop-weight (Mpa)
e, e_0	void ratio
F	force at failure (kN)
H	height of specimen (mm)
I_L	liquidity index (%)
I_P	plasticity index (%)
K	material dependent constant (-)
k	capillarity (m/s)
L	material dependent constant (-)
l_c	clay content (decimal value)
M	hydraulic modulus (-)
P	degree of pulverisation (%)
q	deviator stress (kPa)
R^2	the square of the multiple correlation coefficient
R^2_{adj}	adjusted R^2
R_{it}	indirect tensile strength (kPa)
r	Pearsons correlation coefficient
w, w_0	water content (%) (same as moisture content)
w_L	liquid limit (%)
w_P	plastic limit (%)
w_N	natural water content (as dug)
w_{m8}	w corresponding to MCV = 8 (%)
w_{c2}	w corresponding to CBR = 2 (%)
w_{s50}	w corresponding to c_u = 50 (%)
w_{IL15}	w corresponding to I_L = 15 (%)

Greek letters

α, β, κ, λ	material dependent constant (-)
α_e, β_e	material dependent constant (-)
τ_{fu}	undrained shear strength (kPa)

Abbreviations

ANOVA	Analysis of variance
ASTM	American Society for Testing Materials
BS	British Standard
CBR	California bearing ratio
CCD	Central Composite Design
CEN	Comité Européen de Normalisation
CVES	Continuous Vertical Electrical Sounding
DIN	Deutsches Institut für Normung
DMM	Deep Mix Method
GB	General Blend
GGBFS	Ground Granulated Blast-Furnace Slag
GP	General Purpose
ICL	Initial consumption of lime
ISSMGE	International Society for Soil Mechanics and Geotechnical Engineering
LBDD	Ligno bond DD
LVDT	Linear voltage displacement transducers
MCA	Moisture condition apparatus
MCV	Moisture condition value
NSW	New South Wales
OCR	Over consolidation ratio
OMC	Optimum moisture content = OWC (%)
OPC	Ordinary Portland cement
OWC	Optimum water content (%)
RSM	Response surface methodology
SGI	Swedish Geotechnical Institute
SGU	Swedish Geological Survey
SNRA	Swedish national road administration
SS	Swedish Standard
TRL	Transport Research Laboratory
TRRL	Transport and Road Research Laboratory
UCS	Unconfined compression strength
UU	Unconsolidated undrained
VTI	Väg- och transportforskningsinstitutet

1 Introduction

1.1 Background

Societies have from ancient times built infrastructure to maintain and develop their surronding. The main function has been to transport people, knowledge and goods between different areas. Our society also needs to transport goods and people, despite the development of the wireless infrastructure. Trucks, cars, and trains play a major part in modern transport work, together with air planes and ships. In order to make these facilities work, a backbone of roads, railways, airfields and harbours is needed.

These backbones have one thing in common - aggregates of soil or equal materials play a major role in their construction. The term aggregate is defined as "a collection or sum of units or parts somewhat loosely associated" (Anon., 1993). The shape and size of aggregates used in different infrastructure systems vary from rough to rounded and very fine-grained to very coarse-grained. The origin of the aggregate could be natural material e.g. sand and gravel or man-made material e.g. crushed bedrock. The supply and quality of aggregates vary with the geology. In

some areas the lack of usable material leads to new innovations. The Appian Way, one of the main roads to ancient Rome was stabilised with lime 300 BC (Lambe, 1962).

There has been a small revolution in road construction during the 20th century due to better geotechnical understanding as well as a high degree of mechanisation. The mechanisation has meant that aggregates have been manufactured more easily, transported and placed without limitations at the construction site. Some types of aggregates are more useful than others and are therefore more desirable. Sand and gravel for example, are very useful aggregate types that could be used in many types of constructions. On the other hand, natural sand and gravel formations play very important roles in the aquifers of ground water.

High quality materials such as sand and gravel should only be used in those cases where no other technical or economic alternative is available. In Sweden, there are alternatives such as tills or crushed bedrock. Approximately 70% of the Swedish land area is covered with tills and the most common type is the fine-grained till. However, fine-grained soils are often regarded as problematic soils in earthworks due to high water sensitivity and frost heave.

Different parts and types of an earthwork or embankment require different quality of the earthwork material. The most important properties are compactability, stiffness, bearing capacity and non-frost-susceptibility. The water sensitivity results in low strength and low bearing capacity at too high water contents. However, properly treated, these soils could perform very well regarding stiffness, strength and in some cases even frost resistance. Since tills are the most common soils in Sweden

they constitute a huge material reserve that has so far been very little utilized.

1.1.1 Use of aggregates in Sweden

The Swedish use of aggregates between 1930 and 2000 is shown in Figure 1.1 (Anon., 2002). The use of aggregates had a maximum of 135 million tons during the 1970s and has then dropped to approximately 70 million tons during 2002; see Figure 1.1 (Anon., 2002). Swedish road construction projects consume more than 50% of the annually produced aggregates and the most common material type is the crushed bedrock, see Figure 1.2 (Anon., 2002).

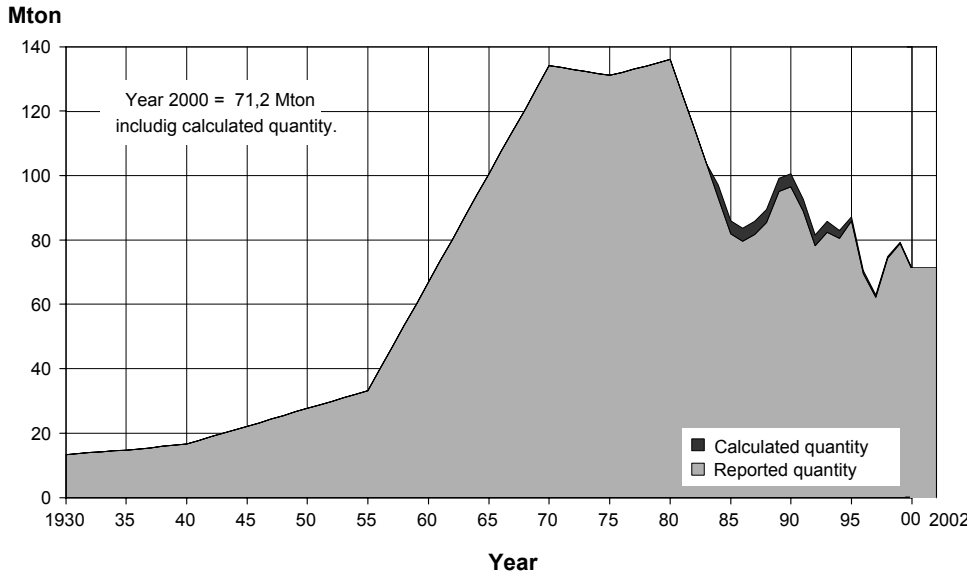


FIGURE 1.1 *The total use of aggregates in Sweden during 1930 to 2002 (After Anon., 2002).*

This could be compared with the conditions in the UK, where roads use only 32% of the total aggregates (Shearwood, 1995).

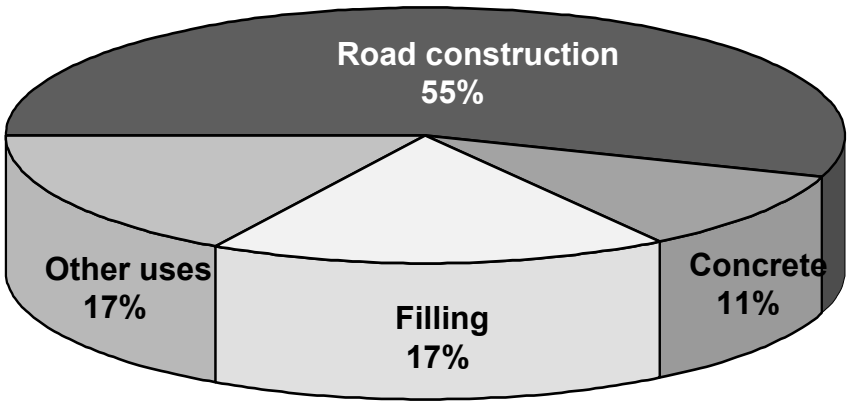


FIGURE 1.2 *The total deliveries of aggregate 2001 distributed as percentages on consumption areas (Anon., 2002)*

The origin of the aggregates is shown in Figure 1.3.

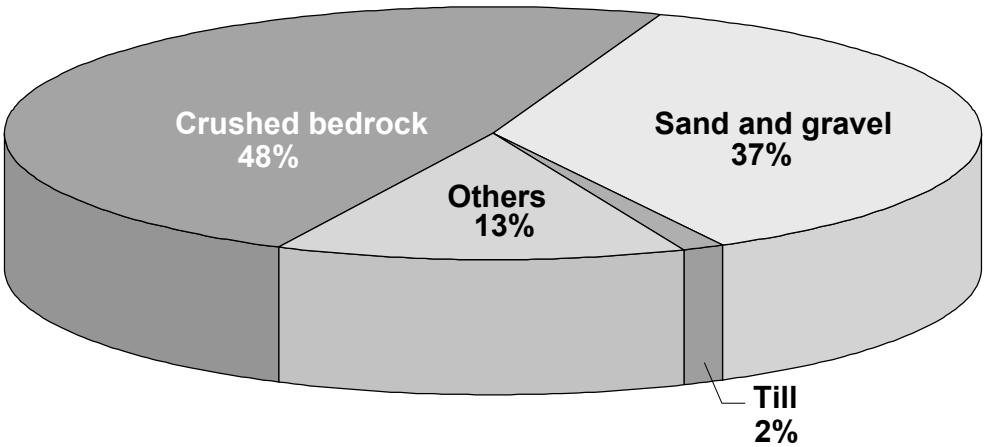


FIGURE 1.3 *The deliveries of aggregates in Sweden 2000 distributed as percentages of types of material (Anon., 2002).*

1.1.2 Alternative materials for earthworks

The source of fill material for earthworks could consist of all types of geological or man-made material. Man-made material could consist of crushed bedrock, waste, recycled materials and by-products. Waste, recycled materials and by-products mainly originate in industrial and urban areas and are most suited to be used close to the production sites.

1.1.3 Making use of on-site soils

Making use of on-site material in an earthwork project could benefit both economic and environmental interests. In projects where fine-grained till is predominant there need to be special investigations and considerations. The soil's acceptability criterion needs to be determined as part of the pre-investigation. The project's schedule needs to be considered when it comes to handling fine-grained tills i.e. can the soil be handled during the particular season's normal weather conditions and will it obtain the right conditions during the available time?

There is an increase in privately funded projects and these are normally more sensitive to delays or long construction time. The economics of making use of on-site soils are critically dependent on the selection of appropriate construction methods. Different types of tills have different properties and the construction method should be chosen to match the actual properties.

1.2 Objectives and scope

Only limited amounts of fine-grained tills are today used as earthwork material. The main objective of this thesis is to improve the knowledge of how to treat a fine-grained till, so that a greater amount can be utilised as earthwork material. To achieve an increasing use of fine-grained tills, they must be handled in a certain way or treated/modified to achieve the desired properties. This thesis is focused on the compaction- and strength properties of tills, both untreated and treated with a stabilising agent.

Results from this study can be applied to different types of earthworks such as embankments, dams, clay liners etc.

The study is limited to fine- and medium-grained tills and inorganic binders. A further limitation is that the study does not include estimates of the environmental benefits of utilisation of fine-grained tills.

1.3 Outline of the thesis

This thesis is divided into three major parts. First, chapter 2 with introduction to experimental designs in geotechnical laboratory investigations, where some of the problems with introducing statistical methods are highlighted. Second, chapters 3 to 5 with mainly literature reviews. Chapter 5 also includes the author's own experience from two study visits to Australia. Third, chapters 6 and 7 contain laboratory and field testing where chapter 6 contains method description and chapter 7 results and discussion. Chapter 6 includes the authors own results for validation of methods.

1.4 Project description

This Ph.D. project started with focus on the behaviour of clay till. It then changed over to soil stabilisation of fine-grained tills and resulted in a licentiate thesis (Lindh, 2000). After this first stage the lack of knowledge of how compacted fine-grained tills behave without any stabilisation became evident. The second part of this study was then aimed at both stabilised and unstabilised fine and medium-grained tills.

2 Methodology

2.1 Hypothesis and approach

The main hypothesis is that there are possibilities to use fine-grained tills as qualified earthwork material. However, the management has to consider the specific properties of the fine-grained material as regards weather and frost-susceptibility. The approach is to verify the important factors and methods to maintain these factors; further, to study possibilities to improve the properties and find the optimal design.

2.2 Experimental design

In geotechnical engineering including soil stabilisation the practical work is often performed under conditions very different from those in most other branches and the methods used must be robust under the prevailing conditions. There are several natural conditions, which may vary at a site, e.g. soil-grading, water content, the soil's mineral content etc. Statistical methods have been employed to evaluate different treatments in laboratory both for untreated soil and for stabilised soil.

To achieve a high-quality product stabilisation may be an option. The stabilisation mixture must also be designed to account for variation in the amount of stabilisation agent used, mixing homogeneity and degree of compaction.

In order to design a high-quality blend, the determinant factors for the properties of the stabilised soil must be defined. In all experimental work, basic knowledge and pre-experimental planning are important for the quality. The technique used to evaluate the amount of stabilisation agent differs from that used to evaluate which type of stabilisation agent is suitable. To evaluate the required amount of binder, it is necessary to define lower and upper limits of binder contents. This is best done with a small series of pre-tests combined with experience. The pre-tests can be performed with a few specimens and one or two response variables. The final test design is decided based on the results of the pre-tests. To evaluate the difference between different stabilisation agents the first step is to decide which stabilisation agents are suitable for the purpose. The main requirements are based the following response variables:

- Unconfined compressive strength (UCS) at different curing times
- OWC (optimum water content)
- Changes in water content after mixing
- Workability of a soil/binder mixture (working time)

To perform experiments efficiently, a scientific approach and experimental planning must be employed. This is achieved by using a statistical design for experiments. Since the experiments are performed to draw meaningful conclusions from the data, the statistical approach is necessary. Furthermore, since all data in this study are subject to experimental errors, statistical methodology is the only objective approach to perform the analysis (Montgomery, 1996).

There are some basic principles that are useful for understanding experimental design. These are replication, randomisation and blocking. Replication can be defined as how many specimens are treated in exactly the same way. If five specimens have been produced in the same way, there are five replicates. Randomisation is fundamental to statistical methods in experimental design. To ensure that the observations are independently distributed, a randomisation between the observations must be performed. For example, suppose that stabilised soil specimens are made from a soil from a container with lower water content at the top than at the bottom. If two different treatments, A and B, are being tested and all the specimens in treatment A are produced first, then the observations are not independent. Randomising, that is producing the specimens in randomised order will alleviate this problem. Blocking is a technique to increase the precision of an experiment. The idea is to select a portion of the experimental material that should be more homogeneous than the entire set of material and make a block of this portion (Box *et al.*, 1978; Anon., 1995a; Montgomery, 1996).

When statistical methods are implemented in experimentation, the following points should be considered (Montgomery, 1996; Box *et al.*, 1978).

- Find out as much as possible about the problem
- Use non-statistical knowledge of the problem
- Design objectives
- Recognise the difference between practical and statistical significance

Experiments are usually iterative so the points above may need to be reconsidered after the first run.

Hypothesis testing is a technique that is mostly used to demonstrate a difference between different treatments by rejecting a null hypothesis. For example, to evaluate if two different test methods on shear strength differs. The test methods could be single height testing against double height testing. The null hypothesis could for example be that the mean strength is equal between the test methods. This could formally be stated as:

$$H_0: \mu 1 = \mu 2 \qquad \textbf{(EQ : 2.1)}$$

$$H_1: \mu 1 \neq \mu 2 \qquad \textbf{(EQ : 2.2)}$$

where $\mu 1$ is the mean strength of the single height testing and $\mu 2$ is the mean strength of the double height testing. If the test shows that the null hypothesis is rejected then it is customary to call the test statistically significant (Montgomery, 1996). The significance level is defined by α and it is the probability to reject H_0 if H_0 is true. In this study $\alpha = 0.05$ was chosen for all tests. The 5 % level is considered as a "border-line acceptable" error level. The p-value is a decreasing index of the reliability of a result. The higher the p-level, the less we can believe that the observed relation between variables in the sample is a reliable indicator of the relation between the respective variables in the population (Anon., 1995a).

Some other terms that are very useful to know when it comes to experimental designs are:

- Independent variables, these are the input variables, i.e. those that are controlled by the engineer or scientist.
- Dependent variables, these are the response variables, i.e. those that are only measured or registered.

2.2.1 Response Surface Methodology

Response surface methodology (RSM) is a collection of statistical and mathematical techniques useful for developing, improving and optimising processes. This technique has also important applications in the design, development, and formulation of new products, as well as in the improvement of existing product designs. RSM is used in empirical study of the relationships between one or more dependent variables such as strength and density and a number of input variables such as binder content, curing temperature and curing period.

The response-surface (model) is usually represented graphically where the response is plotted against the levels of the factors (Box *et al.*, 1978; Myers and Montgomery, 1995). The model parameters can be estimated most effectively if proper experimental designs are used to collect data. These designs are called response-surface designs. Fitting and analysing response surfaces are greatly facilitated by a proper choice of an experimental design. To choose a response-surface design, some of the features of a desirable design are as follows:

- Provides a reasonable distribution of data points throughout the region of interest.
- Allows model adequacy, including lack of fit to be investigated.
- Allows experiments to be performed in blocks.
- Allows designs of higher order to be built up sequentially.
- Provides an internal estimate of error.
- Does not require a large number of runs.
- Does not require too many levels of the independent variables.
- Ensures simplicity of calculation of the model parameters.

Some of these features are sometimes conflicting, and judgement must often be applied in design selections (Montgomery, 1996).

2.2.2 Recommendations

When designing an experiment, a response surface methodology is very efficient for evaluating the effects of different binders in soil stabilisation. However, employing an RSM unreflectingly could cause great problems in evaluating the results. Different types of designs should be used, depending on the purpose of the test. The lesson learned in this study is that engineering judgement must be incorporated into the whole process from pre-testing to choosing experimental design and in the evaluation of an experiment.

In research different test methods are employed. In order to evaluate if there is a difference and if this difference is significant the hypothesis testing should be used.

Further details are presented in appendix A.

3 Fine-grained tills

3.1 Definition of tills

The common descriptive characteristic of most tills is poor or very poor sorting. This is particularly the case for tills with high contents of silt and clay. Dreimanis and Lundqvist (1984) stated the following conditions to be common to all tills:

- till consists of debris that has been transported by glacier;
- there is a close spatial relationship to a glacier:
  -till is deposited by a glacier (ortho-tills), or
  -till deposited from a glacier (allo-tills);
- sorting by water is absent or minimal
  during the formation of till.

There are many volcanic mudflow deposits of various derivations that may be mistaken for till (Dreimanis and Lundqvist, 1984).

3.2 Fine-grained tills as earthwork material

Fine-grained soils have properties that can be very useful in earthworks and other properties that could cause great problems in this context if the material is not treated properly. It is therefore important to treat these types of soils in such a way that the potential problems never occur, or if they do occur, that they are minimised. Many earthworks start with excavation followed by transportation of the soils from the borrow pit to the construction site. At the construction site, the soils are placed and compacted. During this operation different engineering properties of the soil are in focus depending on the stage of the earthwork.

3.2.1 Specifications for fine-grained fill

In Sweden the tender document for an earthwork often refers to the AMA-system, AMA 98 (Anon., 1999a). The AMA-system is built up in different sections. In the earthwork section there are specifications for road and railway embankments.

The soils are divided into different classes depending on grain size distribution. The materials used in this study are classified as 4A, 4B and 5A, cf. Table 3.1.

Table 3.1: *Fill material for earthworks. (After Anon., 1999a)*

Material type	Example of soil types
4A	Silty till
4B	Clay, clay till
5A	Silt, clayey silt, silty clay, silt till

Embankments constructed properly of these types of soils should be designed with drainage layers and allowed to consolidate according to Table 3.2. See also Figure 3.1.

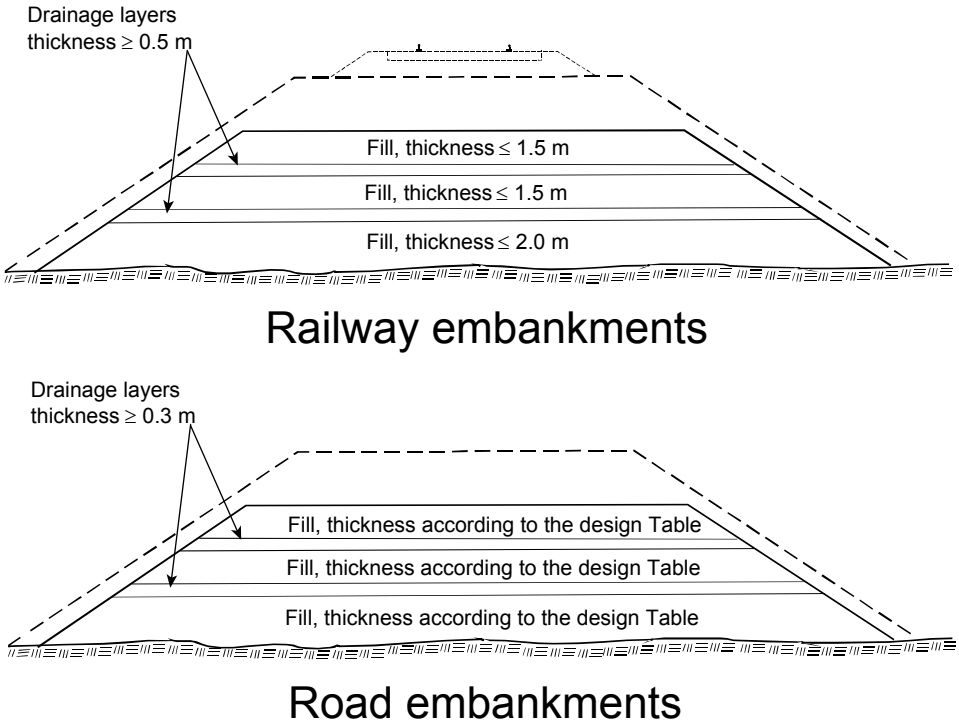


FIGURE 3.1 *Designs for railway- and road embankments of mixed and fine-grained soils, cf. Table 3.2.(After Anon., 1999a).*

Table 3.2: *Drainage layer and consolidation time. (After Anon., 1999a).*

Material type	Cu D_{60}/D_{10}	Water content w (%)	Distance between drainage layers[a] (m)	Consolidation time[b] (months)
3B, 4A	<5	<7	Speciall investigation	-
3B, 4A	<5	7-12	-	-
3B, 4A	<5	>12	<2	4
3B, 4A	<5	>12	2-4	6
3B, 4A	≥5	<5	Speciall investigation	
3B, 4A	≥5	5-10	-	-
3B, 4A	≥5	>10	<2	4
3B, 4A	≥5	>10	2-4	6
4B	-	<20	Speciall investigation	-
4B	-	20-35	<2	3
4B	-	>35	-	[c]
5A	-	<7	Speciall investigation	-
5A	-	7-12	<2	6
5A	-	>12	2-4	9

a. Embankment height if dranage layers are missing
b. Under unfrozen conditions
c. According to settlement measurements

The literature contains several studies of this type of design. Some conclusions from those studies are described below.

A trial embankment consisting of boulder clay with drainage layers was constructed in 1966 (Grace and Green, 1979), which confirmed that:

- An embankment could be constructed from a clay with a much higher water content than would normally be allowed by the current specifications.
- The rate of the pore pressure dissipation could be controlled by the spacing between the drainage layers.

The same type of design was used in the construction of a 26 metre high motorway fill on the E6 road northeast of Oslo (Østlid, 1981). The embankment was of "sandwich" type with 0.2m sand layers and 1.4m fill layers, cf. Figure 3.1. The sand layers were used to reduce the pore pressure in the clay and to increase the rate of settlement. The fill material consisted of soft silty clay with a water content ranging from 22% to more than 30%.

The same type of construction was tested before the construction of the outer ring road around Malmö (Brorsson and Pettersson, 1997). This trial embankment was 40 metres long and 4.5 metres high. The fill consisted of clay till and was placed in layers of 0.3 to 0.5 meters. The conclusions from this test were that clay till with a moisture condition value (MCV) of 4 could be used for less qualified fills if an appropriate consolidation time was allowed, combined with an after-compaction. Another conclusion was that the settlement induced by the self-weight of the fill only became about 1% of the embankment height.

From a general point of view, there are three different requirements that the soil must meet in order to be suitable for embankment purposes (Arrowsmith,1979; Dennehy, 1979; Dohaney and Forde, 1979). These are:

1. the ability to use normal methods and normal equipment to excavate, transport, place and finally construct with the soil.
2. the ability to form embankments with stable side-slopes
3. non significant future settlement in the embankment.

3.3 Engineering properties of compacted fine-grained tills

The demands for the engineering properties of the soil can be related to the different stages in the earthwork. An earthwork often starts with excavation followed by transport, filling and compaction; filling can in some cases be performed without any densification apart from the fill's own deadweight. However, most engineering fills are intended to support different types of structures such as roads, railways or buildings and therefore need to be densified by compaction. In order to keep an even quality of the fill, there have to be criteria for acceptability and control of the fill material.

3.3.1 Excavation

Fine-grained tills occur in different degrees of overconsolidation from normally consolidated to heavily overconsolidated, depending on the formation and history of the till. The degree of overconsolidation determines the effort with which the soil can be excavated. Heavily overconsolidated fine-grained tills can be very hard to excavate and require heavy excavation equipment. The overconsolidation ratio (OCR)

also influences the volume increase of the soil after the excavation. A high OCR results in a higher swelling factor compared to normally consolidated soil.

During excavation the soil's trafficability, hardness, volume increase, boulder content etc. are important factors that influence the efficiency of the earthwork. Five different problems in the excavation of glacial tills have been identified by Trenter (1999). These are:

- misidentification of rockhead
- presence of large boulders
- water-bearing silts and sands
- water-bearing bedrock
- selection of appropriate plant

Of these problems the presence of large boulders and water-bearing silt and sands are identified as most important for this study. However, selection of appropriate plants for the earthwork could also be critical for the quality of the fill. In addition, water-bearing bedrock could cause uplift of the soil as well as increasing the soil's water content during excavation.

A system for classification of the excavability of soils has been presented in a Swedish study (Magnusson and Orre, 1985). The soils were divided into five different classes depending on rip resistance, see Figure 3.2.

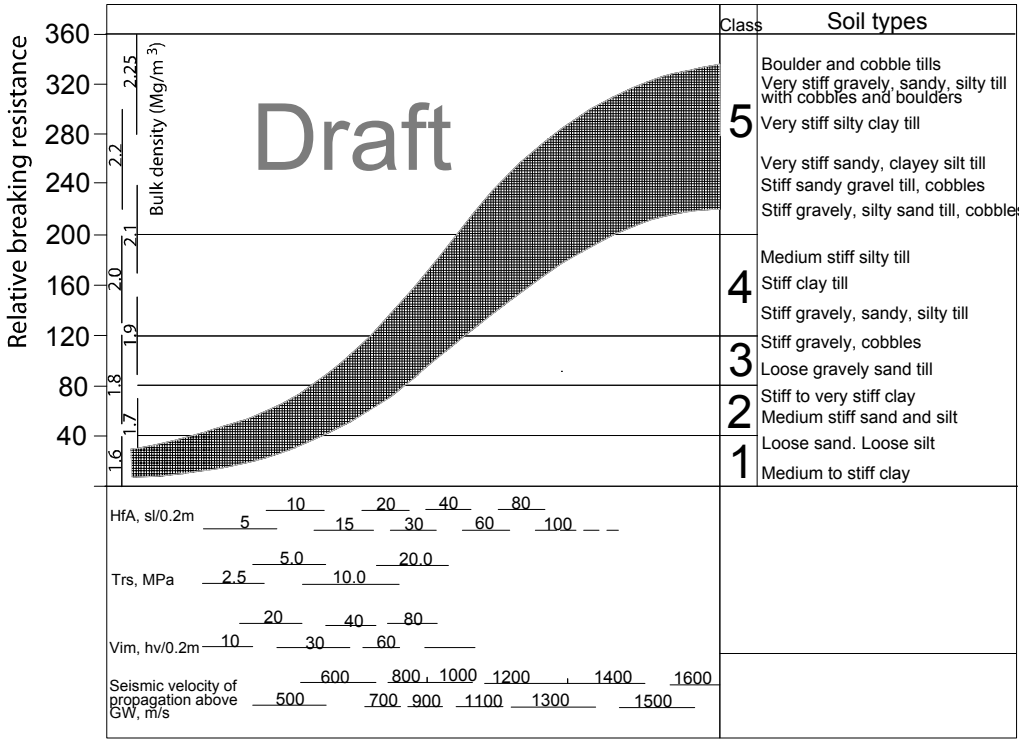


FIGURE 3.2 *Excavateability-classification according to Magnusson and Orre (1985).*

3.3.2 Fill acceptability and control

Snedker (1973) identified three different critical factors to determine if a material is useable for an embankment. These are:

- Can a plant run on it?
- Will any bank constructed be stable?
- Will excessive settlement take place?

The two first criteria are dependent on the soil's shear strength.

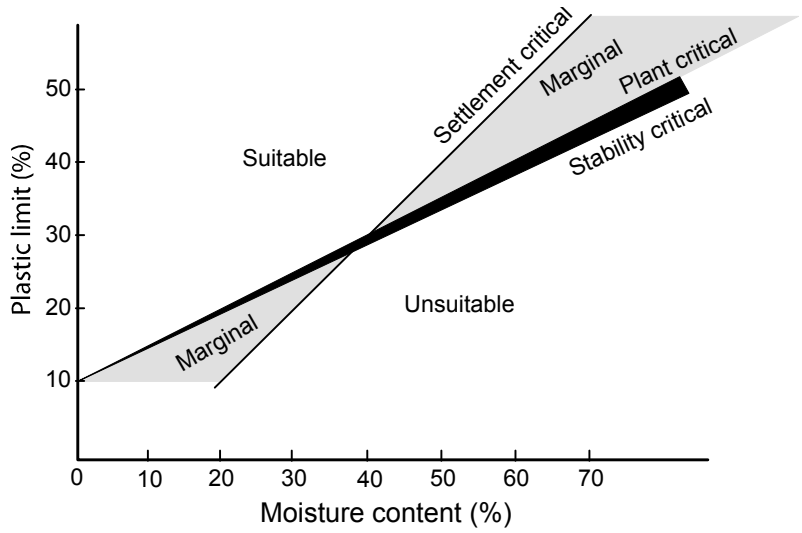
The three criteria for acceptance have been formulated in terms of water content and plastic limit (Snedker, 1973). These are summarised here:

- For a plant, $mc = 2 \cdot w_P - 21$
- For stability, $mc = 2 \cdot w_P - 21$ (for a 10 m bank)
  and $mc = 2.16 \cdot w_P - 25$ (for a 6.7 m bank)
- For settlement, $mc = w_P + 2$ (for a 10 m bank)
  and $mc = w_P + 9$ (for a 6.7 m bank)

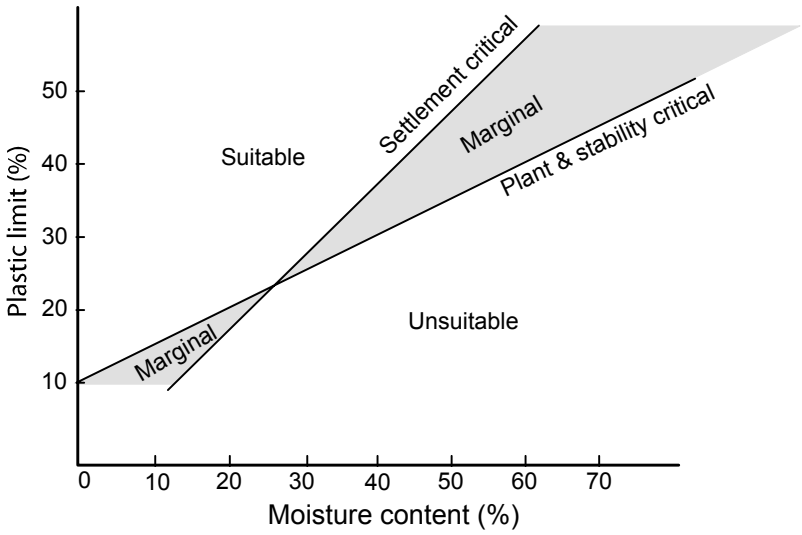
where:

mc = water content, %
w_P = plastic limit, %

The lines above are plotted in Figure 3.3 for the two embankment heights. Subplot A shows the 6.7 m bank and subplot B shows the 10 m bank.



A - Limits for a 6.72 m bank with 1 in 2 slopes.



B - Limits for a 10 m bank with 1 in 2 slopes.

FIGURE 3.3 *Soil acceptability related to plastic limit and on natural water content and for different embankment heights. (After Snedker, 1973).*

The marginal soils in Figure 3.3 could be used by stabilising with the addition of lime or cement or by sandwiching with granular material (Snedker, 1973).

In those cases when the fill is intended to be used as an impermeable structure, some extra control might be necessary.

According to Jones *et al.* (1995) the acceptability in earthworks is related to the excavation, handling, trafficability and compaction characteristics of the material to achieve a low permeability barrier. However, the overriding requirement for materials used as landfill lining is the permeability. Since permeability tests are time-consuming they suggested that a relationship should be established between permeability and some soil properties that enable control of earthworks-operation by a more rapid acceptability test, such as MCV.

Murray *et al.* (1996) investigated the permeability criteria for a low-plastic clay. They found that soils with MCV between 7 and 16 meet the permeability requirements for a landfill lining, see Figure 3.4. To achieve the desired permeability, the soil must be compacted at the wet side of the optimum water content, cf. Figure 3.4.

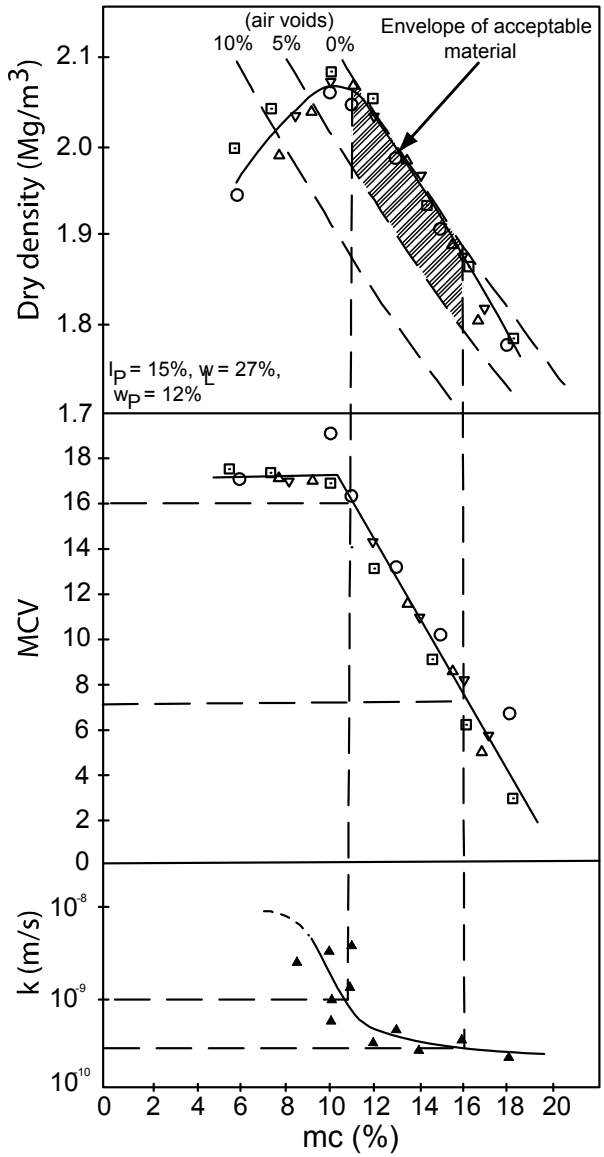


FIGURE 3.4 *Permeability for MCV compacted specimens of a low-plastic clay. (After Murray et al., 1996).*

The acceptability criteria for fine-grained tills are usually set in relation to the engineering properties that affect or control the volume change-characteristics of the compacted fill (Trenter, 1999). The most important criteria are:

- particle size distribution
- water content and plastic limit
- California bearing ratio (CBR value)
- undrained shear strength
- compaction characteristics
- moisture condition value (MCV)

Jones and Greenwood (1993) identified other factors that also may limit the use of fine-grained soils as fill material, such as:

- earthwork balance
- availability of import
- susceptibility to water content change
- frost susceptibility

The acceptability criteria are often assessed at the design stage of a project, based on site investigation and experience. The control process is performed during the earthwork. However, sometimes the acceptability criteria have to be reconsidered owing to the actual conditions during the construction stage. As an example, the original specification for the core at Cow Green Dam was based on water content and the modified specifications were based on undrained shear strength from 100 remoulded samples (Kennard *et al.*, 1979).

Typical limits for soil acceptability have been proposed to lie in the ranges according to Table 3.3.

Table 3.3: *Typical limits for soil acceptability (Jones and Greenwood, 1993).*

	Wet	Dry
c_u[a]	30-50 kPa	150-200 kPa
MCV	7 - 9	12.5 - 15

a. c_u = undrained shear strength

In earlier British practice, the upper limit of acceptable water content was often specified based on the plastic limit, w_P. It was then specified as $1.2 \cdot w_P$. The plastic limit is measured on the fraction of soil passing a 425 μm sieve (400 μm in Sweden). However, since tills contain significant portions of coarser materials there are fundamental difficulties in applying this method to these soils (Jones and Greenwood, 1993).

Jones and Greenwood (1993) tested alternative methods of specifying the upper water content, the wet limit, and they introduced four different water content parameters, see Table 3.4.

Table 3.4: *Alternative methods of specifying the soil's wet limit (Jones and Greenwood, 1993).*

Criteria	Associated limiting water content
MCV ≤ 8	w_{m8}
CBR[a] ≤ 2	w_{c2}
c_u[b] ≤ 50 kPa	w_{s50}
I_L[c] ≥ 0.15	w_{IL15}

a. California Bearing Ratio (CBR).
b. Undrained shear strength.
c. Liquidity index.

Linear regressions between the various limits in Table 3.4 from tests on some soils are shown in Figure 3.5.

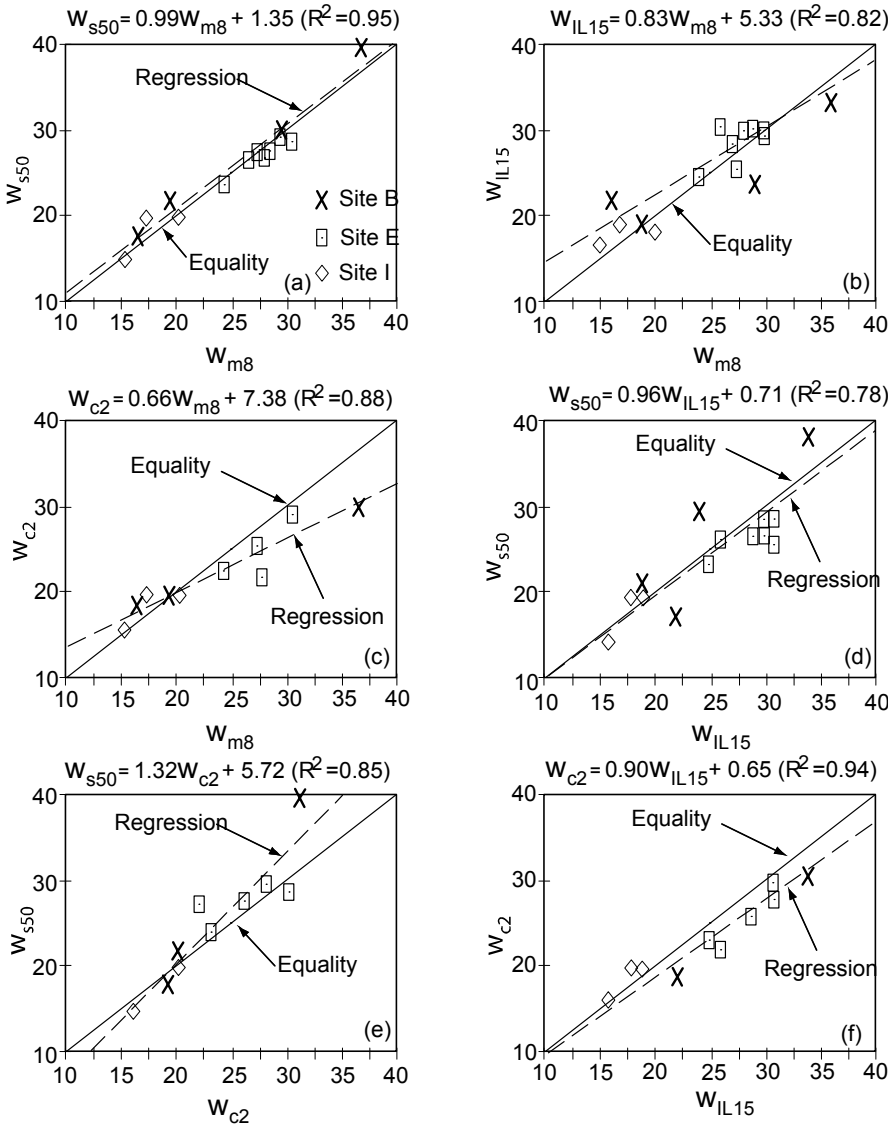


FIGURE 3.5 *Relationships between the upper water content limit set by various criteria for a sandy silty clay. (After Jones and Greenwood, 1993).*

The results in Figure 3.5 show that the regression between undrained shear strength (c_u) and MCV has the strongest correlation followed by

CBR and c_u correlated against liquidity index. The regression between shear strength and liquid limit shows the weakest correlation.

In many parts of the world, CBR (California Bearing Ratio) is used as an acceptance criterion for road materials as well as for sub-grade material. The CBR method is not employed in this study since it is not used in Sweden. As a guide for those readers who are more familiar with CBR than with undrained shear strength, an empirical relation is given in Equation 3.1 (Black and Lister, 1979).

$$CBR \approx \frac{c_u}{23} \qquad \textbf{(EQ : 3.1)}$$

where:

CBR is in per cent
and
c_u is in kPa.

Black and Lister (1979) suggested that Equation 3.2 is a better correlation due to the soil's ability to cohere even when it is immersed in water and the suction is zero.

$$c_u \approx 23 \cdot CBR + 1 \qquad \textbf{(EQ : 3.2)}$$

Jenkins and Kerr (1998) suggested a relationship between c_u and CBR as:

$$c_u \approx 27.15 \cdot CBR^{0.586} \qquad \textbf{(EQ : 3.3)}$$

This equation has a correlation coefficient of 0.953, which is very good. Black and Lister (1979) also found a linear relationship between undrained shear strength and water content, which will be discussed later in chapter 7.

Brown *et al.* (1987) found that the undrained shear strength c_u can be related to CBR by Equation 3.4.

$$c_u \approx 7.8 \cdot CBR \qquad \textbf{(EQ : 3.4)}$$

The correlation in Equation 3.4 was obtained between results from CBR and shear vane tested on saturated anisotropically overconsolidated specimens. Brown *et al.* (1987) also found that undrained shear strength measured by the pocket penetrometer and CBR tests on compacted samples had the following relation, see Equation 3.5.

$$c_u \approx 34 \cdot CBR \qquad \textbf{(EQ : 3.5)}$$

The different relations between shear strength and CBR are shown in Figure 3.6. Four of the relations are linear while the relation presented by Jenkins and Kerr (1998) is a power relation. Since CBR could be considered as a strength parameter a linear relation is more likely to be correct.

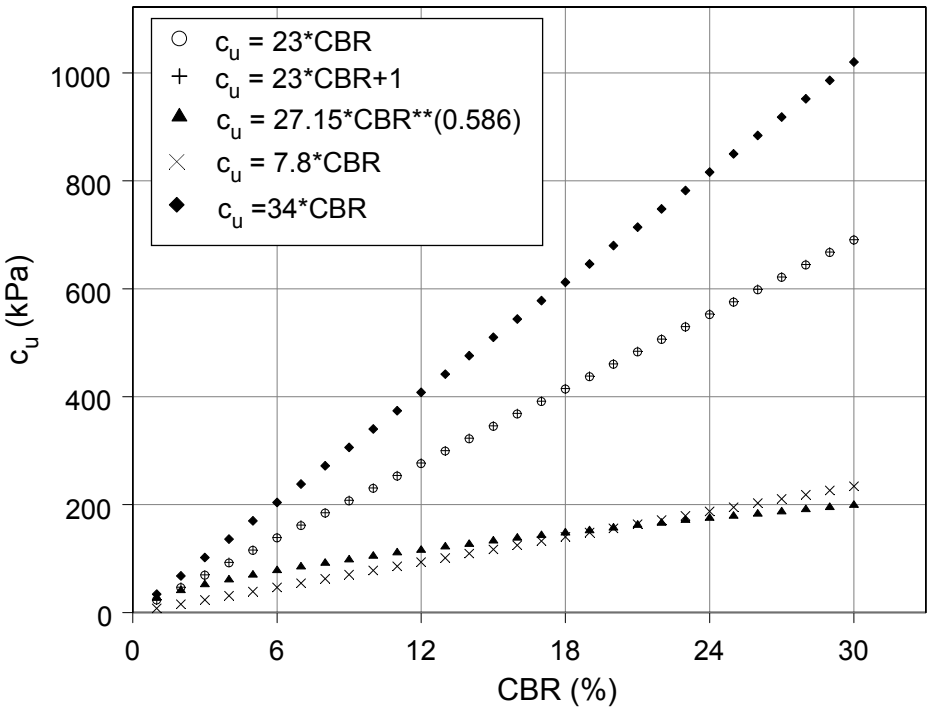


FIGURE 3.6 *Undrained shear strength as a function of CBR.*

3.3.2.1 Water content and plasticity

Compared to coarse materials, fine-grained tills are extremely sensitive to wetting and drying owing to the low permeability and plasticity. Coarse-grained soils have high permeability that permits excess water to drain sufficiently fast from the soil. In a fine-grained soil, the permeability is low and the soil is thereby harder to dewater. The soil's sensitivity to wetting and drying is determined by the amount of fines in the soil. However, the silt fraction has a greater importance for the water sensitivity compared to the clay fraction (Lindh and Winter, 2003).

The relation between MCV and water content is expressed by Equation 3.6.

$$w = a - b \bullet MCV \qquad \textbf{(EQ : 3.6)}$$

Where:

- w is the water content (%)
- a, b soil-dependent factors

Parsons (1981) made an attempt to correlate the potential change in MCV with the intercept (a) in the MCV calibration line, see Figure 3.7 and Figure 3.8.

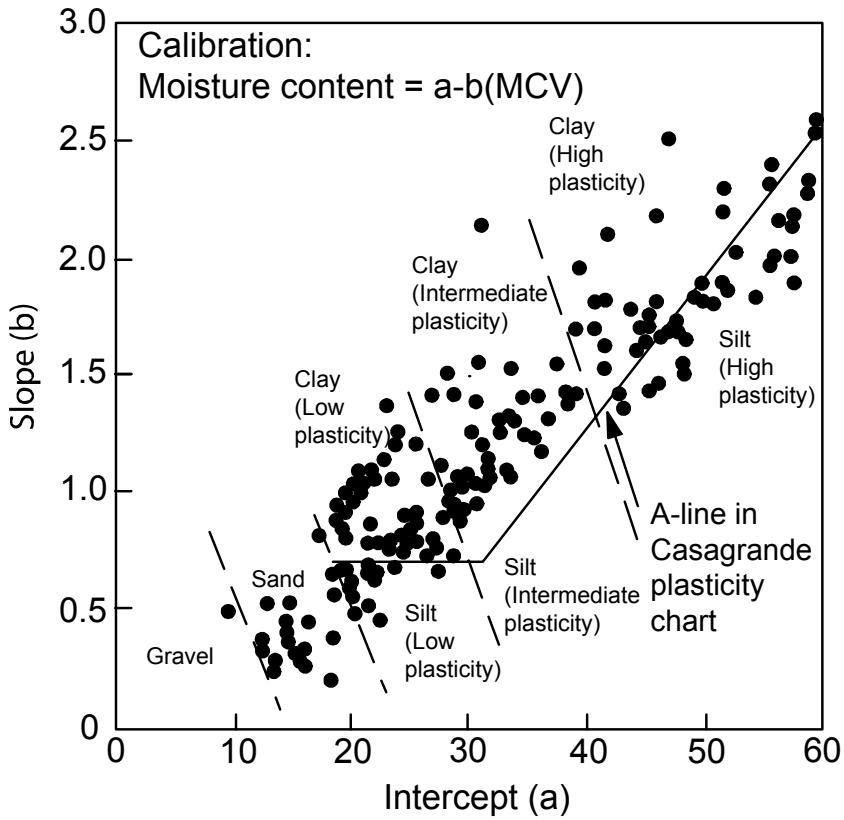


FIGURE 3.7 *Tentative soil classification based on the slope and intercept of the moisture condition calibration, with results from numerous calibrations. (After Parsons, 1981).*

Figure 3.8 illustrates both the deterioration of the compaction properties of the materials during wet weather conditions and the improvement in dry weather conditions. The greatest improvement of the compaction properties during dry weather occurs in soils with low intercepts (a value). The most sensitive soil to wetting has an intercept between 25 and 50.

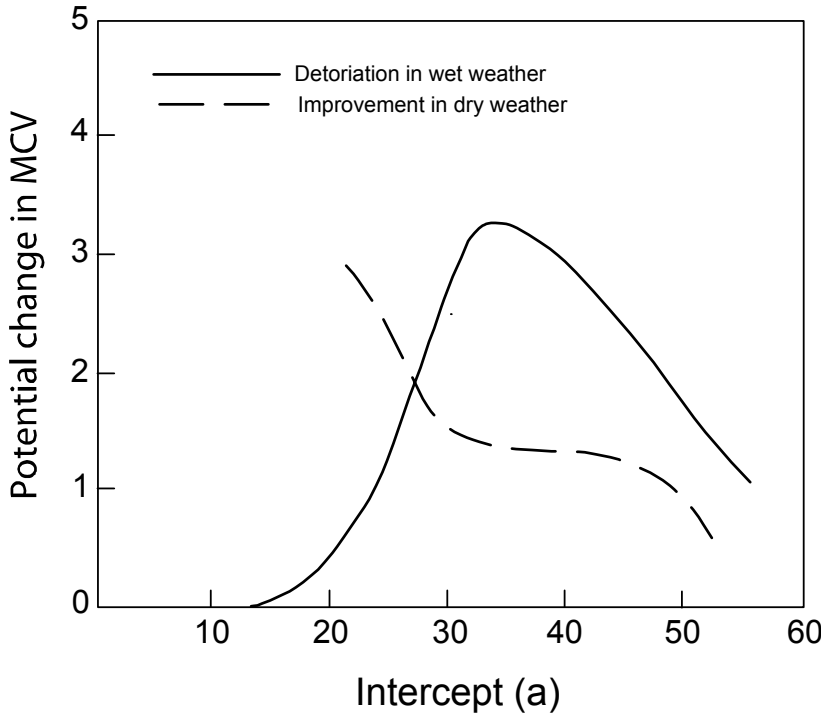


FIGURE 3.8 *Schematic relations between the potential change in MCV and the intercept a in the MCV calibration during wetting and drying. (After Parsons 1981).*

Another method to determine the soil's sensitivity to wetting and drying has been developed by Matheson and Winter (1997). This method is also based on MCV where the slope of the MCV calibration line and the intercept (a) at MCV = 0 is used to classify the soil's sensitivity, see Figure 3.9.

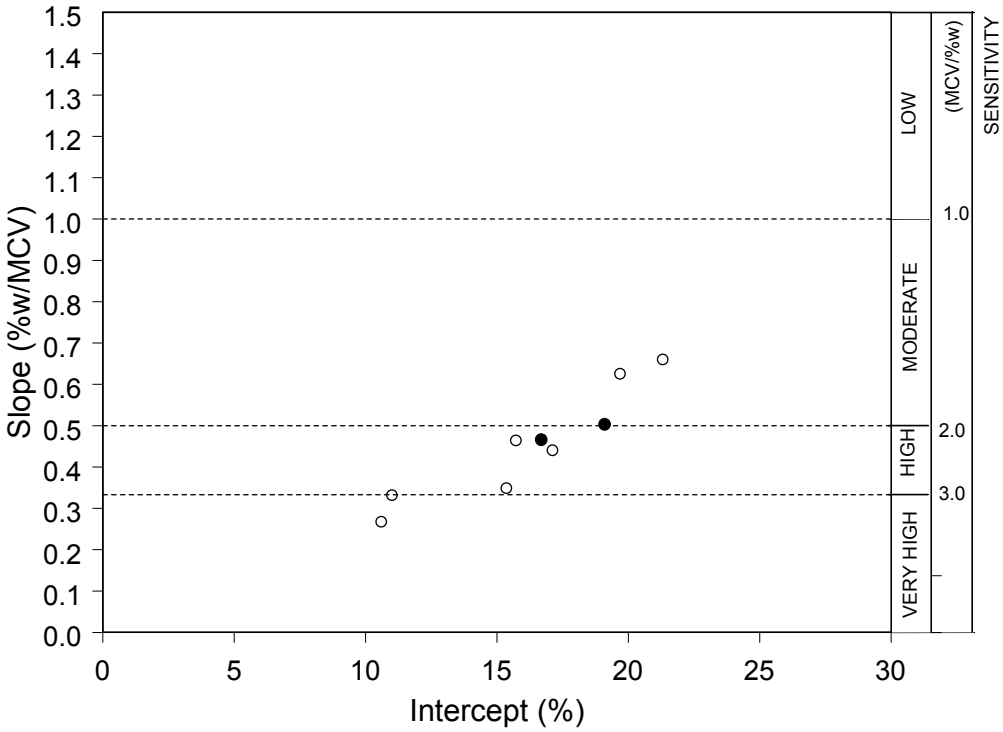


FIGURE 3.9 *Plot of MCV calibration line slopes versus intercept (a). After Matheson and Winter (1997).*

Smith *et al.* (1998) composed a model to forecast the soil's acceptability regarding the changes in the in situ water content. The aim was to make an advisory system for earthwork operations. Their long-term model can be used to forecast the mean weekly MCV for the 52 weeks of the year and thereby assist the designer in forecasting the likely acceptability of the soil: This means that the designer can choose the most favourable period for earthwork and minimize the use of imported fill. The input data consists of soil suction, surface vegetation, metrological data and traditional ground investigation.

When a fine-grained soil is too wet in its natural state or has become too wet owing to weather conditions or handling, it needs treatment to become acceptable. Grace and Green (1979) identified three main methods of treating wet fill material.

- By stockpiling

  A treatment that is applicable to self-draining material with a small percentage of fines.
- By aeration

  This type of treatment can be used for both granular and plastic materials. It is time-consuming and can only be used during favourable climatic conditions.
- By consolidation with time, with or without surcharge.

  This method is used to reduce the excess pore pressure and can be combined with sand drains and horizontal drainage layers, cf. Figure 3.1.

In this thesis, yet another method is proposed.

- By modification/stabilisation

  This method can be used on wet fine-grained soils to change the plasticity and also to reduce the water content if necessary. Modification/stabilisaton improves the soils compaction properties owing to agglomeration of the fines and thereby this also improves the treated soils shear strength. This method involves a great reduction in the soils consolidation time.

3.3.3 Trafficability by earth-moving plants

Modern earthworks are constructed with various types of plants and it is essential for the success ot the earthworks to be able to predict the soils trafficability by earth-moving plants.

All activity by earth-moving plants needs a soil with a certain strength to carry the load of the equipment. The minimum required soil strength depends on the type of earth-moving plants. Varying values of the minimum strength can be found in the literature for different types of earth-moving plants. Parsons (1979) made a compilation of the required value of undrained shear strength from various sources, see Table 3.5.

Table 3.5: *Minimum values of undrained shear strength for operation of various types of earth-moving plants (After Parsons, 1979).*

Type of plant	Method of measuring c_u (kPa)				
	MCV/ hand vane	Cone index / ucs relation	Un-specified [a]	Field vane and triaxial tests[b]	Triaxial tests
Bulldozer with extra low contact pressure	-	15-20	-	-	-
Bulldozer with low contact pressure	-	20-40	-	-	-
Small tracked plant	-		-	-	30
Ordinary bulldozer	-	40-70	-	-	-
Towed scraper	-	50-70	-	-	40
Medium twin-engined scraper	25-40	-	35	-	-
Twin-engined scraper	-	40-50	-	-	-
Medium sinle-engined scraper	70-110	-	-	-	-
Self loading scraper	-	≥100	-	-	-
Large rubber-tyred scraper	-		50	-	-
Scraper tyre pressure 340-380 kPa	-	-	-	60-80	-
Scraper tyre pressure 240-310 kPa	-	-	-	40-60	-
Medium/heavy plant	-	-	-	-	60
Heavy motorized scrapers	-	-	-	-	100
Dump truck	-	≥100	-	-	-

a. Arrowsmith (1979), see Figure 3.10.
b. Dennehy (1979), see Figure 3.11.

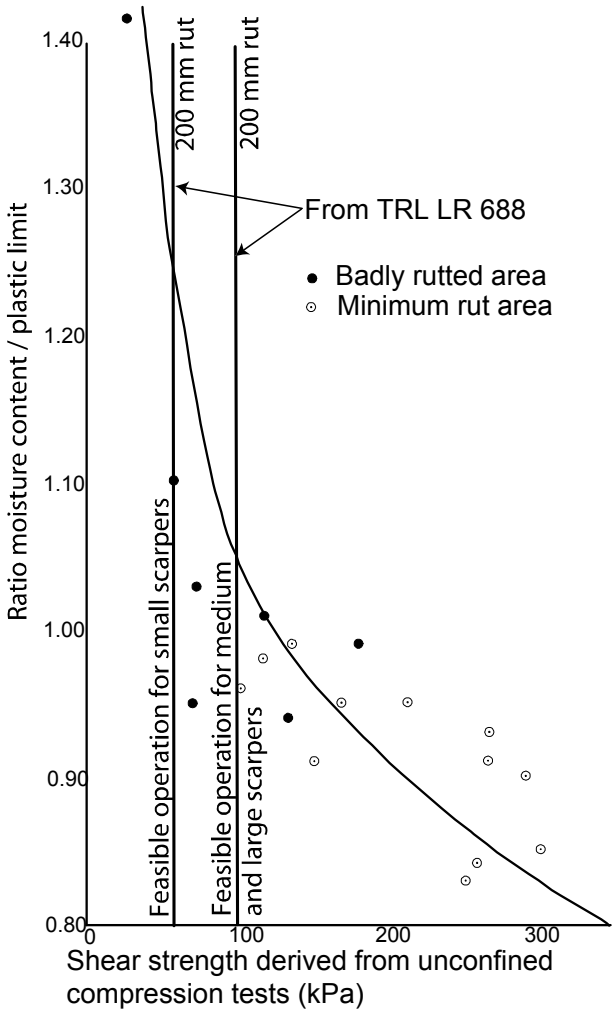


FIGURE 3.10 *Trafficability of different scrapers in relation to soil strength and ratio water content/plastic limit. (After Arrowsmith 1979) Original data from Farrar and Dareley (1975).*

The results presented in Figure 3.10 show that the required shear strength for a small scraper is approximately 60 kPa and a large scraper requires approximately 100 kPa. This means that the ratio water content -

plastic limit should be ≤ 1.25 for a small scraper and ≤ 1.05 for a large scraper to keep the rut below 200 mm.

The results presented in Figure 3.11 show that an undrained shear strength of 40 kPa gave a rut of 275 mm for a light- to medium-towed plant with a tyre pressure of 240 to 310 kPa. For a medium- to heavy-towed plant with a tyre pressure of 340 to 380 kPa a rut of 275 mm was obtained for an undrained shear strength of 60 kPa. The undrained shear strength in Figure 3.11 was obtained with both vane tests and triaxial tests.

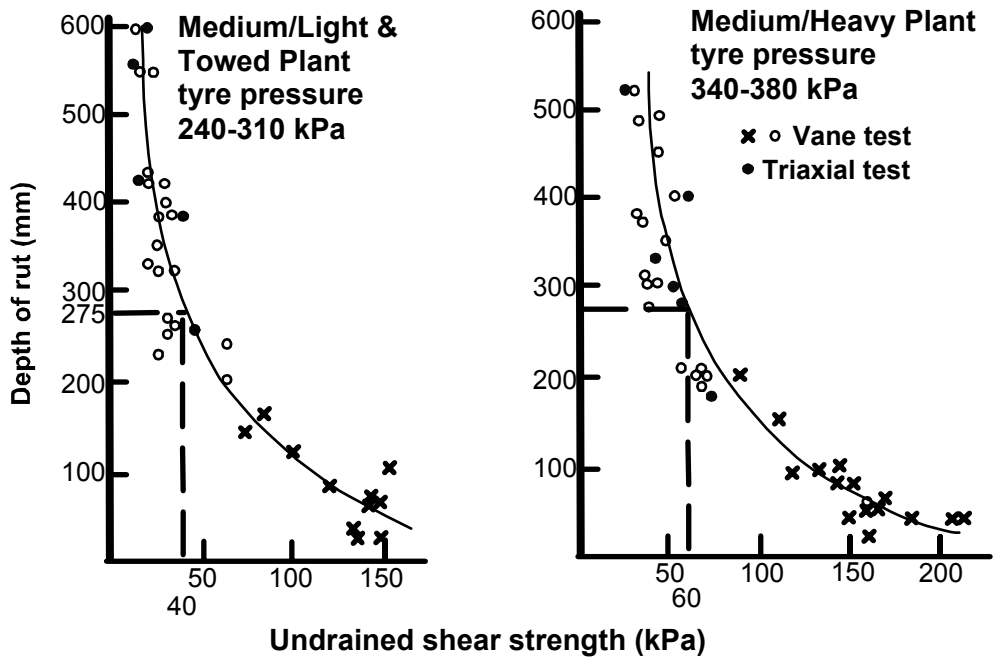


FIGURE 3.11 *Rut as a function of shear strength for earth-moving plants. After Dennehy (1979).*

Parsons (1981) compiled minimum MCV for the optimisation of various types of earth-moving plants, see Table 3.6.

Table 3.6: *Minimum moisture condition values of the soil for the effective operation of various types of earth-moving plant. (After Parsons, 1981).*

Type of plant	Minimum MCV[a]
Twin-engined scraper	6-9
Single-engined scraper	8-11
Dump truck - 3-axle, rigid chassis, struck capacity < $15m^3$	8.5-9.5
Dump truck - 2-axle, rigid chassis, struck capacity $15\text{-}25m^3$	10-12
Dump truck - 3-axle, articulated chassis, struck capacity < $15m^3$	5-7

a. Factors affecting values within the range given are: wheel load; wheel diameter; tyre width; number of driven wheels.

It is important to find a fast and reliable criterion for the trafficability limit. Parsons and Toombs (1988) studied the trafficability of soil by earth-moving vehicles. The study included seven different vehicles and three different soils. The trafficability was evaluated for a compacted soil and a loose soil (200 mm-thick loose tilth). Three response variables were measured; Depth of wheel rut, rolling resistance and wheel slip, see Figure 3.12.

Parsons and Toombs (1988) found that for a 6-wheel articulated dump truck the lower MCV limit was approximately 6.5 with 6-wheel drive and approximately 7.5 with 4-wheel drive. This is a small increase in required MCV compared to earlier recommendations, cf. Table 3.6.

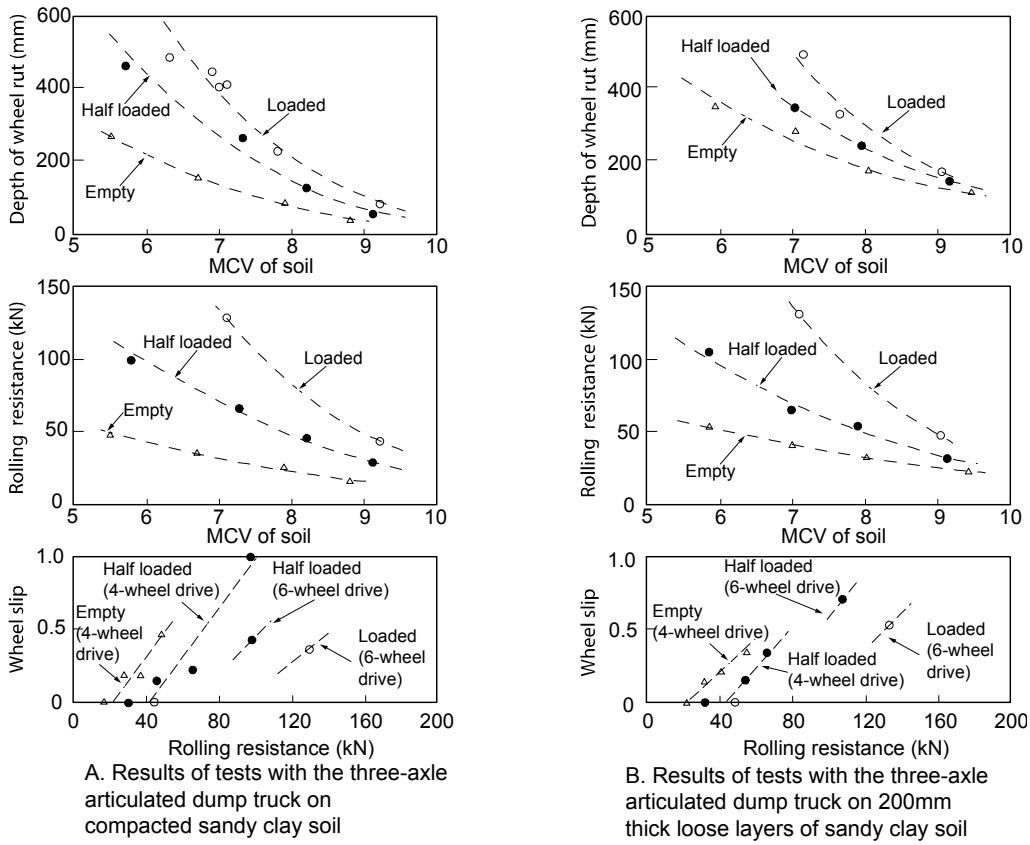


FIGURE 3.12 *Test results from an articulated dump truck on a sandy clay soil. (After Parsons and Toombs, 1988).*

Figure 3.12 shows that the depth of rut as well as rolling resistance decreases rapidly from MCV 6 to MCV 9. This shows that the efficiency of the earthwork could gain considerably if the MCV could be kept at 9 or higher.

3.3.4 Compaction properties

The soil's compaction properties depend on its particle size distribution and water content. The possibility of densification of a fine-grained till by compaction depends heavily on the water content. In a fine-grained soil with a high water content the compaction results in a pore pressure increase. Since the fine-grained soil has a low permeability, the pore pressure decreases only slowly and no further densification is possible before a significant portion of the pore pressure has disappeared. Another factor that affects the soil's compaction properties is the occluded air (Olson, 1963; Langfelder *et al.*, 1968). This factor is dependent on the air permeability of the soil, se Figure 3.13.

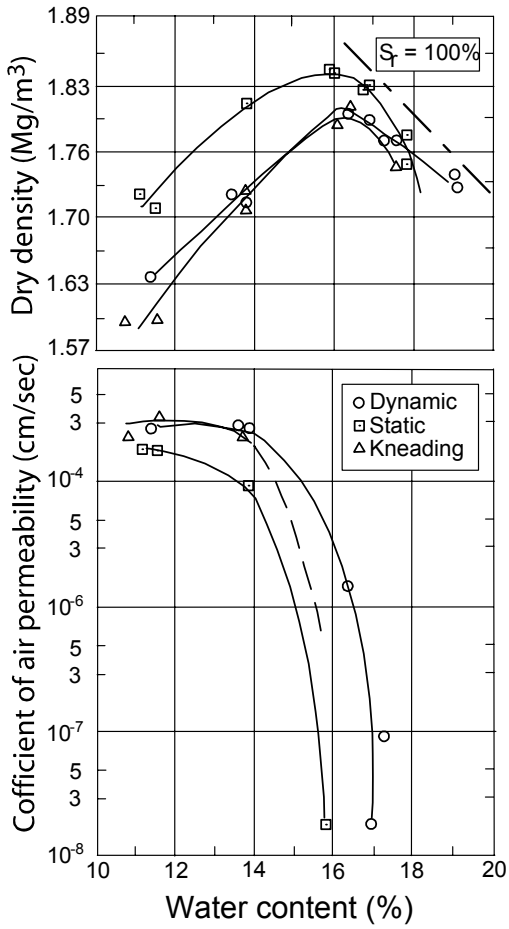


FIGURE 3.13 *Air permeability in a clayey sandy silt. (After Langfelder et al., 1968).*

The effect of the compaction does not only affect the compacted layer. It has also a great effect on the next layer in the construction (Valerux and Morel, 1980), cf. Figure 3.14.

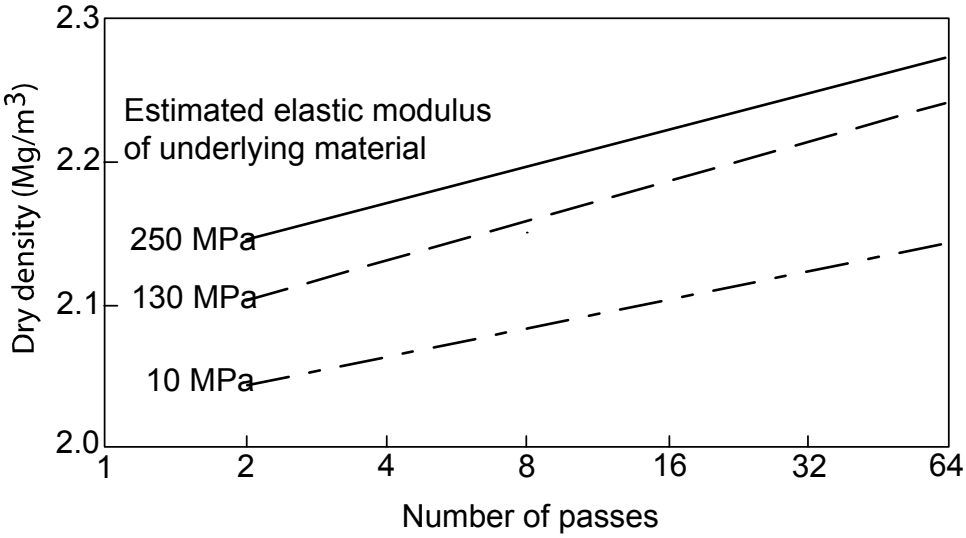


FIGURE 3.14 *Obtained dry density as a function of number of passes with a vibrating roller on crushed gravel (After Valeux and Morel, 1980).*

The underlying materials in Figure 3.14 were cement-stabilised gravel (E = 250 MPa), a natural sandy gravel (E = 130 MPa) and a low plastic silt (E = 10 MPa). This shows the importance of a stiff formation level.

Frost *et al.* (2001) also showed the importance of the supporting layer. They found that the thickness of the underlying foundation has a moderate increase in dry density of the subbase. However, the thickness of the underlying layer has a well-defined trend of increase in composite stiffness for progressively thicker foundations.

4 Binders

4.1 Inorganic binders

This study has been limited to inorganic binders. However, some organic additives were also tested, mainly in combination with an inorganic binder.

4.1.1 Lime

Lime is a broad term that encompasses quicklime, (calcium oxide, CaO), hydrated lime, (calcium hydroxide, $Ca(OH)_2$) and carbonate of lime (calcium carbonate, $CaCO_3$). Hydrated lime is prepared from quicklime, which in turn is prepared from calcium carbonate. The following equations describe the relations between the three different types of lime:

$$CaCO_3 + heat = CaO + CO_2 \quad \textbf{(EQ : 4.1)}$$

$$CaO + H_2O = Ca(OH)_2 + heat \quad \textbf{(EQ : 4.2)}$$

$$Ca(OH)_2 + CO_2 = CaCO_3 + H_2O \quad \textbf{(EQ : 4.3)}$$

Calcium carbonate is used as raw material for the production of quicklime, but can also be used as an additive to adjust pH for agricultural purposes. If quicklime is produced from impure calcium carbonate that contains clay, hydraulic lime, which contains less calcium, is formed. However, the hydraulic lime then contains reactive silicates and aluminates similar to those in Portland cement, cf. hydraulic moduli (Sherwood, 1993). Besides being calcitic, lime can also be magnesian or dolomitic. Calcitic lime is preferred for soil stabilising purposes since it contains most free calcium (Little, 1995).

For soil stabilisation purposes, lime can be used either as quicklime, hydrated lime or as slurry lime. The advantages and disadvantages of these three additives are discussed below. Quicklime is the most economical lime since it contains approximately 98% available CaO for reaction with the soil particles, compared to 73% in hydrated lime.

These values are theoretical and in field conditions the hydration process is not totally efficient. Little (1995) considers that both lime types are equal sources for CaO. Since quicklime uses water in the slaking process its most efficient use is to reduce excess water. The same slaking process also generates heat, which is beneficial for the chemical reactions and also reduces excess water by evaporation. Compared to hydrated lime, quicklime is denser, requires less transport space and produces less dust.

Some of the advantages of quicklime have also been shown to be disadvantages under some conditions. Since quicklime consumes water in the slaking process, dry soils need to be sprinkled. Another disadvantage

is that the slaking process for quicklime is less efficient and this could lead to heterogeneous properties in the stabilised soil. In industrial slaked lime, all quicklime is reacted to hydrated lime. When the quicklime is mixed with soil, the slaking process is dependent on the soil water. Quicklime is also more harmful to the site personnel's health.

Hydrated lime is quicker to apply than slurry lime and is also more efficient to dewater soils. Soil stabilisation with hydrated lime generates a lot of dust that can be a problem, especially in urban areas. Hydrated lime, like quicklime, may need the addition of water during hot and dry weather conditions. Slurry lime produces no dust and minimises the drying action but is less usable with wet soils. The application rate is slow for slurry lime and special equipment is required, such as special tank trucks (McDowell, 1972; Anon., 1987; Little, 1995; Greaves, 1996).

In the US, the hydrated lime is more common than quicklime as a stabilising agent. However, of the total amount, quicklime accounts for more than 10% of the lime used for soil stabilising purposes according to Little (1995) and for 25% according to TRB (Anon., 1987). The figures for hydrated lime are not available from these sources.

4.1.2 Cement

Heating limestone and clay to a temperature of about 1450°C produces Portland cement clinker. Portland cement clinker is composed of four major components, of which alite is the most important. It constitutes 50 - 70% of Portland cement and the chemical formula is tricalcium silicate (Ca_3SiO_5). Alite is also the most important cement component for strength development during the first 28 days after reactions with water. It reacts quickly with water.

Belite is the second largest component in Portland cement. It constitutes 15 - 30%. Belite, which is dicalcium silicate (Ca_2SiO_4), reacts slowly with water and its contribution to the strength is also slow. The strength after one year is approximately the same for pure alite and pure belite.

Aluminate makes up 5 - 10% of Portland cement. The component is tricalcium aluminate ($Ca_3Al_2O_6$) and reacts rapidly with water. This rapid reaction can cause a too rapid setting. Calcium sulphate (gypsum) is therefore used as a set-controlling agent.

Ferrite constitutes 5 - 15% of Portland cement. It is in the form of tetra calcium aluminoferrite (Ca_2AlFeO_5) and has an initially a high reaction rate that becomes very low with time.

The hardening process is a result of the reactions between the major components and water (Taylor, 1997). The chemical reactions between cement, water and soil are described below in the binder - soil reaction section, see chapter 5.

A certain performance is expected of a cement-stabilised soil regarding strength, permeability and durability characteristics. However, pavements treated solely with cement have been found to have some limitations. Cement-soil mixes have a short working time (typically less than 2 hours) and this often prevents a satisfactory compaction. Cement-treated soil also attains too high strength, which results in shrinkage cracking with reduction in durability (Vorobieff, 1997). Rodriguez *et al.* (1988) advocated that the minimum and maximum unconfined compressive strengths should be 1.7 MPa and 5.4 MPa respectively. If the

maximum strength is exceeded, no flexibility remains in the layer and it becomes brittle.

4.1.3 Blast-furnace slag

Blast-furnace slag is a waste product from the steel industry. Lime is used to reduce the silica in the melted ore. The limestone reacts with material rich in SiO_2 and Al_2O_3 at a temperature between 1350-1550 ^{o}C. If the slag is cooled rapidly to 800 ^{o}C, it forms a glass, which is a latent hydraulic cement. This type of slag is named ground-granulated blast-furnace slag (GGBFS) (Sherwood, 1993; Taylor, 1997). The chemical composition of some blast-furnace slags is presented in Table 4.1 (Taylor, 1997).

Table 4.1: *Chemical composition of different GGBFS from France and Luxembourg. (After Taylor, 1997).*

Substance	Mean (%)	Minimum (%)	Maximum (%)
Na_2O	0.39	0.25	0.50
MgO	5.99	3.63	8.66
Al_2O_3	13.29	10.26	16.01
SiO_2	33.48	31.96	37.29
P_2O_5	0.13	0.00	0.19
SO_3	0.04	0.00	0.19
K_2O	0.70	0.44	0.98
CaO	42.24	37.92	44.38
TiO_2	0.55	0.49	0.65
MnO	0.64	0.34	1.31
FeO	1.24	0.29	9.32
S^{2-}	0.94	0.68	1.25
F^-	0.16	0.06	0.31
Cl^-	0.019	0.003	0.050
Ign. loss	0.42	0.00	1.04
Total	99.68		

The GGBFS particles are rich in glass and are unstable when mixed with water and in the presence of an activator. The activator and the GGBFS react rapidly and form a cementitious material. In Sweden the GGBFS that is commercially avalible has the trading name Merit 5000

,where the name indicates the blain value i.e. the specific surface of the binder. This product is used as a component in blast-furnace cement.

The data in Table 4.1 could be compared to the binders used in this study, cf. Table 4.2. The Swedish slag contains less calcium and more silica compared to the slags from France and Luxembourg.

Table 4.2: *Chemical analyses of binders used in this study.*

Chemical compound	Cement OPC (%)	Bygg cement[a] (%)	Anlägg-ning-cement (%)	Slag (%)	Hydrated lime (%)	Quicklime (%)
CaO ($Ca(OH)_2$)	61.9	61.5	64.6	36	72.9[b] (92)	94
SiO_2	19.8	19	22.8	36	0.8	1.5
Al_2O_2	4.1	3.8	3.5	10	0.6	0.8
Fe_2O_3	2.6	2.7	4.2	0.4	0.2	0.4
MgO	3.4	2.5	0.88	13	1.2	1.5
K_2O	1.3	1.0	0.61	0.6	0.1	0.1
Na_2O	0.3	0.18	0.09	0.4	0.1	-
SO_3	3.6	3.3	2.2	2.5	-	-
Loss on ignition	2.5	-	0.57	-	24	1.2
Cl	0	0.044	0.011	-	-	-
P	-	-	-	-	0.01	-
Sum	**99.8**	**94.0**	**99.5**	**98.9**	**99.9**	**98.3**
Glass content	-	-	-	97	-	-
Specific surface (cm^2/g)	3753	4700	3120	5000	160 000	-

a. Contains 11.7% limestone and 4.6% C_3A.
b. Equivalent value, hydrated lime contains no CaO.

4.1.4 Fly ash

Today there is no energy production from coal-fuelled power plants in Sweden and thereby no domestic fly ash of this kind to handle. However, there are biofuelled power plants that generate fly ash. The fly ash from biofuel contains more organic material and varies more due to the composition of fuel. Fly ash is not studied in this thesis.

4.1.5 Blended binders

A blended binder can consist of two or more inorganic binders. It can also consist of a blend of organic binders and inorganic binders e.g. bitumen and cement.

A French study showed that a heavy, high pozzolanically active clay attained higher compressive strength with a blend of lime and cement compared to one with only cement or only lime (Sherwood, 1993). Sherwood considered that use of a mixture of lime and cement should be confined to a limited number of soils, at least for economic reasons. High-plastic soils that are difficult to mix with cement and do not give a long-term strength increase when treated with lime only are such soils. In France, an often-used road material consists of gravel mixed with 15 - 20% of ground granulated slag from the metallurgical industry and 1% lime. The lime is used as an activator for the slag. This product is called "grave-laitier" and is mixed in plants. It has a very long working period and can be stored for several days without problems. The compacted material is insensitive to rainfall and frost. Grave-laitier is extensively used in France and it is estimated that 65% of the roads in France have a layer composed of grave-laitier (Lee, 1975; Sherwood, 1993). A similar product is used in South Africa. The proportions specified give a ratio of four

parts of ground granulated blast-furnace slag to one part of hydrated lime (Sherwood, 1993).

Hossain *et al.* (1991) compared the properties of an alluvial soil stabilised with cement and cement mixed with rice-husk ash (RHA). The reason for blending cement and rice-husk ash was mainly to overcome a disposal problem arising from the rice husk ash and replace cement with a local material. The rice-husk ash is very rich in silica in amorphous form. The results showed that 25% of the cement content could be replaced with RHA when a total admixture content of 10% was used.

In England, Dumbleton (1962) found that stabilisation with a blend of 5% cement and 5% hydrated lime resulted in lower compressive strength compared with 5% cement for five different types of soils. The soils were sandy gravel, well-graded sand, uniform sand, sandy clay and silty clay. However, in a mixing-time test, a blend of 2% lime and 8% cement outperformed both 10% lime and 10% cement. The latter results were obtained for London Clay.

In Australia binders are divided in three main classes:

- General Purpose (GP) and General Blend (GB) Cement
- Lime
- Blended products

The GB is defined as a cement containing >5% fly ash or ground granulated blast-furnace slag (GGBFS), or both. The GB could also contain up to 10% silica fume. Blended binders consisting of fly ash, blast furnace slag and lime began to be produced by cement manufacturers as a result of the short working time of cement. Generally GB cement provides a 1 to 2 hour longer working period than GP cement.

Blends consisting of fly ash, slag and cement or lime are also available. Up to four different materials are often blended in various proportions at the distribution facilities. One of the main benefits of these multiple blends is the greatly increased working time. The working time can be increased up to 8 hours. This should be compared to GP cement, which has a working period of 2 hours. The blending facilities of today give no limitation in blending binders. Blends utilising cement yield a rapid strength gain within the first 24 hours. In Australia, the use of GP cement has declined as the benefits of using GB cement in soil stabilisation have become evident and been utilised successfully (Vorobieff, 1997).

The hydraulic properties of blended binders are important. A hydraulic binder is defined as a binder which hardens by reaction with water and is resistant to water after reaction. The hardening of the binder takes place both above and below the water table (Anon., 1994a). The hydraulic modulus can be defined as (Taylor, 1997; Regourd, 1985):

$$M = (CaO + MgO + Al_2O_3)/(SiO_2) \qquad \textbf{(EQ : 4.4)}$$

The minimum value for the ratio M in Equation 4.4 should be 1.0 for the binder to be hydraulically active. Slag used for slag-cement purposes should have a modulus of one or above to be accepted according to British standard (Taylor, 1997). Wäre (1974) proposed another definition:

$$M = (CaO + MgO)/(SiO_2 + Al_2O_3 + Fe_2O_3) \qquad \textbf{(EQ : 4.5)}$$

Equation 4.5 shows the hydraulic activity for lime whereas Equation 4.4 describes the hydraulic activity for slag. The ratio in Equation 4.5 should be as low as possible to indicate good hydraulic

property (hydraulically active). Wäre (1974) showed that a lime with low hydraulic moduli (cf. Equation 4.5) should be compacted without any delay. At a delay, the CaO in the binder can react too fast with the hydraulic component and form a cementitious gel. This results in a lower density and a lower strength due to the higher resistance to compaction. For a binder with a high hydraulic modulus, the reaction time depends on the availability of the hydraulic component from the soil (alumina and silica), cf. cement.

4.2 Organic agents

The organic agents are used to change the performance of the stabilised soil. In some cases the agents are used to change the workability of the stabilised soil during construction. Other agents are used to permanently change the properties of the stabilised soil to improve its performance during the service life of the structure.

4.2.1 Agents for improving the workability of a stabilised soil

Stabilised soil requires more compaction energy compared to unstabilised soil to achieve the same dry density. An increased compaction work decreases the production rate.

In the concrete industry, plasticisers are often used to improve the workability of the concrete. In this study, lignosulphonate has been employed as a workability-improving agent. Lignosulphonate reduces the need for water and improves flow ability in the cement slurry. Lignosulphonate is also used in ceramics to improve the workability of the clay, reduce free water and make it possible to use less plastic clays in

manufacturing. The lignosulphonates in the study were manufactured by Borregaard and their trading names are Wafex and Lignobond DD. Due to a change in the production process, the Wafex product will be phased out and will be repaced by Lignobond DD.

Lignobond DD and Wafex also contain some sucrose that acts as a retarder for the bindings in the cement slurry. The retardation of the cement slurry is beneficial to soil stabilisation because of an increase of the working period of a cementitious binder (Lindh, 2002).

4.2.2 Agents for improvement of the durability of a stabilsed soil

Both natural and stabilised soils behave differently if they are saturated or unsaturated. For a stabilised soil as well as for unstabilised soil the unconfined compression strength decreases with increasing saturation. A well-functioning drainage is very important to keep the soil unsaturated. A supplementary method is to make the soil hydrophobic with an agent, see Figure 4.1.

FIGURE 4.1 *A drop of water on a hydrophobed soil. The glas bowl has a inner diameter of 66mm.*

This technique has been employed in Australia, where several soils have been stabilised with a hydrophobic agent. The technique is called stabilisation using dry powdered polymers (DPP). The polymer acts to preserve the dry strength of the soil since it creates a hydrophobic soil matrix (Anon., 2003a).

A hydrophobic agent can be used to improve clay liners that are used for landfill purposes. Another application can be to improve the stabilisation/solidification treatment of a contaminated soil.

5 Stabilisation - modification

5.1 History

Soil stabilisation using lime is a very old technique. There is evidence that the Appian Way, the principal southward road from Rome in classical times (first section built in 312 B.C.), was constructed using lime stabilisation. Unlike lime stabilisation, cement stabilisation is a relatively new technique. It commenced in 1917, when Amies patented an initial procedure for improving soils by mixing them with variable proportions of Portland cement. In the USA, the use of lime as a road-construction material dates back to the 1920s. In 1938, the Texas Highway Department supported a program of rehabilitating state roads by mixing lime with soils with high plasticity. In the 1940s the US military stabilised both runways and roads in Texas and some of these roads are still in service. Both virgin soils and worn-out materials were stabilised with lime. The roads that were stabilised with this technique were often built on an emergency basis during World War II. They were therefore often built with marginal materials and by inexperienced personnel. Many of these roads had failed and the conventional method would have been to remove the old material and replace it with new. This was beyond the

military budget and other techniques had to be used. In this phase, soil stabilisation with lime was tried. The main objectives of renewed investigations of these old roads were to evaluate the durability of lime stabilisation (Lambe, 1962; Kelley, 1988). A field investigation in 1977 showed that several of these roads were still in excellent conditions (Kelley, 1988).

In Sweden the techniques were introduced in 1959. The first rotovator in Sweden was used in the beginning of the 1960s. Several projects, both for road-building and for other construction purposes were carried out during the 1960s. At the beginning of the 1970s, over 4 square kilometres had been stabilised in Sweden for a construction purpose (Assarson, 1975). Due to a change in highway specifications by the Swedish National Road Administration (SNRA) in 1984, the economic incentive for the contractor to perform shallow soil stabilisation disappeared for a time and the technique vanished from the market.

In Australia the first specialised contractor introduced a P&H triple-rotor stabiliser around 1950, see Figure 5.1. This equipment produced a high quality mix and made it possible for a rapid expansion over mainland Australia. During the 1960s, cement stabilisation started to increase in quantity. Today, Australia is considered the leader in stabilisation techniques (Wilmot, 1998).

In Great Britain, cement stabilisation has been used since 1945 (Sherwood, 1968). Lime stabilisation was carried out to some extent in the UK during 1950s and 1960s but was not widely used until the late 1970s. In the period 1956 to 1962, the Road Research Laboratory performed thorough investigations on lime and cement stabilisation.

Lime stabilisation is now used extensively in the UK with approximately half a million cubic metres of soil treated in 1995. It is used when the soil is unacceptably wet or plastic, when the workability needs to be improved, when off-site disposal needs to be avoided and when greater soil strength and stability is required (Greaves, 1996). Soil stabilisation in Great Britain is regulated in Specification for Highway Works, Design Manual for Roads and Bridges Volume 4 (Anon., 1995b).

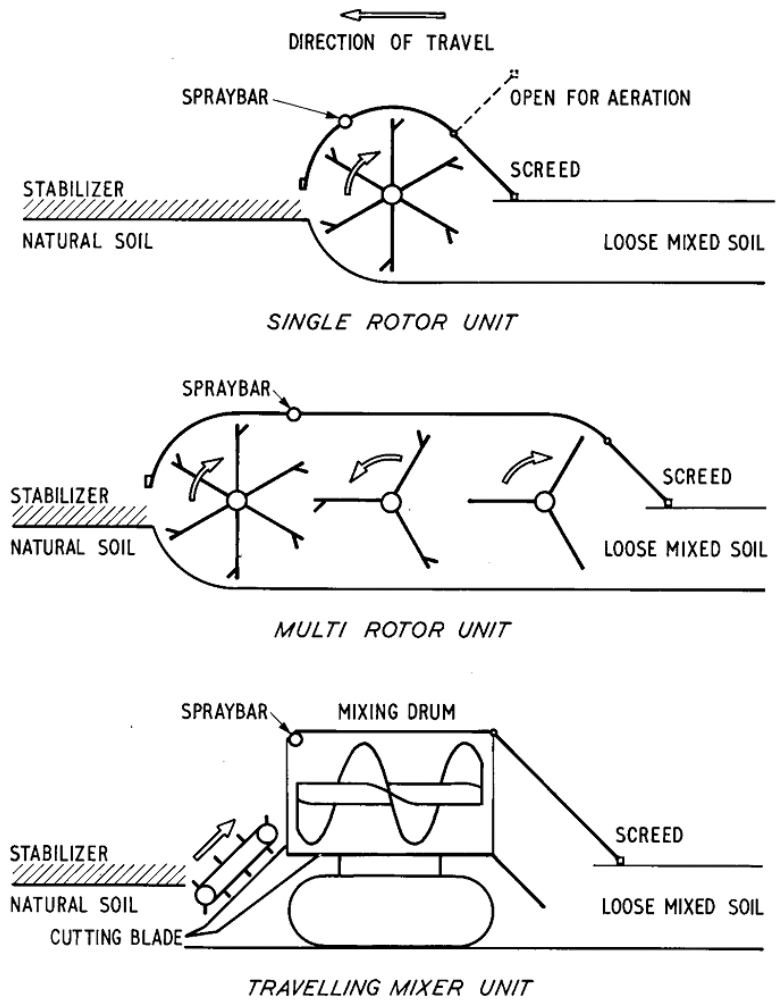


FIGURE 5.1 . *Schematic presentation of different stabilisers' mode of operation (Ingles and Metcalf, 1972).*

5.2 Soil stabilisation today

Today, soil stabilisation is very rational thanks to the developments in soil stabilising equipment that have been taking part during the last ten years. More rational handling and the possibilities to produce blended binders at the binder depots have also increased the efficiency and quality in soil stabilisation. Soil stabilsation can be performed both in situ and at a stabilising depot.

5.2.1 In situ soil stabilisation

In situ soil stabilisation is performed on site to improve and stabilise the site material. Both cuttings and embankments can be stabilised in situ. In some fill cases it is much more rational to stabilise the fill material at the borrow pit before excavation and transportation to the embankment.

5.2.1.1 Binder spreading

The first phase in soil stabilising is spreading the binder on the soil that is to be stabilised. There is, however, new stabilising equipment that allows the binder to be introduced during the milling of the soil, thereby minimizing the dusting. Even so, the most common method today is to use a spreader, see Figure 5.2, which distributes the binder in a controlled way.

FIGURE 5.2 *Binder spreading. The binder is here spread in two layers.*

The binder container is placed on load cells, which means that the driver gets immediate information on the spread rate from the control unit. Placing boxes or geotextiles with known areas on the ground before spreading and weighing the content of the spread binder is also a commonly used method to check the spread rate, see Figure 5.3.

FIGURE 5.3 *Equipment used to check the amount of spread binder.*

In some cases when quicklime is used, it is slaked with water from a tanker, see Figure 5.4. A tanker is also used during dry conditions to increase the soil's water content, in order to meet the stabilised soil's compaction criteria regarding optimum water content (OWC).

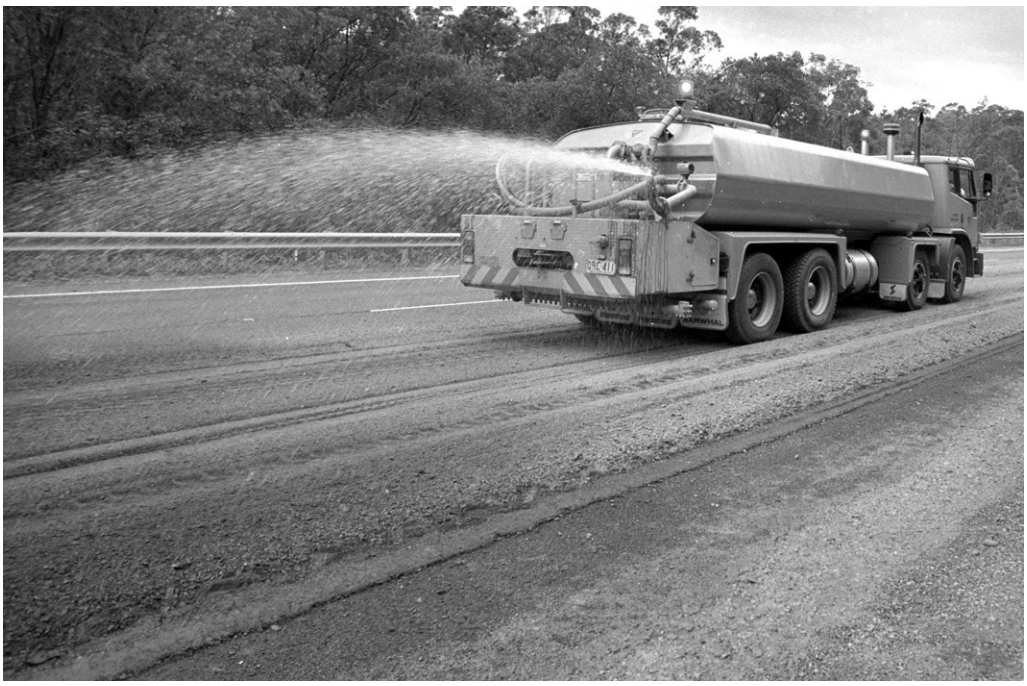


FIGURE 5.4 *Slaking quicklime with water spread from a tanker.*

5.2.1.2 Milling

The second phase in soil stabilisation is the milling. The purpose of the milling is to mix the soil and the binder with a method that produces a homogeneous blend. It has also the purpose of breaking up larger lumps of the soil to achieve a material that has more advantageous grading without lumps and thereby better compaction properties. The milling is performed with large mixing equipment also known as stabilisers, see Figure 5.5.

FIGURE 5.5 *Milling with a CMI stabiliser at Karuha bypass, NSW Australia.*

The milling could be performed in one or more runs. To achieve a good quality in the milling, two runs are recommended (Whilmot, 2000).

5.2.1.3 Compaction

Compaction of a stabilised soil is important to achieve a good quality and to obtain the desired service life of the stabilised material. Stabilisation changes the compaction properties to give a material that needs more compaction energy compared to untreated soil to achieve the same dry density (Dumbleton, 1962; Kézdi, 1979; Littleton *et al.*, 1988; Sivapullaiah *et al.*, 1998; Lindh, 2001).

Compaction of a 300 mm stabilised layer needs a two-phase compaction to achieve good compaction of the lower part of the layer. The first phase is performed with a pad foot roller since the pad foot applies the compaction energy, not only to the top of the layer but also to the lower part of the layer, see Figure 5.6.

FIGURE 5.6 *Compaction with grid roller followed by a pad foot roller.*

In Australia, a pre-compaction run is sometimes performed with a grid drum roller to secure that no soil lumps remain before the pad-foot-compaction, see Figure 5.6. The second compaction phase is performed with a smooth drum roller to compact the top part of the stabilised layer.

After compaction the stabilised layer is trimmed with a grader to achieve the correct level, see Figure 5.7.

FIGURE 5.7 *Levelling the stabilised soil with a grader.*

In some cases the smooth drum roller compaction is exchanged for, or combined with, a rubber-tyre roller to achieve the right finishing, see Figure 5.8.

FIGURE 5.8 *Multi-tyre roller. This type of roller is used to give the stabilised layer the last smoothing.*

5.3 Binder functionality

Traditionally lime and cement are the major stabilising agents. Normally, these binders are used separately. Lime is used for clay soils and cement is used for coarser soils. Clay soils are more difficult to stabilise with cement than with lime, owing to the available amount of calcium oxide in the binder. Ingles and Metcalf (1972) describe how different stabilising agents are applicable for different soils. In Figure 5.9 the applicability is rated according to the soil's particle size. Figure 5.9 shows that lime has maximum effect up to a particle size of 0.01 mm. Cement, on the other hand, has a lower limit for maximum efficiency at a particle size of about 0.06 mm, below which the result of the stabilisation may be uncertain. Ingles and Metcalf (1972) also classified how different soil components respond to stabilisation, see Table 5.1.

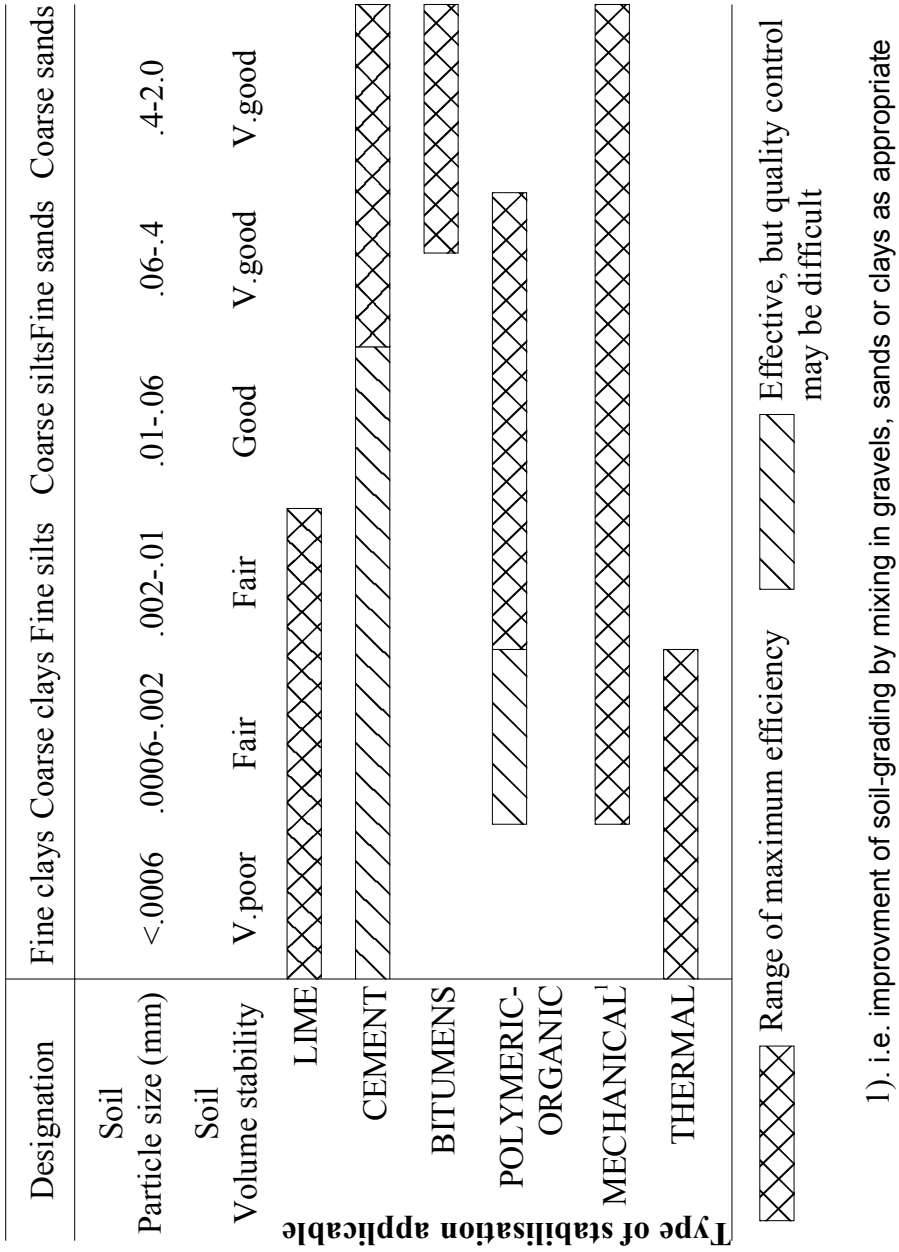


FIGURE 5.9 *Applicabilty of stablisation agents (After Ingles and Metcalf, 1972).*

Table 5.1: *Stabilisation response of major soil components (Ingles and Metcalf, 1972).*

Dominant soil component	Recommended stabilisers	Reasons
Organic matter	mechanical	other methods ineffective
Sands	clay loam cement bitumen	for mechanical stability for density and cohesion for cohesion
Silts	none known	-
Allophanes	lime	for pozzolanic strength and densification
Kaolin	sand cement lime	for mechanical stability for early strength for workability and later strength
Illite	cement lime	as for kaolin as for kaolin
Montmorillon-ite	lime	for workability and early strength
Chlorite	Cement	theoretical (reported stabilisation experience is sparse)

Åhnberg *et al.* (1995) recommend binders according to strength development in laboratory compacted samples. These tests are designed for the Deep Mixing Method (DMM) and are not directly applicable to shallow soil stabilisation. However, the study performed by Åhnberg *et al.* (1995) shows that it is possible to treat both organic soils and silty soils even though the results vary, see Table 5.2.

Table 5.2: *Classification of strength development of soils stabilised by the addition of different binders (Åhnberg et al., 1995).*

Binder	Soil									
	Clayey silt	**Silty clay**	**Clay**	**Clay (quick)**	**Clay (saline)**	**Clay (sulphide**	**Organic clay**	**Clayey mud**	**Gyttja**	**Peat**
Cement	++[a]	++	+[b]	++	+	+	+	+	+	+
Lime&cement	+	+	+	+	+	+	+	•[c]	•	•
Lime	•	+	•	+	+	-[d]	•	•	-	-

a. ++ very good effect
b. + good effect
c. • satisfactory effect
d. - no, or poor effect

Assarson (1968) recommended that the choice of binder should be based on soil grading, see Figure 5.10. In area A (see Figure 5.10) stabilisation of the clay is difficult to perform owing to difficulties in admixing. In area B the effect of grain-size distribution is insignificant since the mineral chemical reactions of the fine particles are dominant. For this area, mechanical stabilisation would require excessive amounts of imported material and is therefore not considered. Treatment of these soils with quicklime or hydrated lime can favourably modify both grain structure and plasticity. Thus, area B is the main area of lime stabilisation. In area C the grain-size distribution covers both concrete and asphalt pavements. Here the soil behaviour is determined mainly by the grain distribution itself. Soils that fit into this area can have their strength and other properties improved by cement or bitumen stabilisers (Assarson,

1968; Kézdi, 1979). According to Kézdi (1979), soils in area C cannot be treated with lime for use in road construction because the calcium hydroxide does not carbonate in this environment; cf. lime mortar that is dependent on carbonisation to develop strength. In area D, stabilisation is inappropriate, since the coarse material cannot be processed with available equipment (Kézdi, 1979).

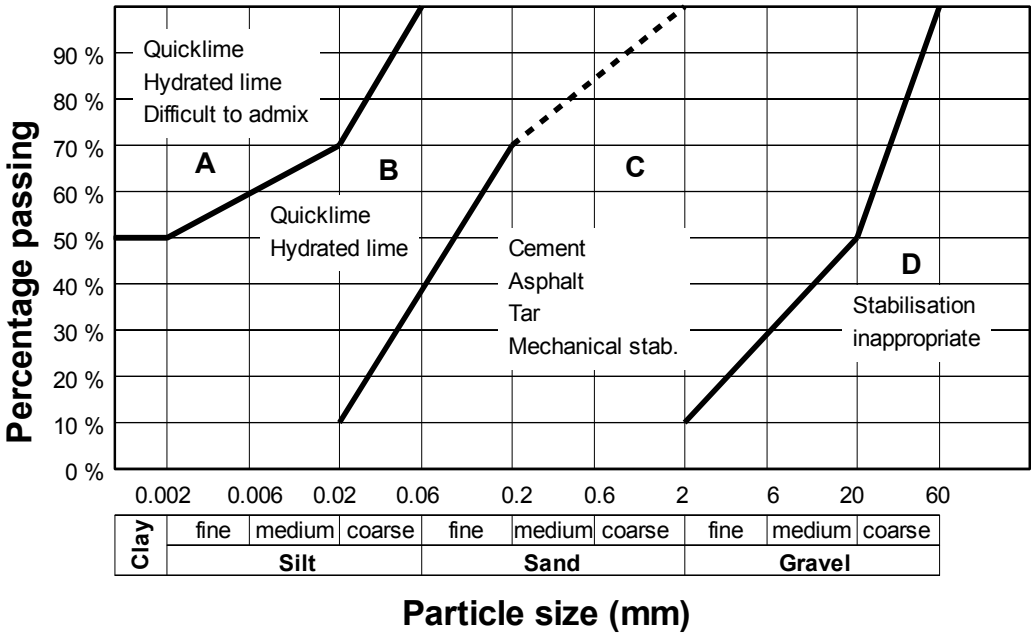


FIGURE 5.10 *Recommended grading limits for different binders. (After Assarson, 1968).*

To achieve a balance between the long-term performance and flexibility in the construction process, many Australian manufactures are producing specific binders. The binder manufacturers have given trade names to their binders. Information on the proportions of the ingredients is not always available. AustStab has classified 9 different binder

categories shown in Table 5.3 to assist pavement engineers in choosing an appropriate type of binder.

Table 5.3: *Suitability of additive to soil type. 1) Usually very suitable. 2) Usually satisfactory. 3) Usually not suitable. (Anon., 1999b).*

Binder classification	**Crushed rock**	**Well graded gravel**	**Silty/clayey gravel**	**Sand[a]**	**Sandy/silty clay**	**Heavy clays**
GP Cement	1	1	1	2	2	3
GB Cement	1	1	1	1	1	2
Cementitious blend	1	1	1	1	1	2
Lime	2	2	1	3	2	1
Lime&cement	3	3	2	3	2	1
Lime & fly ash	3	1	1	3	2	2
Bitumen	1	1	2	2	3	3
Bitumen/Cement	1	1	2	2	3	3
Insoluble polymer	2	1	1	3	1	2

a. Depends upon grading. uniform sands require higher additive contents

From these nine different categories a great number of different binder blends can be manufactured. To evaluate the effect of these blends on different soils, a testing tool must be used.

5.3.1 Soil modification

When quicklime is used, it first reacts with the soil moisture and a slaking process is started. The chemical reaction can be described as (Sherwood, 1993).

$$CaO + H_2O \Rightarrow Ca(OH)_2 + heat \qquad \textbf{(EQ : 5.1)}$$

Each 100 g CaO binds 32 g H_2O and, due to the exothermic reactions, extra water may evaporate. After the slaking process, the hydrated lime acts as a Ca^{++} source. When the Ca^{++} is dispersing in to the clay-water system, the divalent calcium cations almost always replace the cations normally adsorbed to the surfaces of the clay particles.

The reason why calcium replaces most cations available in the pore water is explained by the Lyotropic series. The Lyotropic series generally states that cations with higher valence replace those with lower valence. The Lyotropic series is written as:

$$Li^+ < Na^+ < H^+ < K^+ < NH4^+ << Mg^{++} < Ca^{++} << Al^{+++}$$

The cation to the right replaces the one to the left. When Na+ ions are replaced with Ca^{++} ions, the water layer that surrounds the clay particles is reduced, see Figure 5.11 (Andersson, 1960; Wäre, 1974; Little, 1995). This results in a coagulation of the clay to a gel. The viscosity increases considerably and the consistency changes from liquid to plastic or solid, depending on the initial water content. The soil water that has become separated from the clay particles is now located in the cavities between the coagulated particles forming the aggregate (Andersson, 1960). This ion exchange is completed within one hour after the soil and

lime are mixed (Eades and Grim, 1966). Rogers *et al.* (1997) consider that different clays need different curing periods for full modification and the changes occur within 24-72 hours.

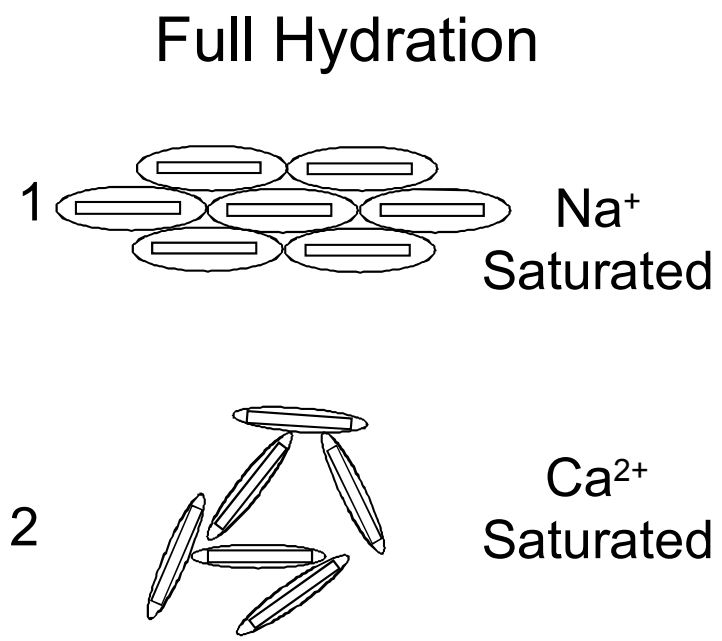


FIGURE 5.11 *Textural change due to cation exchange from Na^{+} to Ca^{++}. The addition of Ca^{++} shrinks the water layers and makes it possible for the plate-like particles to flocculate. (after Little 1997).*

This flocculation/agglomeration transforms the clay to a material with a needle-like interlocking molecular structure compared to the plate-like structure of an untreated soil. This reaction is called lime modification (Anon., 1990 (BLA, 1990); Little, 1987). For a comparison,

the relative strength of different bond types is approximately as follows in Table 5.4 (Kézdi, 1979):

Table 5.4: *Relative strength of different bond types (Kézdi, 1979).*

Forces/bonds	relative strength
van der Waal forces	1-10
hydrogen bond	10-20
ionic and covalent bonds	40-400

The flocculation/agglomeration of a clay till is illustrated in Figure 5.12.

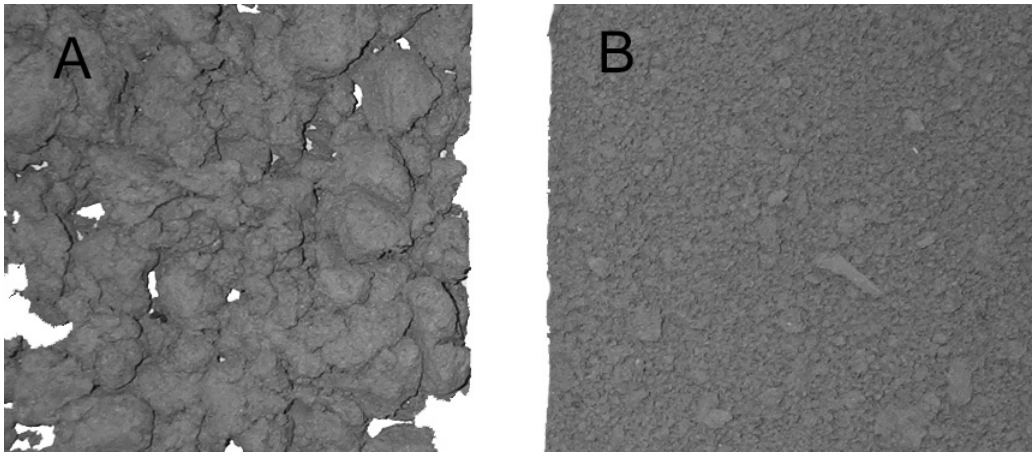


FIGURE 5.12 *Photo A shows an untreated clayey sand till. Photo B shows the same soil treated with 2% hydrated lime. Each photo covers approximately 10 cm.*

The soil plasticity decreases by the flocculation of the clay particles. This effect is also dependent on what clay minerals the soil consists of. Montmorillonite has a higher ion exchange capacity compared to

kaolinite (Sherwood, 1993). Cations-exchange capacities for different clay minerals are presented in Table 5.5 (Drever, 1982). Table 5.5 should be compared to Table 5.1 in which Ingles and Metcalf (1972) recommend binders according to the major soil component.

Table 5.5: *Cations-exchange capacities of clay minerals (Drever, 1982).*

Clay mineral	*Cations-exchange capacity (meq/100g)*
Smectite	80 - 150
Vermiculite	120 - 200
Illite	10 - 40
Kaolinite	1 -10
Chlorite	<10

The soil modification effect is well shown in the compaction properties, see Figure 5.13. Littleton *et al.* (1988) performed a study on Oxford clay where they studied both the immediate effect and the long-term effect regarding compaction and strength properties. Figure 5.13 shows that the major difference is between the stabilised and the unstabilised soil. The differences between soils stabilised with different amount of binders are relatively minor. The plot also shows that Oxford clay with a water content of 30% has a MCV of approximately 8 for the untreated soil and approximately 14 for the treated soil.

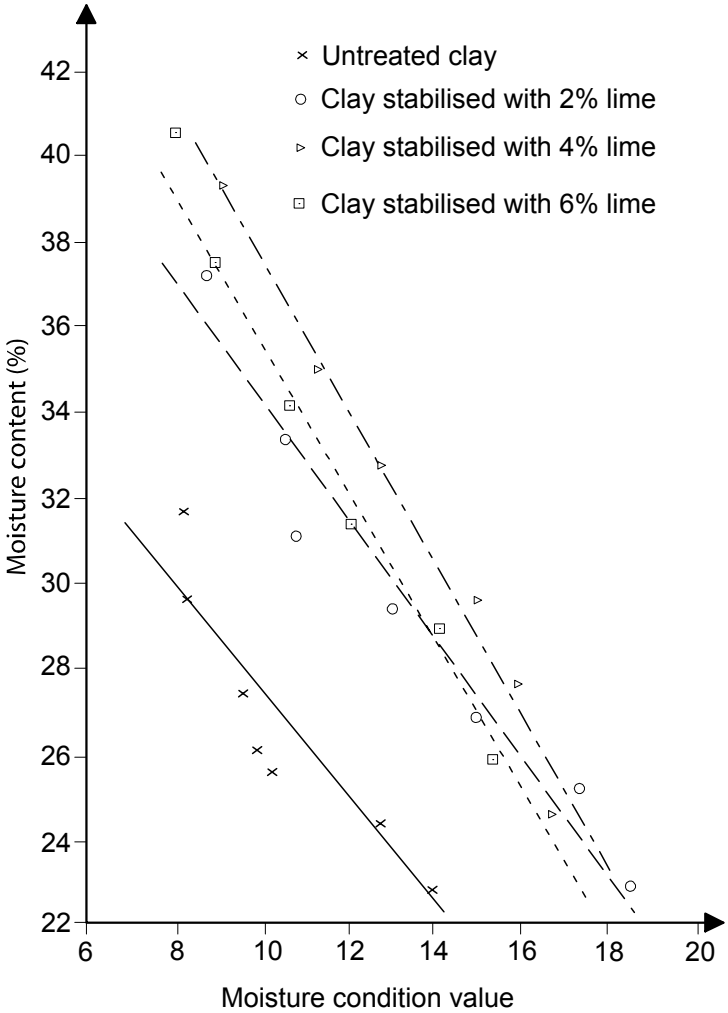


FIGURE 5.13 *MCV calibrations for stabilised and unstabilised Oxford clay. (After Littleton et al., 1988)*

In Figure 5.14 the soil's cohesion is plotted against moisture content. The results show that there is a clear difference between unstabilised and stabilised soil. For a moisture content of 26% the unstabilised soil has a cohesion of approximately 70 kPa whereas the stabilised soil has a

cohesion of approximately 160 kPa. The plot further shows that there is no major difference in cohesion for the soils stabilised with 2, 4 or 6% lime.

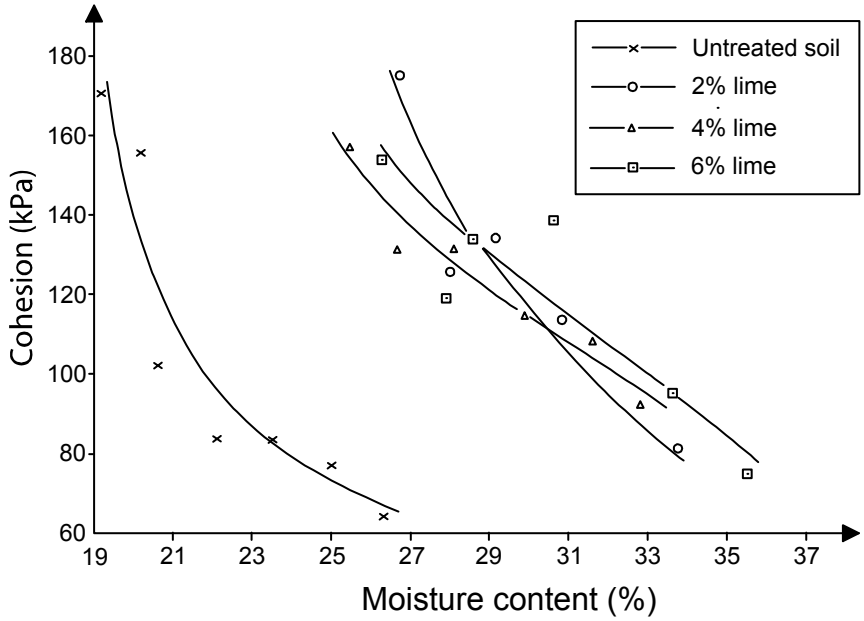


FIGURE 5.14 *Results from undrained triaxial tests. (After Littleton et al., 1988).*

This effect is limited to the modification process. Figure 5.15 shows the difference between the soil modification effect and the soil stabilisation effect. The plot shows that an increased lime content also increases the cohesion of the stabilised soil.

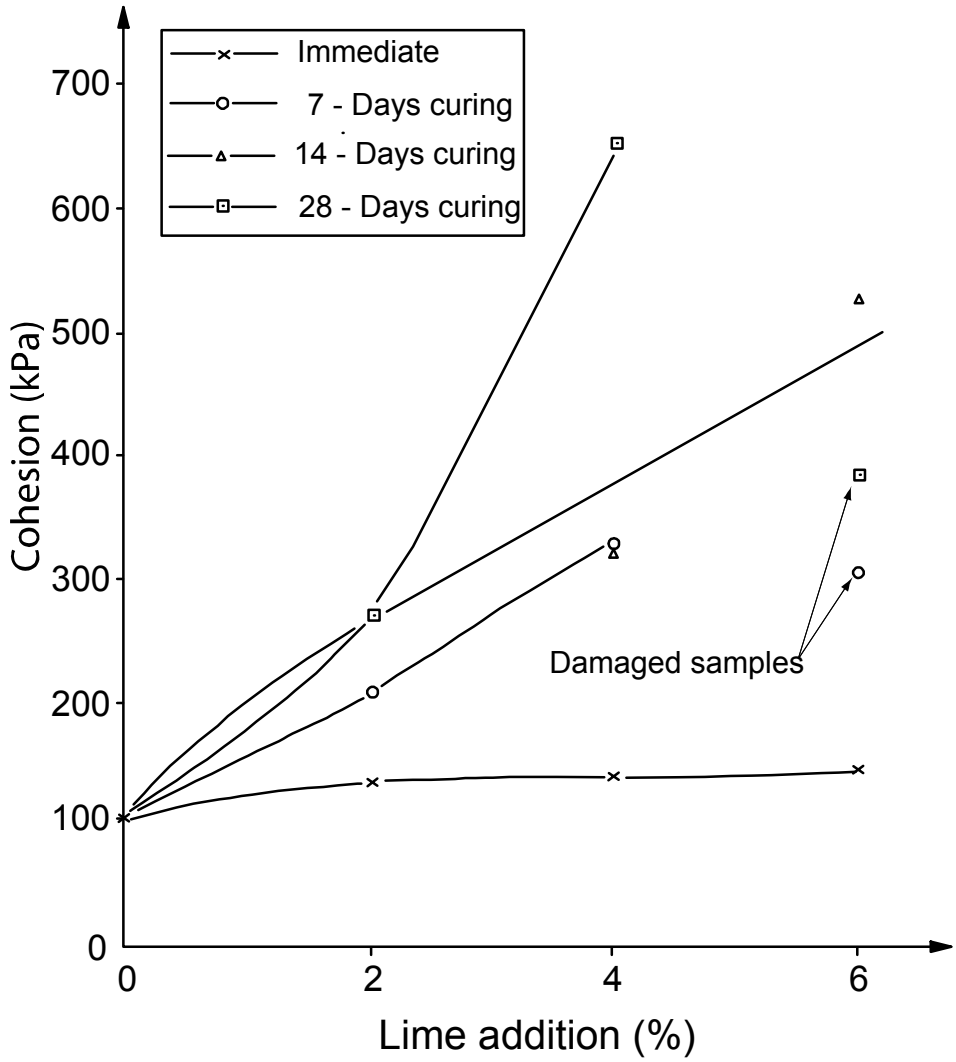


FIGURE 5.15 *Plot of cohesion against lime content for varying periods of curing time for quick triaxial tests on Oxford clay. (After Littleton et al., 1988).*

5.3.2 Soil stabilisation

The soil stabilisation stage starts after the soil modification stage. The soil stabilisation stage is defined as the long-term effect caused by the pozzolanic reactions between lime and clay that produce cementitious products (Perry *et al.*, 1996). These pozzolanic reactions are complex and predominantly influenced by soil conditions and mineralogical properties. However, many clay soils are pozzolanically reactive when stabilised with lime. A pozzolan is defined as a siliceous or alumino-siliceous material, which, in the presence of water and calcium, will form a cemented product due to chemical reactions. Clay is a pozzolan since it is a source of silica and alumina for the pozzolanic reaction.

This reaction is established if the soil-lime-water system has a high pH to make the clay-silica and clay-alumina soluble, cf. Figure 5.16. The pozzolanic reaction is then dependent on access to silica, alumina and indirectly on a high pH environment. The pozzolanic reactions will continue as long as these conditions are fulfilled. The reaction rate is temperature-dependent and is increased by a high temperature. Below $7^{o}C$ the reaction rate is very slow (Kujala, 1984).

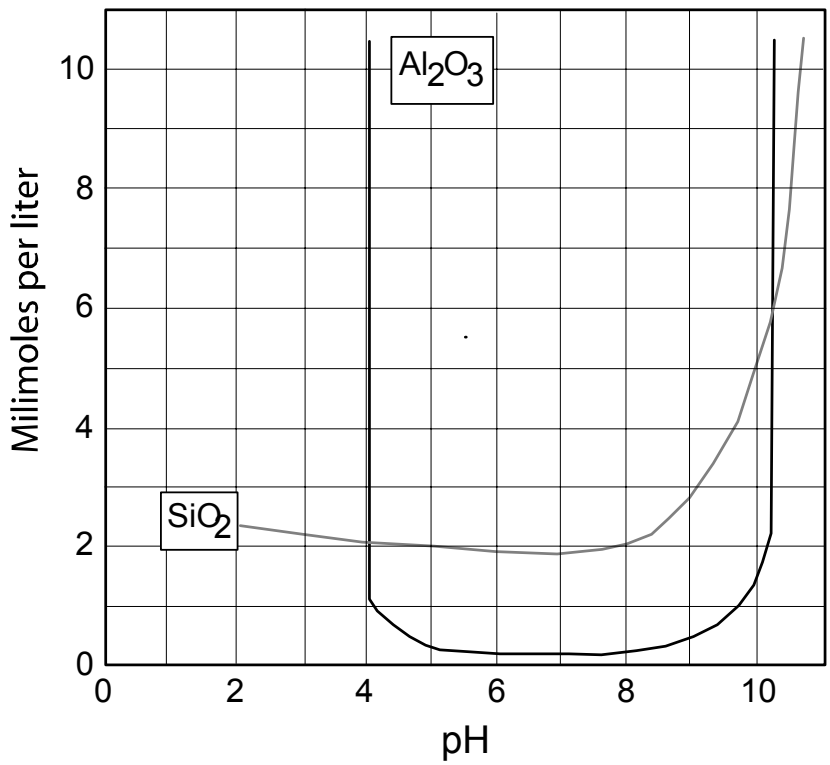


FIGURE 5.16 *The effect of a high pH system is to release silica and alumina from the clay system. Soluble silica and alumina are necessary for the pozzolanic reactions (after Little, 1995 (based on Keller, 1964)).*

The solubility of silica is dependent on both pH and temperature. Both alumina and silica are highly soluble at pH above 10.5 (Keller, 1964).

The pozzolanic reactions can be described by the following equations (Wäre, 1974; Boynton, 1980; Little, 1987; Prusinski and Bhattacharja, 1999):

$$Ca^{++} + OH- + \text{Soluble Clay Silica} \Rightarrow \text{Calcium Silicate Hydrate (CSH)} \qquad \textbf{(EQ : 5.2)}$$

$$Ca^{++} + OH- + \text{Soluble Clay Alumina} \Rightarrow \text{Calcium Aluminate Hydrate (CAH)} \quad \textbf{(EQ : 5.3)}$$

According to Eades and Grim (1966) the gain in strength increases until the pH is around 11. The conclusions of their study were that enough lime was used when the soil-lime slurry had a pH of 12.4 after one hour of curing. This technique was adopted in Great Britain and the method is described in British Standard (BS) 1924 Part 2 (1990) and is called initial consumption of lime (ICL). However, the method is criticised by Rogers *et al.* (1997) as being conservative.

A combination of the ICL-method and the plasticity index (I_P) method results in lower lime quantities according to Rogers *et al.* (1997). The ICL method alone results in overdosage of lime and a more costly design than required.

As soon as cement gets into contact with soil moisture, several chemical reactions start. Calcium silicate reacts with water and forms calcium hydroxide that, together with easily soluble alkali sulphate, gives a high pH environment, approximately pH 13. The chemical composition of cement paste is presented in Figure 5.17. The calcium silicate reactions with water can be described as[1]:

$$2C_3S + 6H \Rightarrow C_3S_2 \cdot 3H + 3Ca(OH)_2 \quad \textbf{(EQ : 5.4)}$$

$$2C_2S + 4H \Rightarrow C_3S_2\ 3 \cdot 3H + Ca(OH)_2 \quad \textbf{(EQ : 5.5)}$$

Calcium silicate hydrate ($C_3S_2 \cdot 3H$) forms a rigid gel. The CSH gel grows from the cement grain and fills up the pore volume between the

1. Cement chemistry uses special abbreviations, cf. Taylor (1997).

soil grains. The porosity decreases and the soil grains are bonded together. From 100g cement and 25g water 100g CSH and 25g $Ca(OH)_2$ is formed. Another chemical reaction that starts is that the tri calcium aluminate ($Ca_6Al_2O_6$) reacts with calcium sulphate (gypsum) and forms ettringite. Equation 5.6 describes the reaction.

$$C_3A + 3CaSO_4 + 32H_2O \Rightarrow C_3A \cdot 3CaSO_4 \cdot 32H_2O \qquad \textbf{(EQ : 5.6)}$$

Ettringite transforms into mono sulphate $C_3A \cdot CaSO_4 \cdot 11H_2O$ when the soluble sulphate is reduced (Åhnberg *et al.*, 1994). The reduction of soluble sulphate with time can be seen in Figure 5.17 (Lawrence, 1966). Ettringite consists of needle-shaped crystals that build up some strength. If the ettringite is formed after the cement has hardened, it can cause expansions and cracking. This could lead to serious loss of strength (Sherwood, 1993). This problem could also arise in lime-stabilised soils, where the soluble sulphate reacts with the cementitious materials that are produced in the pozzolanic processes (Sherwood, 1993; Snedker, 1996).

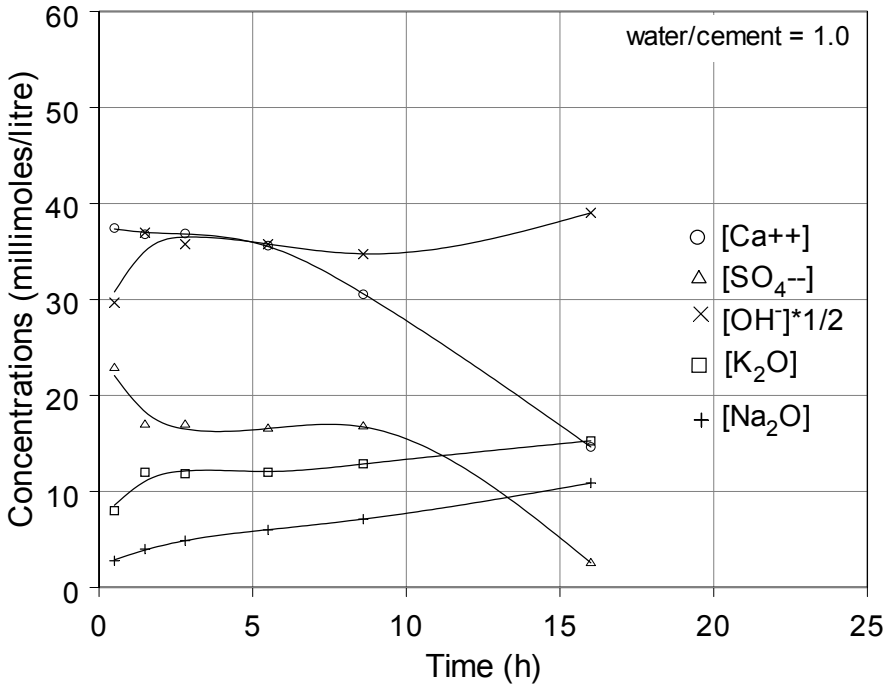


FIGURE 5.17 *Chemical composition of the water phase after mixing cement and water (after Lawrence, 1966).*

For soil stabilisation purposes the working period of the binders is of great importance. As was discussed earlier, cement has a working period of about two hours (Sherwood, 1968; Vorobieff, 1997). The effects of delay between mixing and compaction have been studied by West (1959), Dumbleton (1962), Sherwood (1968) and Toledo (1989). West (1959) showed that a delay between mixing and compaction had a impact on the dry density for a medium clay. However, this effect could not be found on the dry density of sand-cement. West (1959) also showed that cement-stabilised sand has the lowest reduction in strength versus delay time.

Sandy gravel and medium clay show the greatest corresponding loss in strength, see Figure 5.18.

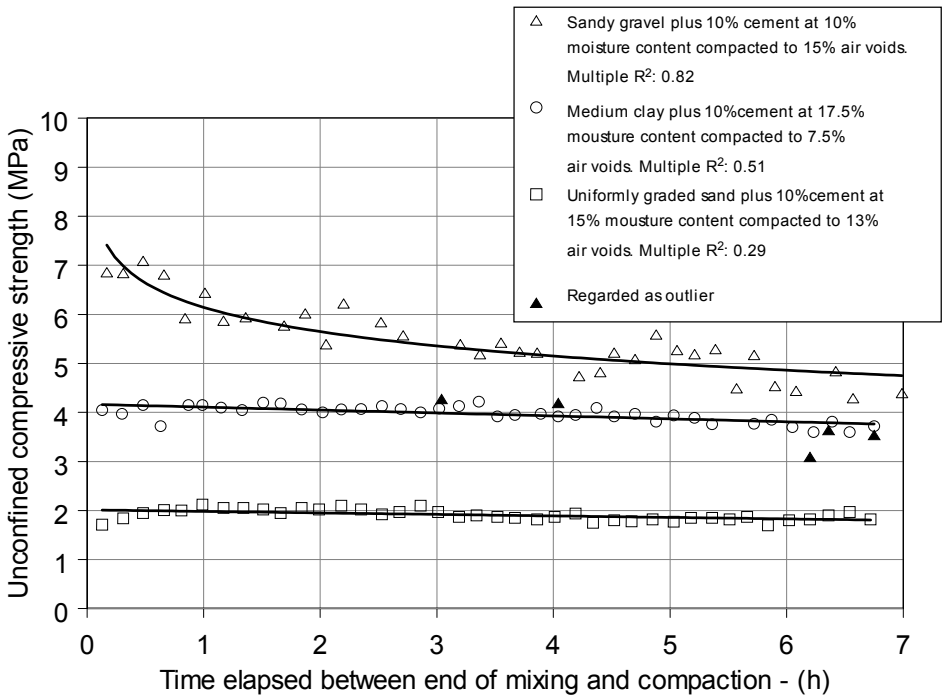


FIGURE 5.18 *Relation between UCS and elapsed time for specimens compacted to a constant density (interpretation based on data from West, 1959).*

Some of the observations in Figure 5.18 are marked as outliers of the author of this thesis and are not used in the regression analysis. Outliers are atypical (by definition), infrequent observations (Anon., 1995a). West (1959) showed that even if the compaction effort was increased until a constant air void volume was reached, there was still a significant decrease in strength with time delay before compaction. The R^2 value gives the proportion of the variability that is explained by the regression

model, in this case log-linear or linear. The correlation between strength and compaction delay could be considered good for the cement-stabilised sandy gravel, weak for the medium clay and non-existing for the uniform sand.

Dumbleton (1962) showed that the dry density of London Clay stabilised with 10% lime was less sensitive to delay between mixing and compaction compared to London Clay stabilised with 10% cement, see Figure 5.19. After a 7-day delay, the lime-stabilised London Clay had a dry density of 1.31 Mg/m^3 and the cement-stabilised London Clay had a dry density of 1.28 Mg/m^3. Dumbleton (1962) recommended that cement-stabilised soils should be compacted without any delay. His recommendation for lime-stabilised soils was that a light compaction should take place on the following day after mixing and then, after several days, the soil should be re-processed and then re-compacted.

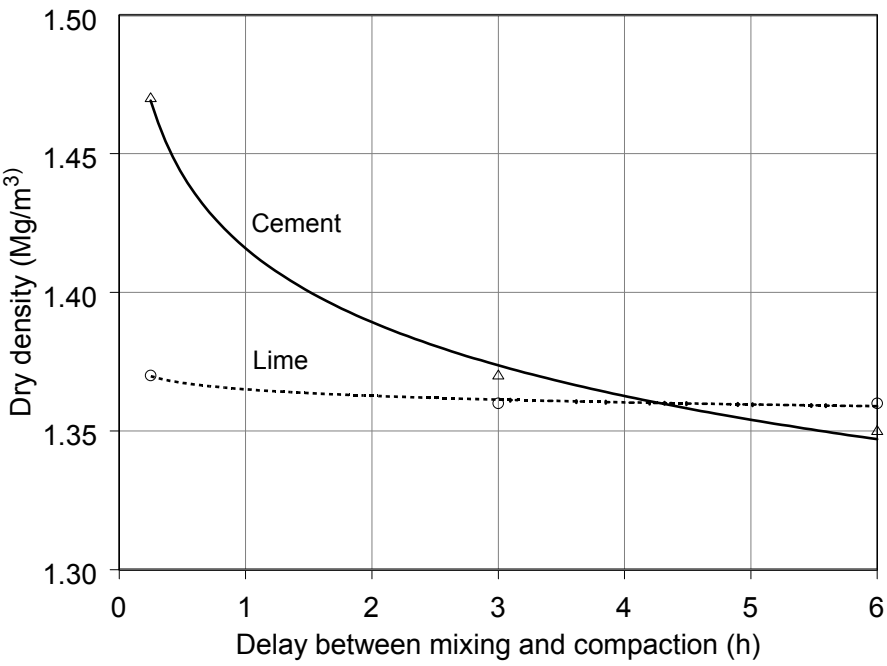


FIGURE 5.19 *Decrease in dry density obtained with equal compaction caused by delays between mixing and compaction. Tests performed on London Clay mixed with lime and cement respectively (After Dumbleton, 1962).*

Sherwood (1968) considered that extra compaction efforts are required if there is more than two hours delay between mixing and compaction. In Table 5.6 the relative strength decrease after 2 hours of delay in compaction as compared to the strength obtained with immediate compaction is shown for a number of materials.

Table 5.6: *The effect of a time lap of two hours between mixing and compaction on the strength of cement-stabilised materials (Sherwood, 1968).*

Material	Loss of strength (%)
Sandy Gravel	60
Medium Clay	50
Fine Sand	0
As-dug Gravel	22
Crushed Gravel	9
Slag	0
Limestone (fine grading)	17
Limestone (coarse grading)	29
Limestone	6
Fine Sand	0
Sandy Gravel	14
Gravel-sand-clay	12
Gravel-sand-clay	16
Crushed rock	12
Well graded sand	16

De Toledo (1989) showed that a delay time of 2 hours between mixing and compaction decreased the obtained dry density obtained by standard Proctor from 101% to 91%. The 7-day unconfined compressive strength decreased from 3.75 MPa to 2.65 MPa after a delay time of 2 hours. These results were obtained in laboratory tests.

5.3.2.1 Frost susceptibility

In several countries, including Sweden, the soil's frost susceptibility is of major concern in road construction because of its adverse effect on the long-term performance of roads. The major remedy is to increase the thickness of the road construction with non-frost-susceptible soils or aggregates. Since soil stabilisation gives the opportunity to decrease the thickness of the pavement, the frost-susceptibility of the stabilised soil is important. Several studies have been performed to evaluate this property in stabilised soils. The results have shown both positive and negative results for lime-stabilised clayey soils.

Littleton *et al.* (1988) compared the frost-susceptibility for untreated and lime-treated clay. They found that both untreated and lime-treated Oxford clay were non-frost susceptible, see Figure 5.20. However, they also found that lime-treated soil could be frost-susceptible if it was compacted very wet.

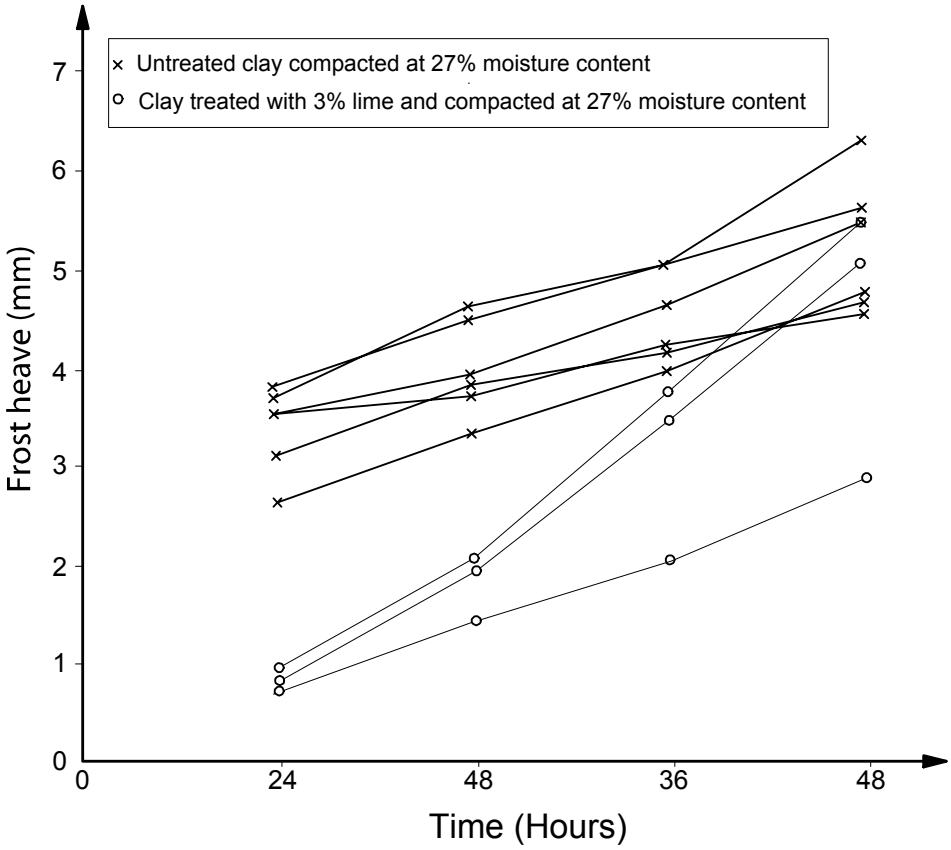


FIGURE 5.20 *Results from frost heave tests performed on lime-treated Oxford clay compacted at a moisture content of 27%. (After Littleton et al., 1988)*

Another study by Arabi *et al.* (1989) showed that a soil treated with a low amount of lime could become more frost-susceptible than untreated soil, see Figure 5.21.

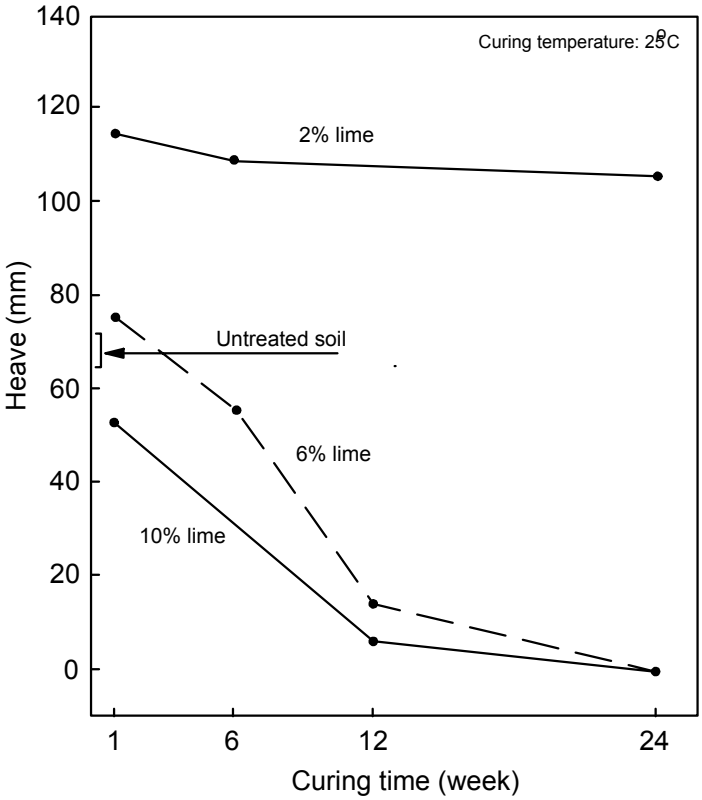


FIGURE 5.21 *Frost heave after 7-days of freezing at different curing times for the specimens cured at 25°C before freezing. (After Arabi et al., 1989).*

6 Laboratory- and field tests

6.1 Purpose and approach

The laboratory tests were performed to study compaction and strength properties and make a general characterisation of the soils. The general characterisation was made to enable comparisons with existing data found in the literature and to supply forthcoming studies with data. Some of the special tests were performed for the stabilised soil. The approach during the laboratory testing was to produce the best possible results, even though this may not reflect site conditions.

6.2 Experimental design

The laboratory testing in this study is divided into two different types: tests using standard methods and those using non-standard methods. The non-standard methods have been employed in those cases where standard methods were unavailable or when the standard methods are out of date. For the standard methods, Swedish Standard or British Standard

have been employed. The forthcoming European standard by CEN has also been employed when applicable.

6.2.1 Tested soils

Twelve different soils were tested in this study. All soils were fine-grained or medium-grained tills from different locations in southern Sweden.

6.2.2 Soil preparation

During this study a great volume of soil has been processed. The processing includes homogenisation, removing of particles larger than 20 mm, and wetting and drying. After wetting and drying, the soils were stored in plastic containers for at least seven days to achieve equilibrium before any test was performed. As an extra precaution, the plastic containers were stored in a climatic room with a temperature of 8°C and a relative humidity of 80%. According to British Standard the storing time should be at least 24 hours (Anon., 1990c). The seven days' storing was not chosen from any theoretical consideration; rather it was chosen to be well above the minimum recommendations in a standard. It was found later that Olson and Langfelder (1965) also used this storing time and 7-days' storing time is used in the ASTM Filter Paper Method (Anon., 1997).

Soil specimens were prepared in three different ways: from soils at the natural water content, from air-dried soils and from oven-dried soils. Mostly, soilswithout pre-drying were used to prepare specimens, i.e. the soil with its natural water content was wetted or partly dried to the desired water content. This procedure is criticised because of hysteresis effects when wetting and drying fine-grained soils (Winter, 2001).

However, the soils were excavated mainly during the wet season with rather high natural water contents. No extra water was therefore needed, and only partial drying was used, to achieve the desired water contents.

The British method to avoid hysteresis in MCV calibration is to air-dry the soil at room temperature (Anon., 1990c). Specimens can then be prepared from this air-dried soil. A fine-grained soil that is air-dried contains more residual water compared to oven-dried soils. The problem with this type of preparation is that air-drying also breaks up some aggregates and bonds between the fine-grained particles (Winter, 2001). These aggregates and bonds cannot be recreated. Associated changes in the soil behaviour will probably be caused by the breaking of the water meniscus between the fine-grained particles and aggregates. In the wetting process deionized water is used, and this can also create a dilution that affects the results.

The oven-dried soil was dried at 105°C for at least 24 hours. Distilled water was used to prepare soil specimens with the desired water content from both the air-dried and oven-dried soils.

6.2.2.1 Soil preparation for testing the ageing effect on strength

When a soil is stabilised with binders, there will be a strength increase with time. The strength increase depends on both binder type and amount of binder. The tests concerning ageing effect on shear strength were performed to investigate if there was any strength increase for compacted natural soil also. If there was any such strength increase, this was to be measured and compared to the strength increase produced by the stabilising agent. The tests were performed in two different ways, with specimens stored within the MCV mould during storing and specimens

first removed from their moulds and then stored. The two different methods were used to find out if different results were obtained with the different procedures. Neither of the methods fully simulates field conditions regarding compaction. Field compaction could be regarded as falling somewhere between them. Figure 6.1 shows MCV cylinders that were prepared for storing.

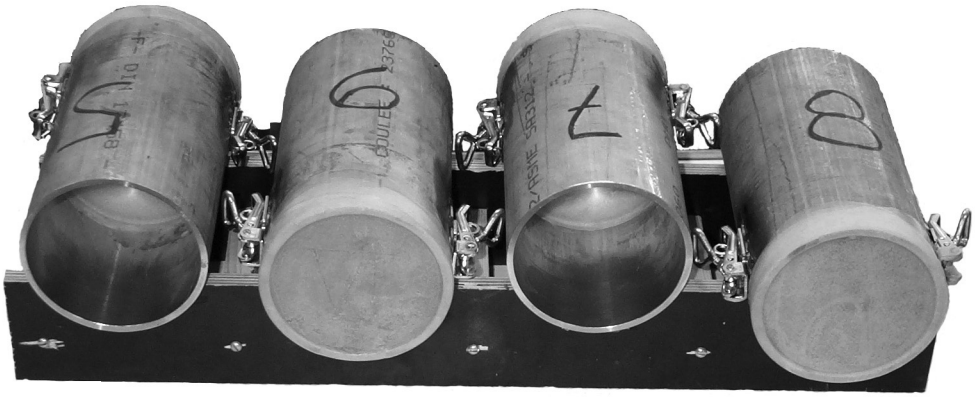


FIGURE 6.1 *MCV cylinders of stainless steel containing compacted soil covered with paraffin.*

The specimens were weighed before and after storing and if the difference in weight was more than 1 gram the specimens were excluded from the test series.

6.3 Soil classification

The standard methods for soil classification employed in this study are:

- Soil constitutes and structure - symbols, terms and definition *SS 02 71 13*
- Mineral soils - Grain size fractions *SS 02 71 06*
- Particle size distribution - Sieving *SS 02 71 23*
- Particle size distribution, sedimentation - Hydrometer method *SS 02 71 24*
- Organic content - Colorimeter method *SS 02 71 07*
- Grain density and specific gravity *SS 02 71 15*
- Water content and degree of saturation *SS 02 71 16*
- Void ratio and porosity *SS 02 71 17*
- Cone liquid limit - *SS 02 71 20*
- Plastic limit - *SS 02 71 21*

The classification tests are mainly straightforward and are therefore performed according to the standard. However, in some cases the requirements in the standard have not been followed. These exceptions are commented on later.

6.3.1 Water content

The water content, also called moisture content, is determined as the ratio between the mass of water and the mass of dry soil. The mass of dry soil was determined after a drying time of 24 hours in an oven with a temperature of 105°C. For compacted soil-specimens (large bodies) the drying time was extended to at least 48 hours. Due to the large volume, the tested soil specimens were not allowed to cool before weighing. The Swedish standard SS 02 71 16 involves cooling the samples in a desiccator before weighing, and the difference between cooled and hot samples was therefore tested, see Figure 6.2.

The laboratory result shows that the difference between the treatments is significant (at a = 0.01). However, the actual difference in value was only 0.03% and therefore accepted.

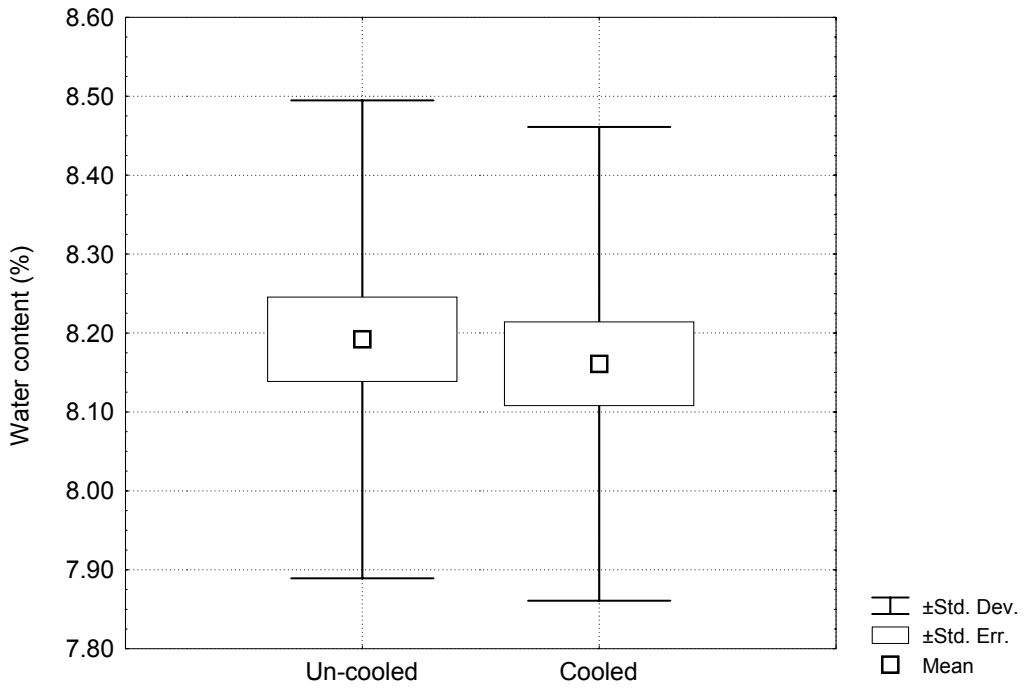


FIGURE 6.2 *Measured water contents in hot specimens and specimens cooled in desiccator. The graph is based on 32 samples.*

According to Swedish standard 02 71 16 (Anon., 1989), a specimen for determination of the water content of a clay should have a mass of 10 - 30 g; for a fine sand 100 g; for medium and coarse sand 200 g; for gravel and till with grain size < 20 mm the specimen mass should be at least 500 g.

This 500 g limit is seldom used by Swedish laboratories to determine water contents in fine-grained tills since it is impractical (Persson, 2004; Holmén, 2004; Petterson, 2004).

In recommendations for geotechnical laboratory testing given by ISSMGE, the tested soil mass should be related to the D_{90} value (Anon., 1998a). The recommended minimum specimen mass is presented in Table 6.1, cf. "Water content" on page 156..

Table 6.1: *Minimum specimen mass for water-content determination (After Anon., 1998).*

Grain size diameter D_{90} (mm)	Minimum mass of moist specimen[a] (g)
1.0	25
2.0	100
4.75	300
20.0	500
38.0	1500
76.0	5000

a. Using a test specimen smaller than the minimum mass indicated requires discretion, though it may be adequate for the purpose of the test. A specimen having a mass less than the indicated value shall be noted in the report of the results. In many cases when working with a small sample containing a relatively large coarse-grained particle, it is appropriate not to include this particle in the test specimen. If this occurs, it should be noted in the report of the results.

6.4 Compaction properties

The procedures in the tests are mainly based on a standard, but some features are adjusted in order either to fit older practice or to better simulate site conditions. Some of the compacted specimens were used for unconfined compressive testing or for triaxial testing.

6.4.1 The modified Proctor method

The modified Proctor tests are based on Swedish standard SS 02 71 09 (Anon., 1994d) . There is a minor difference between Swedish standard and British standard (Anon., 1990d) regarding the number of blows. In Swedish standard there are only 25 blows (2,482 kJ/m^3) compared to 27 in the British standard (2,682 kJ/m^3). Otherwise the tests are identical.

6.4.2 The MCV method

The MCV method is a compaction test method that has been standardised in Great Britain. Other European countries will adopt the test method established by the forthcoming CEN-standard. Since Sweden and other countries have not yet adopted a standard for the MCV method, a fairly detailed description of the method is given here.

Parsons (1976) developed the Moisture Condition Value (MCV) test method at the Transport Research Laboratory (TRL). The test offers a rapid determination of the moisture condition of earthworks materials. It is aimed at construction control to assess the acceptability of materials in relation to the specified upper limit of the water content (Parsons, 1976). In MCV testing, a special device, the moisture condition apparatus (MCA), is used.

The apparatus has a mould with a detachable base and an inner diameter of 100mm. A free-falling rammer with a mass of 7kg and a diameter of 97mm is attached to an automatic-release mechanism. Normally, a soil specimen of 1.5kg is used together with a drop height of 250mm. A lightweight disk is placed on top of the soil to avoid extrusion of soil between the rammer and the sides of the mould. The disk also prevents smearing by the soil onto the rammer sides, see Figure 6.3.

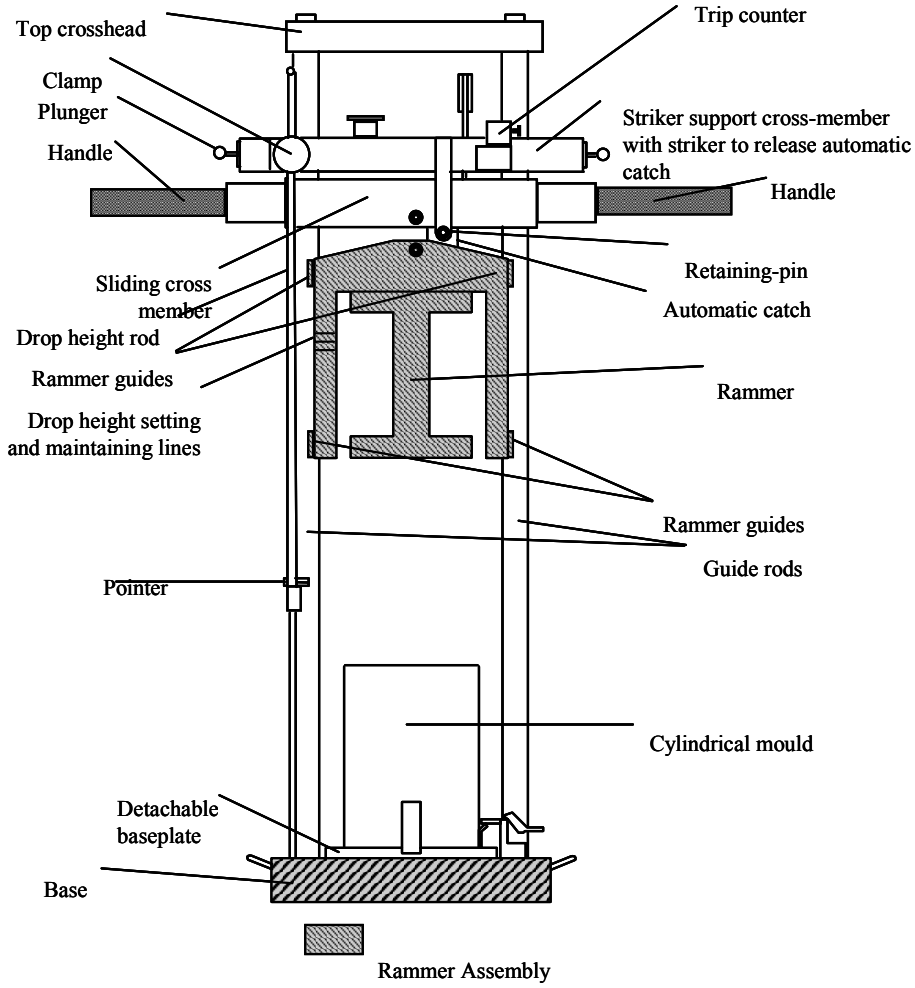


FIGURE 6.3 *The Moisture-Condition Apparatus.*

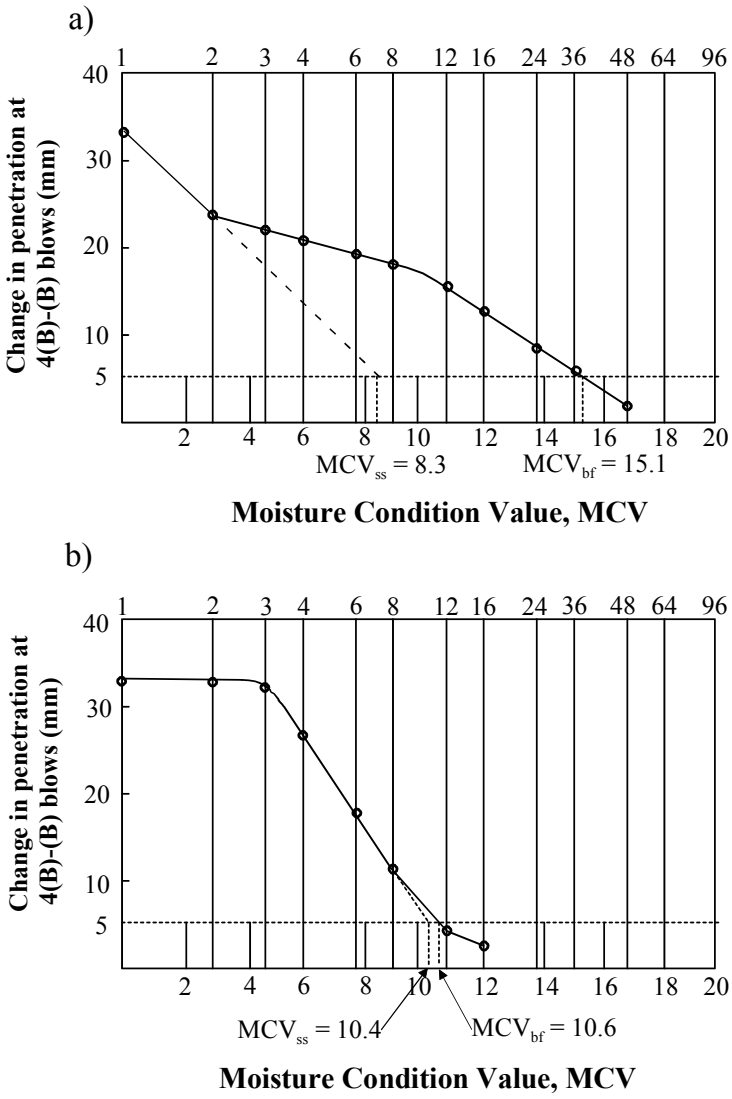
Parsons developed the method for use exclusively with cohesive (or fine-grained) materials. However, it was later adapted to enable its use with many granular (coarse-grained) materials, particularly tills with the

wide particle size range encountered in the UK (Matheson and Oliphant, 1991; Matheson and Winter, 1997).

The MCV of a soil sample is defined as the lowest compaction energy required to obtain maximum attainable compaction at a specific water content. However, it would take too long to reach full compaction and an alternative procedure is used instead. To calculate the MCV, the penetration of the rammer at any given number of blows is compared to the penetration at four times as many blows and the difference is determined. This difference in penetration is plotted against the logarithm of the lower number of blows.

For cohesive soils, the straight-line extension of the steepest part of the curve is then normally used to define the point at which the 5mm change in the penetration line is crossed (Anon., 1990c). For granular soils the best-fit method is empoyed, see Figure 6.4. Matheson and Winter (1997) make a strong case for the exclusive use of the best-fit line. The best-fit method has been empoyed in this study.

The MCV is defined as 10 times the logarithm of the number of blows corresponding to a difference in penetration of 5mm on the plotted curve.



Key: MCV_{ss} - MCV by steepest Straight line method
MCV_{bf} - MCV by best-fit method

FIGURE 6.4 *Determination of MCV: (a) cohesive (fine-grained) materials; and (b) granular (coarse-grained) materials.*
(after Matheson and Winter, 1997).

A determination of MCV versus water content for the soil is performed by several tests at different water contents. A linear regression is performed from the results . This regression forms the equation:

$$w = a - b(MCV) \qquad \textbf{(EQ : 6.1)}$$

where w is the water content (%); a is the intercept with the water content axis (%); and b is the regression coefficient or the slope of the line (% water content change per MCV) (Cf. Equation 3.6).

From the use of parameters a and b certain conclusions can be drawn about the compaction properties of the soil. The parameter a is an arbitrary low-strength water-content value, which can be used as a crude index value. The b parameter indicates the sensitivity of the soil to a change in water content, a small b value indicating high sensitivity. A regression, or calibration line, is illustrated in Figure 6.5.

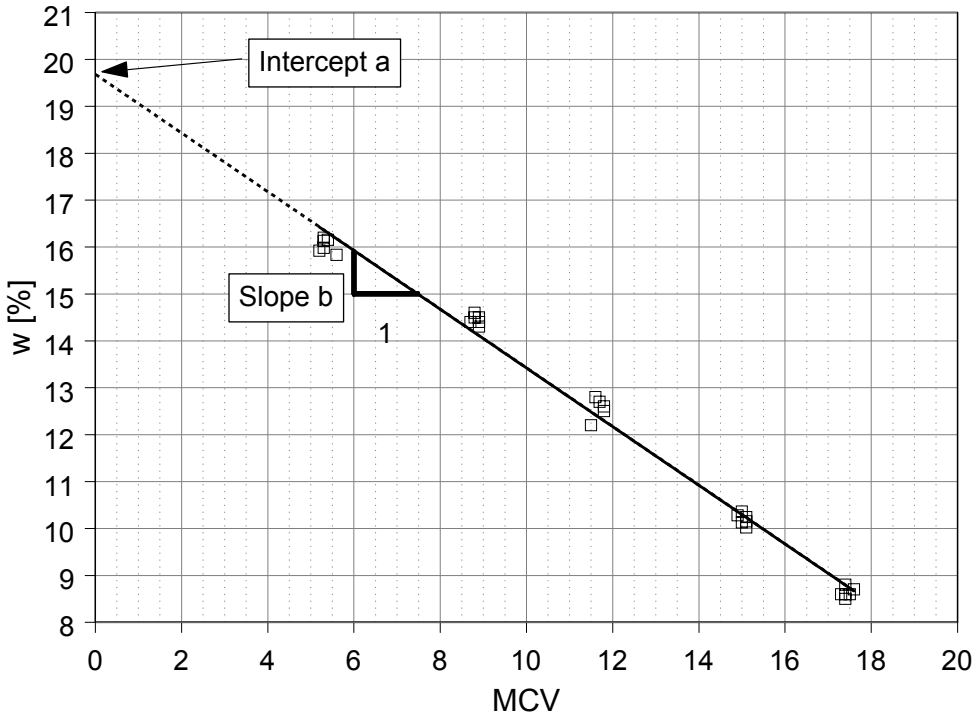


FIGURE 6.5 *MCV calibration line for the Östra Torn material.*

The applicability of the MCV test is defined by the position of the plotted particle size distribution on the ternary diagram (Oliphant and Winter, 1997) shown in Figure 6.6. The MCA (moisture-condition apparatus) has been in routine operation since 1983 in Scotland

(Matheson and Oliphant, 1991; Matheson and Winter, 1997), an area which, like Sweden, is dominated by glacial tills.

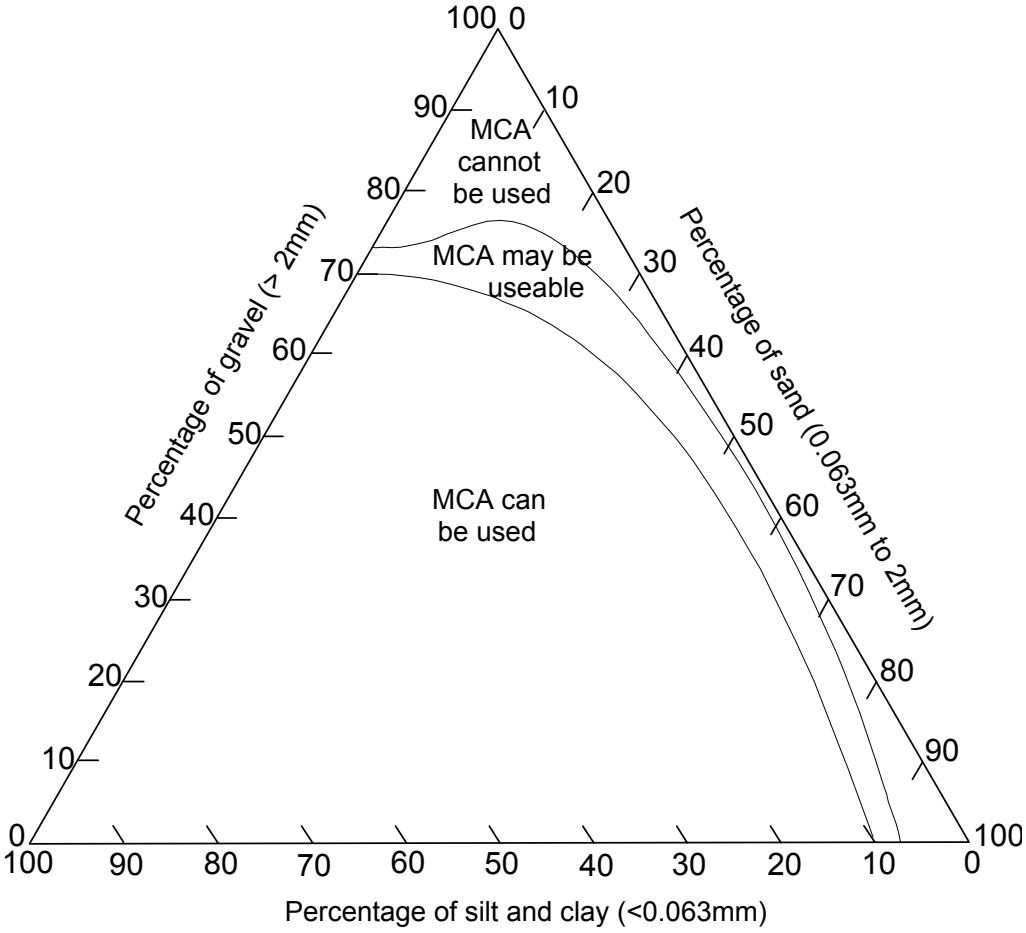


FIGURE 6.6 *Soils suitable for the use of the MCA. (after Oliphant and Winter, 1997; Matheson and Winter, 1997).*

In contrast to the Proctor method, the MCV method applies different amounts of compaction energy depending on soil type and actual water content. Another major difference is the way of applying the compaction energy. The Proctor method applies the compaction energy with a 50mm diameter rammer and the position of the rammer is changed during the

operation in order to ensure an even distribution of energy to the upper face of the specimen. The mould has a diameter of 101mm. In the modified Proctor test, the energy is applied at five different layers. In contrast, the MCV method applies the compaction energy to a single layer and over the entire area of the test specimen.

6.4.2.1 The effect of sample preparation on MCV

The British standard, which uses 1500g (±20g) of soil has been applied. However, several different soil masses were used during the introduction of the MCV method in Sweden. This makes a comparison with earlier Swedish data difficult; since no standard had then yet been adopted.

One reason to increase the soil mass in the earlier tests was to increase the height of the compacted specimens. For Swedish clay tills, 1800g of soil will result in specimens with a height of approximately 100 mm. If two such specimens are put on top of each other, the composite specimen will have a height of approximately 200 mm. Since the diameter of the specimen is 100 mm this will produce a slenderness ratio of 2:1, which is the standard for unconfined compressive strength tests. The British Standard is now widely adopted when using the MCV in Sweden and the corresponding height of approximately 75 to 85 has to suffice.

The effect of using different soil masses was studied by Lindh and Winter (2003). Six different soil masses were used and four specimens were prepared from each soil mass. The variation in specimen height is illustrated in Figure 6.7.

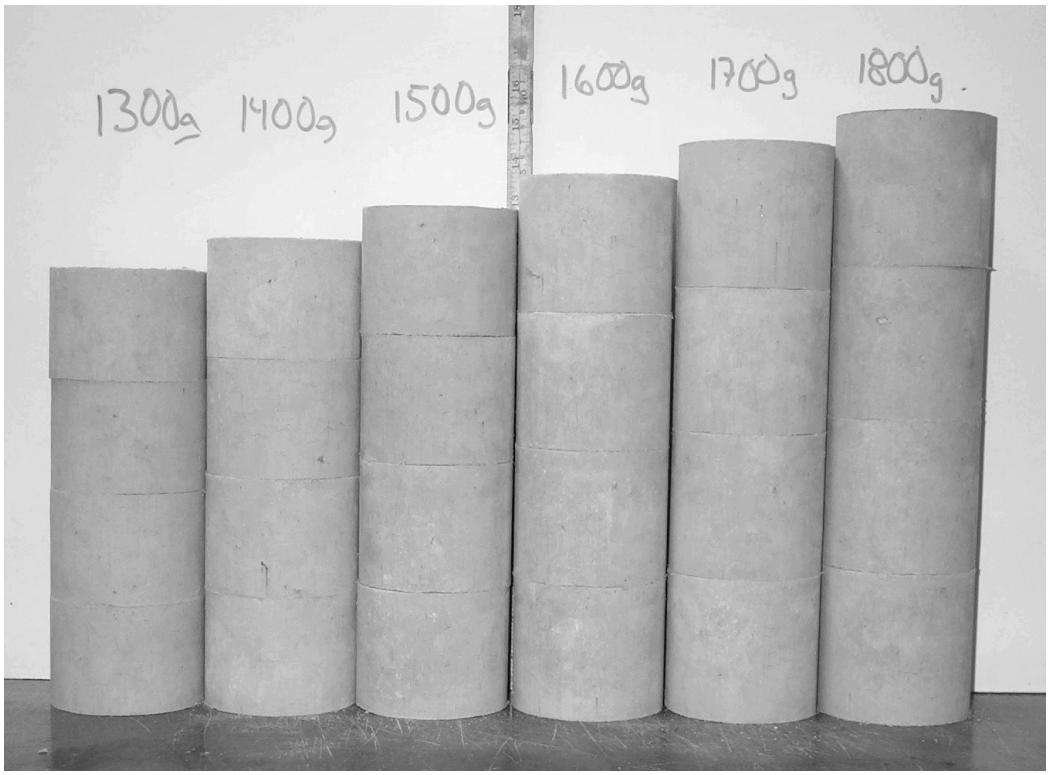


FIGURE 6.7 *Photo showing the height of four MCV samples stacked on top of each other. The material used was E22FN. The different masses of the tested specimens varied from 1300g to 1800g with 100g increments. Note that 1500g is the standard.*

The procedure was repeated with the Östra Torn material (cf. Table 7.1) to see if the results were dependent on soil type, see Figure 6.8. The results show similar behaviour for both soils and no further investigation was performed. However, the regression lines do not show au unambiguous connection between soil mass and MCV. This illustrates the difficulty in translating old Swedish MCV data to correct MCV data according to the present standard.

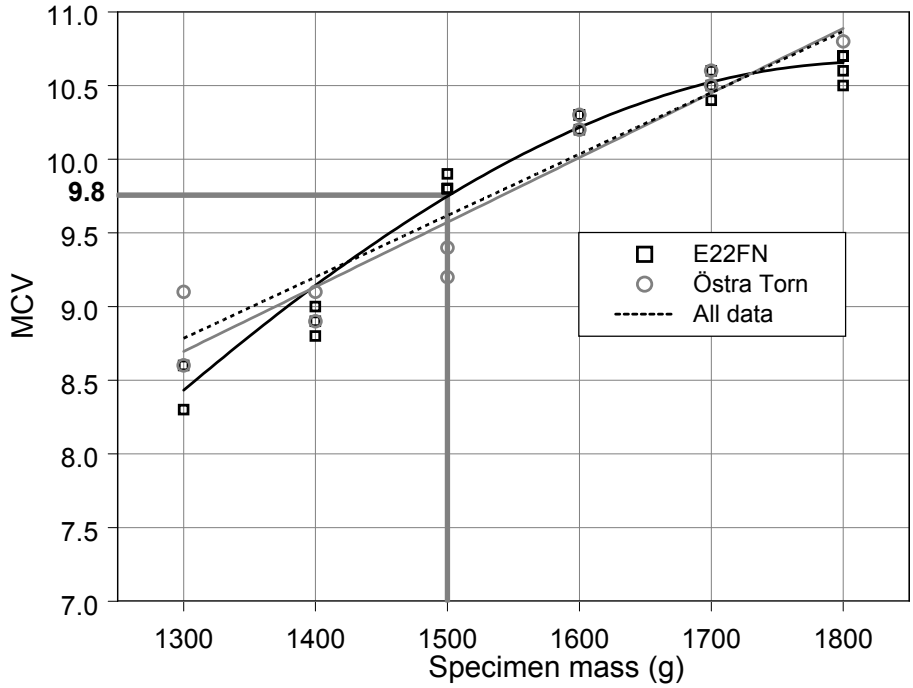


FIGURE 6.8 *MCV as a function of specimen mass for the E22FN and Östra Torn materials. (After Lindh and Winter, 2003).*

However, for the two tested soils the regression gives an approximate method to correct the value from old data if the used specimen mass is known. The regression model is presented in Equation 6.2.

$$MCV = 3.367 + 0.00417 \cdot specimenmass \qquad \textbf{(EQ : 6.2)}$$

where the specimen mass is in grams.

The regression model explains 89% of the variation. The approximate correction is 0.4 MCV for each 100 g specimen mass. For some previous earthwork projects 1800g of specimen mass was used in the MCV tests. The relation shows that the MCV has then been overestimated by about 1.25.

6.4.3 Vibratory compaction

Vibratory compaction was used mainly to produce stabilised soil specimens. The method employed is modified compared to SS 02 71 09 (Anon., 1994d). The main purpose of modifying the method was to obtain a compaction equipment that efficiently produces specimens which are confined during curing. In the modified method, the specimens are compacted in PVC-tubes instead of steel tubes. The dimensions are changed to a diameter of 103 mm to fit the PCV-tube and a height of approximately 230 mm. The PVC-tube is supported by an outer steel tube, see Figure 6.9. The latter is used to prevent radial expansion during compaction.

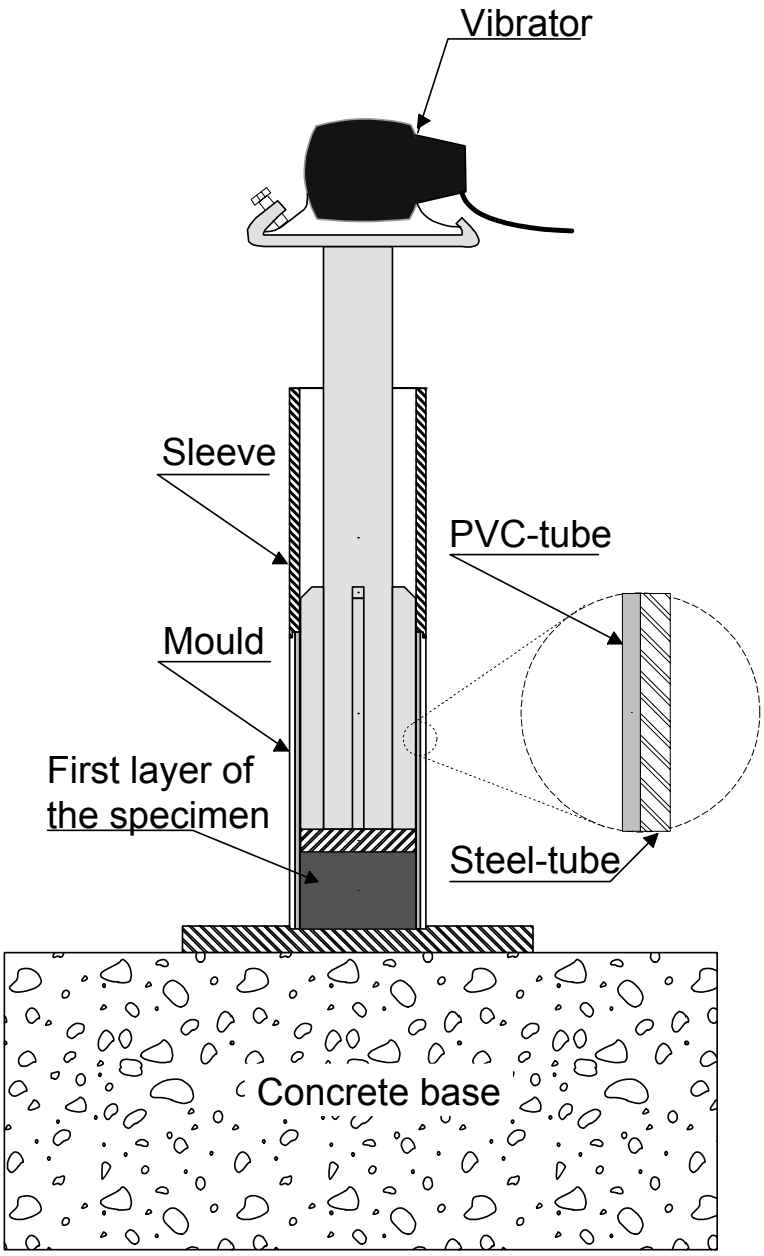


FIGURE 6.9 *Vibratory compaction equipment used in this study.*

A specimen is built up of three compacted layers, each of 1.5 kg material and compacted for 120 seconds. The surfaces of the two first layers are scarified to ensure interaction between the layers. After compaction, the specimen in the PVC-tube is removed from the compaction equipment and placed in a jig in a concrete saw and there cut to a height of 206 mm. The PVC-tube prevents the sample from being smeared with water during the sawing.

6.4.3.1 Comparison between vibratory and MCA compaction

A comparative test on stabilised soil was performed with the modified vibratory compaction equipment and MCV method respectively. Two different soils used in these tests and six different binder recipes were tested, see Figure 6.10 and Figure 6.11. The six different binder recipes are presented in Table 6.2.

Table 6.2: *Different binder recipes used in the compaction comparison.*

Recipe	Binder
1	cement
2	cement (50%) + slag (50%)
3	slag
4	slag (50%) + lime (50%)
5	lime
6	lime (50%) + cement (50%)

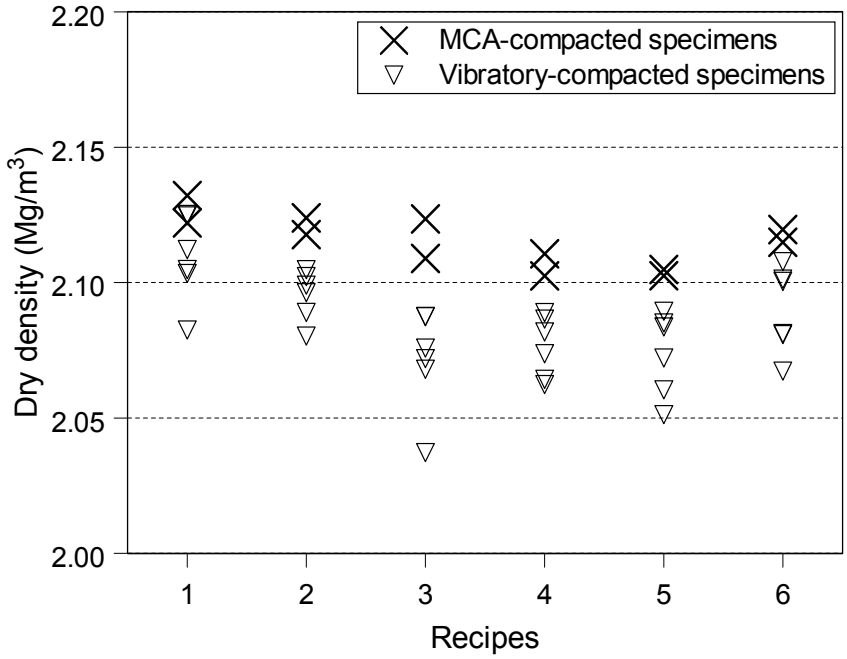


FIGURE 6.10 *Dry density for MCV and vibratory-compacted specimens using six different binder recipes. Soil type Bromölla. (Lindh, 2001)*

In Figure 6.10 it is seen that the vibratory compaction produces specimens with a slightly lower dry density for the stabilised Bromölla soil. However, for the stabilised Petersborg soil, the vibratory compaction method produces slightly higher dry density compared to the MCV method, see Figure 6.11 (Lindh, 2001).

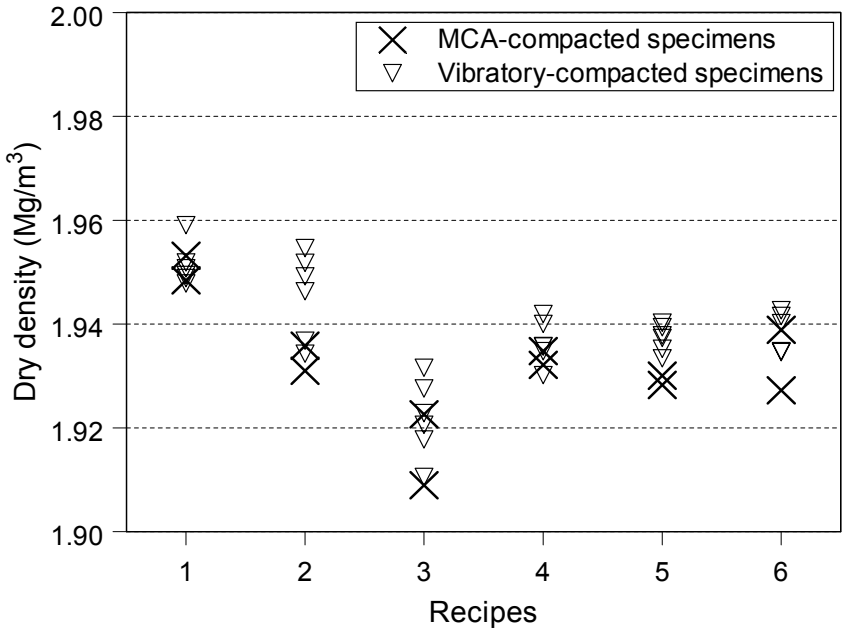


FIGURE 6.11 *Dry density for MCV and vibratory-compacted specimens using six different binder recipes. Soil type Petersborg. (Lindh, 2001)*

6.5 Strength testing

Three different test methods were used in the strength testing.

- Unconfined compression tests
- Indirect tensile tests
- Triaxial tests

6.5.1 Unconfined compression test

The unconfined compression tests were performed according to Swedish standard SS 02 71 28 (Anon., 1992) except for the deformation rate. The Swedish standard prescribes a rate of 1 - 2 mm/min for a sample with a height of 100 mm, which gives a deformation rate of 1 - 2%/min. This can be compared with British standard (Anon., 1990e), where the deformation rate shall be 2%/min. In the DIN/ISSMGE recommendations (Anon., 1998b) the test shall be performed with a constant rate of strain of 0.5% to 2% per minute.

In this study several deformation rates were used based on type and dimension of the specimen. For the double-height specimens, i.e. two MCV compacted specimens on top of each other, a rate of 4 mm/min was chosen. The height of those specimens varied between 155 and 175 mm, which gives a strain rate of 2.3 - 2.6%/min. For single-height specimens, i.e. only one MCV specimen, a rate of 2 mm/min was chosen and for Proctor-compacted specimens with a specimen height of 116.5 mm a rate of 3 mm/min was chosen, which corresponds to a strain rate of 2.6%/min. The main reason for the deformation rates usedwas a recommendation based on double MCV specimens based on 1800g, cf. "The effect of sample preparation on MCV" on page 116. Since this mistake was considered minor and a large test series had been performed it was decided to keep this deformation rate.

The shear strength was evaluated according to Equation 6.3.

$$c_u = \frac{P_f}{2A_0}(1 - \varepsilon_f) \qquad \textbf{(EQ : 6.3)}$$

Were:

- c_u = undrained shear strength (kPa)
- P_f = axial load at failure (kN)
- ε_f = relative axial deformation at failure
- A_0 = initial sample area (m^2)

In this evaluation the shear plane is assumed to form an angle of 45 degrees with the horizontal plane. If the shear plane in question has an angle greater than 45 degrees then the shear stress in the actual shear plane will be lower. For a shear plane with an angle of 60 degrees the difference will be about 15% (Anon., 1992).

In this study the unconfined compression strength test was performed with friction-free ends, cf. Rowe and Barden (1964). These were obtained by polished stainless steel plates at the top and bottom of the specimen. Paraffin oil was used as lubricant to further decrease the friction between the specimen and the end plates.

In some cases with specimens of high water content, the paraffin oil resulted in high suction between the soil sample and the end plates. The suction force on the end plates was identified, since the bottom plate was lifted together with the specimen.

A certain limitation in using unconfined compression tests in combination with double MCV compacted specimens was found during

this study. When testing specimens with low MCVs the failure pattern in the specimens assumed the shape of two barrels, see Figure 6.12.

Since MCA compaction compacts the specimen in a single layer, there is most likely a varying density profile, with the highest density closest to the rammer. To minimise the influence of this in the strength tests, all specimens were prepared in the same way, i.e. the ends that had been closest to the base plate were turned towards each other, cf. Figure 6.3.

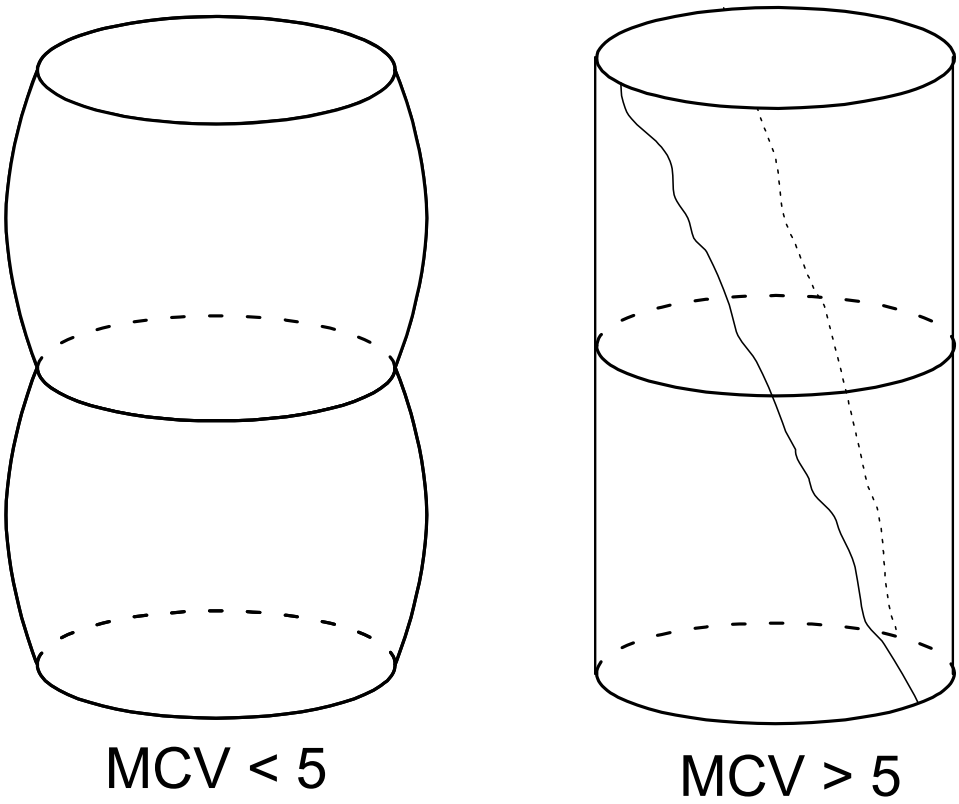


FIGURE 6.12 *Schematic failure patterns for double MCV specimens in unconfined compression tests. For MCV < 5 specimens, the failure pattern generally takes the shape of two barrels placed on top of each other.*

A possible explanation for the waist on the composite specimen may be that during compaction the boundary of the specimen obtains higher compaction due to higher pore pressure at the centre of the MCV specimen. This phenomenon is limited to low MCVs i.e. high water contents.

The friction and suction forces between the specimen and the end plates do not explain the waist of the specimen. It could however, cause shear forces that partially explain the barrel shape failure pattern of the specimen. A combination of high pore pressure at the center of the sub-specimens and a better compaction at the ends and the boundary between the specimens could possibly explain the barrel shape of wet specimens.

FIGURE 6.13 *Photo showing failure planes for four composite MCV specimen with MCV > 7. The specimens consist of compacted E22FN material stored 28 days (without drying) before testing. The interaction between the two MCV specimens one on top of each other is clearly visible.*

For MCVs greater than approximately 5, the interaction between the two specimens (cf. Figure 6.12) appears to be good and the composite specimen seems to act like a homogeneous specimen, see Figure 6.13. The limiting MCV at which the two specimens behave as a homogeneous specimen, is to some extent dependent on the soil type. This was not studied further, since the limiting MCV value is below 6 and the soil is thereby too wet for earthwork.

6.5.1.1 The effect of end friction in strength testing

Rowe and Barden (1964) studied the effect of frictionless ends in triaxial testing, see Figure 6.14 . In their study they also studied the effect of different strain rates. They found that a strain rate of 0.03% per minute gave a more uniform result compared to a strain rate of 2% per minute.

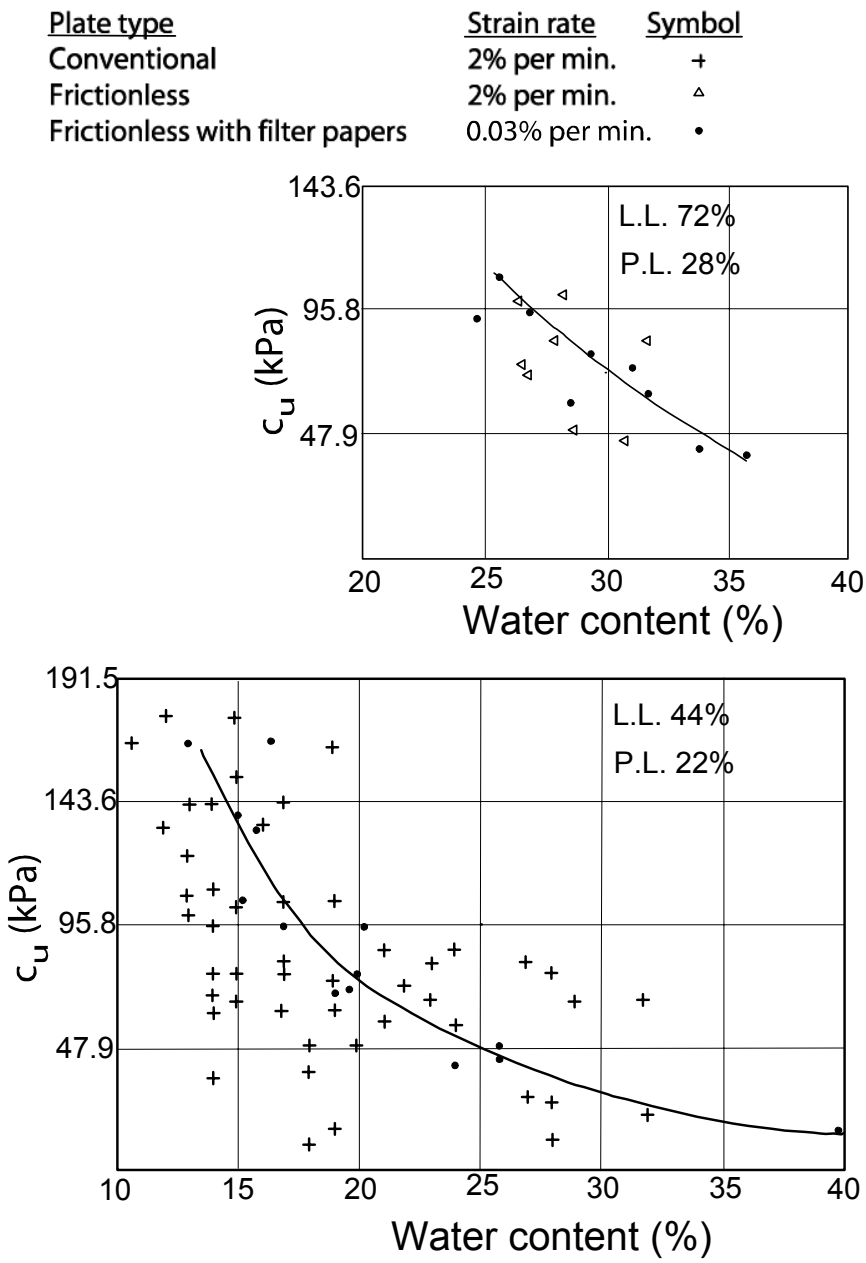


FIGURE 6.14 *Effect of plate and strain-rate on undrained strength of fissured clays. After Rowe and Barden (1964).*

The effect of sample height and effect of free and fixed ends was studied by Jacobsen (1970), see Figure 6.15.

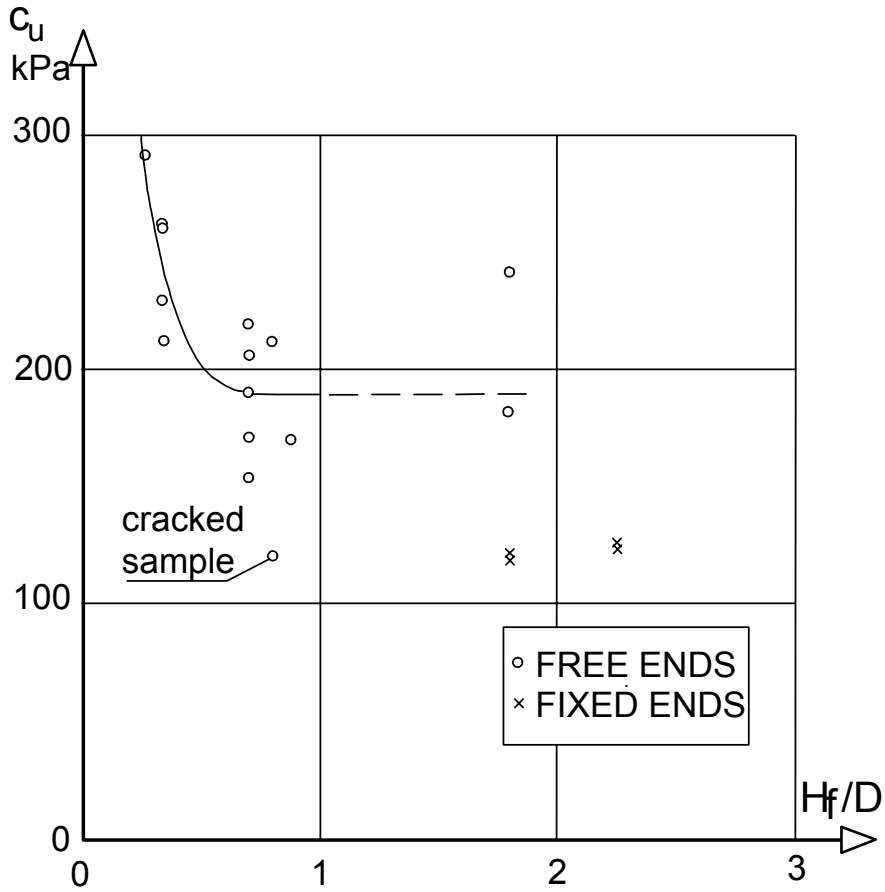


FIGURE 6.15 *The effect of sample height on the undrained shear strength with free and fixed ends. Hf shows the sample height at failure and D the sample diameter. The data were obtained for Kratbjerg clay till. After Jacobsen (1970).*

The results presented in Figure 6.15 shows the influence of smooth preasure heads. Jacobsen (1970) used unconfined compression tests and the results showed that with a height (H) > diameter (D) the specimens developed radial cracks that transformed them into a series of smaller sub

specimens. However, in the present study, this effect was found in both double and single height MCV specimens.

For unconfined compression test that uses the same diameter as height Jacobsen (1970) proposed a relation between unconfined compression tests and vane test, cf. Equation 6.4.

$$c_u \sim 0.93 \cdot c_v \qquad \textbf{(EQ : 6.4)}$$

where c_v is the undrained shear strength from vane test, c_u and c_v in kPa.

6.5.1.2 The effect of strain rate in strength testing

Studies on the influence of strain rate shows that a relative strain rate increase of over 10^4 times may increases the shear strength by more than 70%, see Figure 6.16. On the other hand, the influence of a smaller variation of the strain rate of say ±50% is insignifficant.

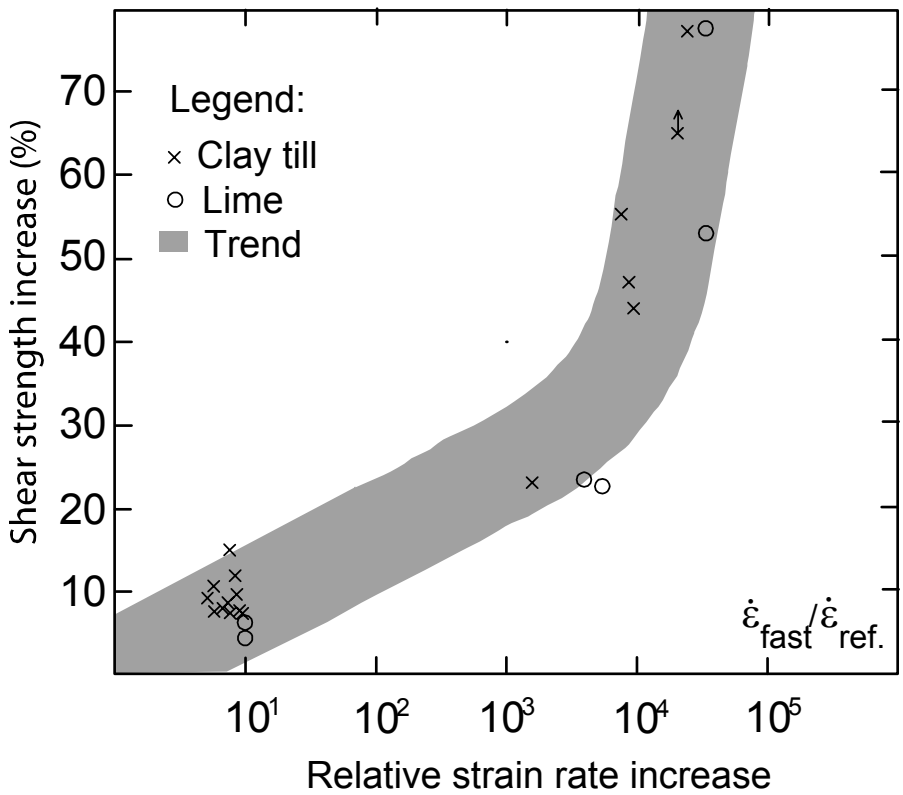


FIGURE 6.16 *Shear strength increases as a function of relative strain-rate increase. After Kristensen et al. (1992).*

Börgesson (1981) has studied the influence of the rate of shear for silt and clay. The results shows that the obtained shear strength in silt is less dependent on deformation rate than that of clay, cf. Figure 6.17.

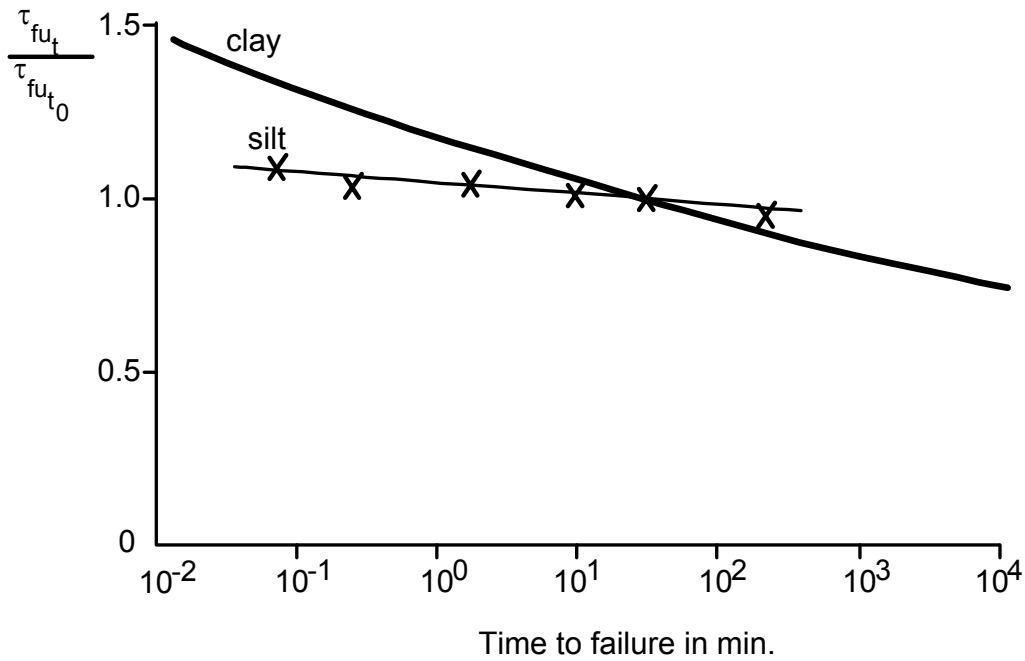


FIGURE 6.17 *Rate dependence of silt compared with the rate dependence of clay. (After Börgesson, 1981. The clay data is originally from Torstensson, 1977). The results are from vane testing.*

In this study, tests on the effect of deformation rate were performed at three different rates; 0.4 mm/min; 2 mm/min and 4 mm/min. This corresponds to a strain rate of approximately 0.3%/min; 1.2%/min and 2.5%/min respectively. A total of nine specimens were prepared, three for each deformation rate. All the specimens were identically prepared. The result showed approximately the same shear strength for the different deformation rates but the variation decreased with increasing rate of strain, see Figure 6.18. However, this small test series was a check

performed only for comparisons and should not be used for any general conclusions.

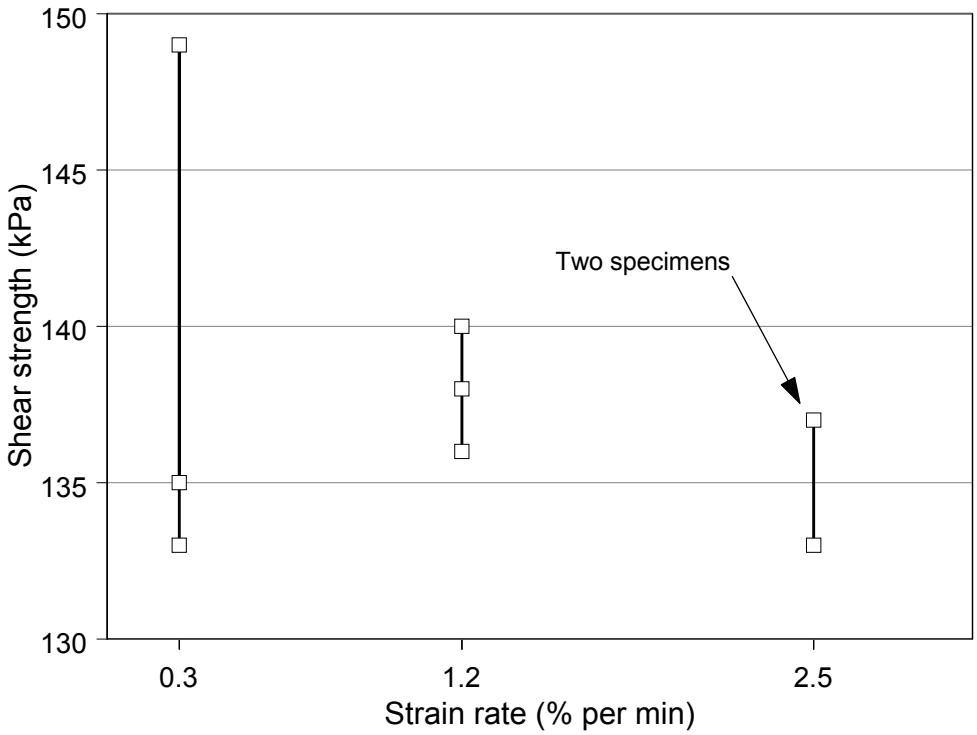


FIGURE 6.18 *Measured shear strength at different strain rates. Three specimens were tested at each deformation rate. Each specimen consisted of two MCA compacted sub-specimens.*

6.5.2 Indirect tensile test

This test was used only for stabilised soil. The indirect tensile tests were performed in accordance with prEN 13286-42 (Anon., 1999b). The apparatus consists of a load frame equipped for tensile testing, see

Figure 6.19. The load frame is computer controlled, using LabVIEW software. The specimens in the tests were cylinders with slenderness ratios between 0.8 and 2.0. The diameter can be 50 mm, 100 mm, 150 mm or 160 mm with a tolerance of ± 2%. In this study only 100 mm was used. The packing strips had a width of 10mm.

Before testing, the specimens were adjusted in the testing machine in such a way that they make contact with the packing strips.

The load was then applied in a continuous and uniform manner without shock in order to obtain a uniform increase in stress of 0.01 ± 0.005 kPa/s. The indirect tensile strength (R_{it}) is calculated from the force of failure, F, by the following formula:

$$R_{it} = \frac{2F}{\pi HD} \qquad \textbf{(EQ : 6.5)}$$

where:

- R_{it} is the indirect tensile strength
- F is the force that produces failure
- H is the length of the specimen
- D is the diameter of the specimen

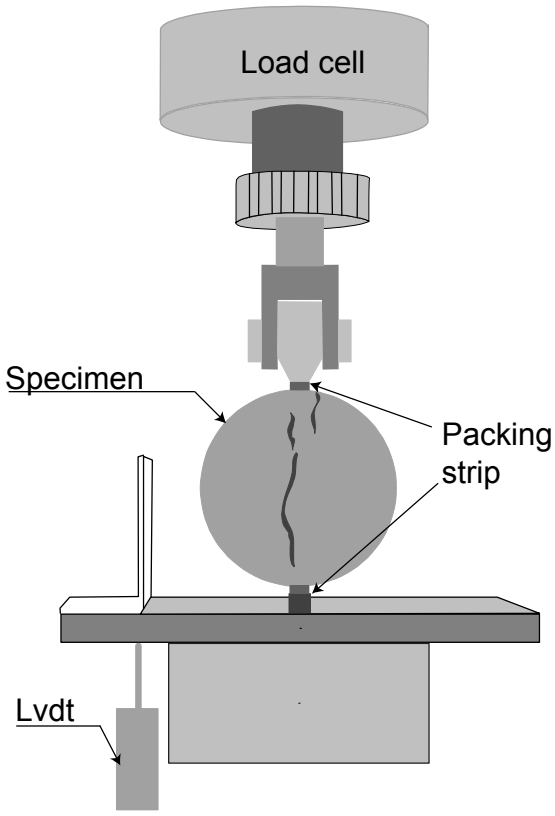


FIGURE 6.19 *Indirect tensile-strength test equipment. The testing equipment is mounted in a load frame.*

6.5.3 Triaxial tests

The triaxial tests have been performed in three different ways; Unconsolidated, undrained; consolidated undrained and drained. The applied cell pressures were 20 kPa, 80 kPa and 200 kPa.

6.5.3.1 Unconsolidated, undrained triaxial tests

The unconsolidated undrained triaxial tests were performed with the same deformation rate as the unconfined compression tests, i.e. 4mm/min. As for the unconfined compession tests, these triaxial tests were performed with non-saturated specimens. Due to the fast deformation rate, the ordinary test equipment could not keep a constant cell pressure for stiff soils. The cell pressure in these tests increased rapidly depending on the dilatability of the material and the usual digital volume pressure control units were not fast enough to compensate for the volume increase. However, this was remedied by using an air/water pressure control unit during the shear phase.

The specimens were prepared in pairs. They were compacted in the MCA and then put together to form a combined specimen. This specimen was placed in the triaxial cell. The desired value of cell pressure was applied and then the shearing phase was started. The unconsolidated tests were finished within one hour after compaction.

6.5.3.2 Consolidated undrained triaxial tests

For the consolidated undrained tests, the chosen consolidation time was 48 hours. The specimens were supplied with vertical paper drains between the specimen and the rubber-membrane. The paper drains (cf. Figure 6.20) make the consolidation time of 48 hours sufficient.

FIGURE 6.20 *Photograph showing a specimen with vertical paper drains.*

The consolidated undrained triaxial tests were performed with filter stones providing very rough end surfaces compared to those in the unconsolidated testing. According to Rowe and Barden (1964), the use of rough end plates results in greater scatter in measured shear strength compared to frictionless plates, cf. Figure 6.14 . Jacobsen (1970) showed

that fixed ends resulted in lower shear strength compared to free ends, cf. Figure 6.15.

6.5.3.3 Consolidated drained triaxial tests

The drained tests were performed to analyse both the difference between drained and undrained behaviour and the long-term strength at full saturation. The drained shear strength is, in this country, considered to be a minimum strength for compacted till.

The drained tests were performed with a back pressure of 300 kPa to ensure complete saturation of the specimens. The strain rate used was 0.014%/min.

6.6 Special tests

6.6.1 Initial consumption of lime

The initial consumption of lime (ICL) test was performed according to British standard (Anon., 1990f). In this test the amount of lime that results in a pH of 12.4 in the soil is determined. This amount of lime is regarded as the minimum to maintain the reaction between the lime and the reaction components of the soil to be stabilised.

6.6.2 Freeze and thaw tests

In the literature, there are large number of different testing methods proposed for determining frost heave and freeze durability of soils (McCabe and Kettle, 1995; Garand and Ladanyi, 1982; Balduzzi, 1967; Wäre, 1974; Esmer *et al.,* 1969 among others). Most of these tests are developed for natural soils and there are a few methods developed especially for stabilised soils. The latter tests are often similar to the wetting and drying tests. The specimens are then subjected to immersion in water followed by cycles of freezing for 24 hours and thawing for 23 hours. After each thawing cycle the specimens are brushed and the loss in weight is determined (Sherwood, 1993).

Another test method designed to measure the freeze durability of stabilised soils compares the loss in compressive strength after a specific number of freeze and thaw cycles (Anon., 1987). The frost-heave susceptibility testing methods measure the response at different temperature conditions to determine the soil's heaving pressure or heaving rate, water-intake and maximum heave.

The specimen-testing unit used in this study is a direct frost-heave susceptibility unit, which can hold 16 specimens in each test series. The frost heave is measured on each specimen and the temperature is measured at 40 different locations. The bottom of the unit is a stainless-steel tub containing water. The bottom of the tub is uninsulated and vented to allow the water to be cooled by the forced-air stream produced in a climate-controlled test chamber. A regulated immersion heater is used to control the water temperature and a circulation pump ensures that the temperature of the water in the bath is evenly distributed. The temperature is kept as constant as possible, at 4 - 5°C. The specimen holder is placed on top of the tub, allowing 5 to 10 mm of the specimens to be immersed in the water. The testing unit is shown in Figure 6.21 and Figure 6.22. No special arrangements are used to reduce the side friction between the PVC-tubes and the specimens.

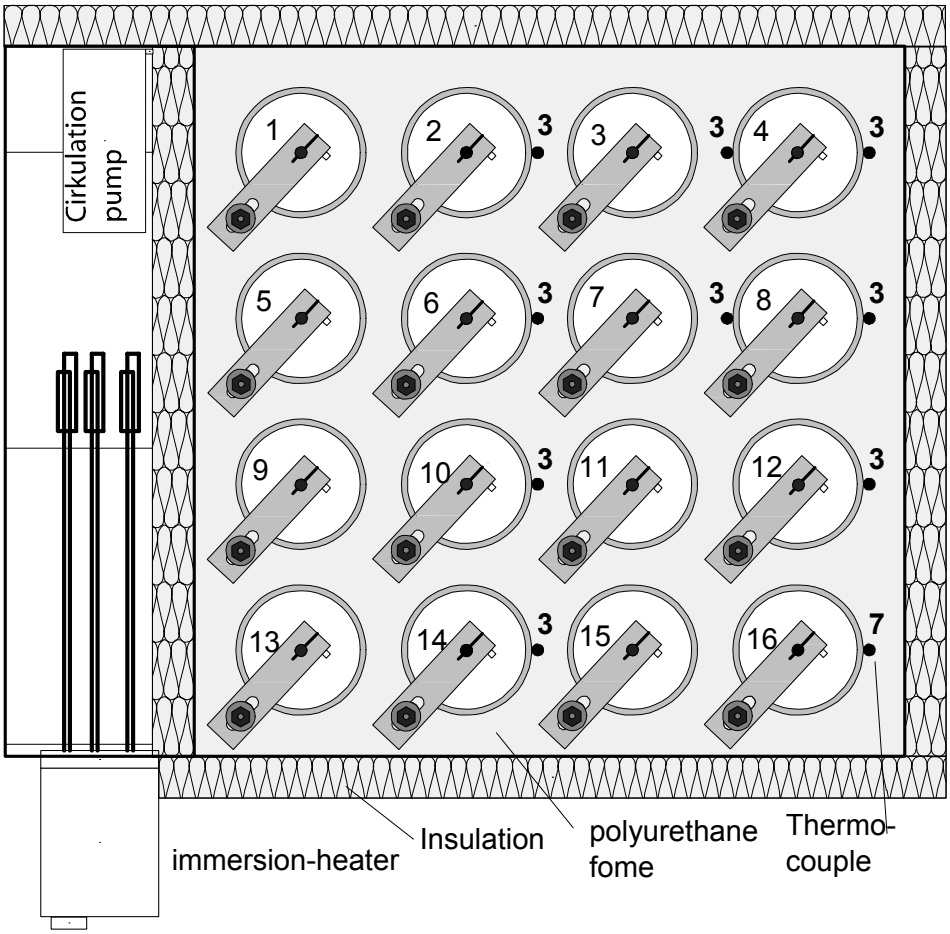


FIGURE 6.21 *A plan view over the testing unit. A bold number shows the amount of thermocouples at respective locations.*

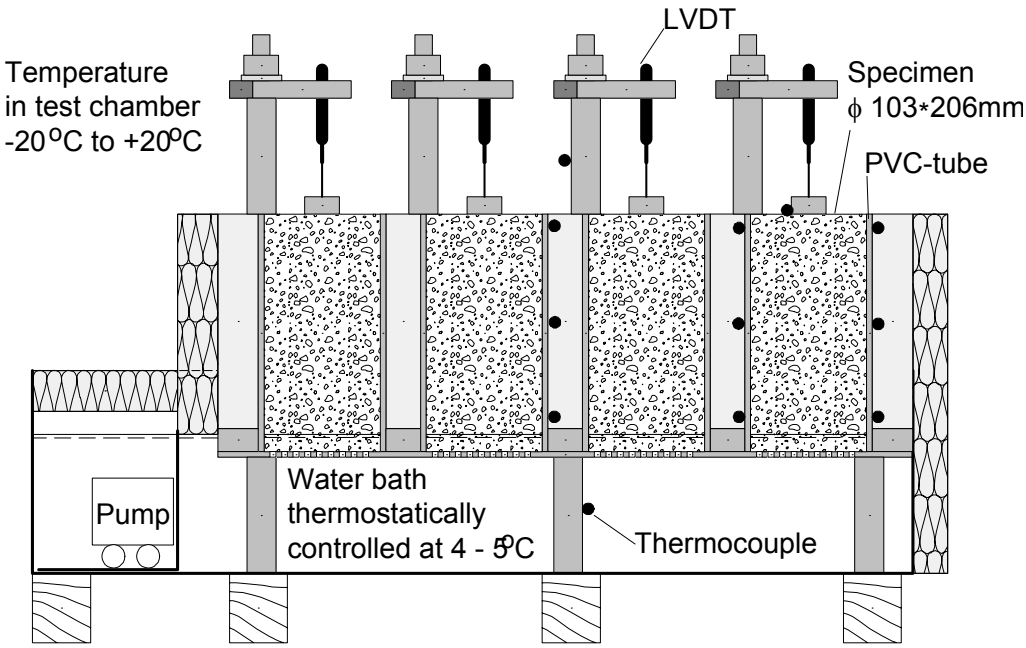


FIGURE 6.22 *Section through the freez-test equipment with the water tub at the bottom and the soil specimens within the PVC-tubes. At the top of the specimens the linear voltage displacement transducers (LVDT) are placed. The forced-air stream produced by the climate chamber also circulates below the tub to cool the water.*

6.6.2.1 Testing procedure

Stabilised specimens need to be stored under special conditions to prevent stress release or loss of water during the curing. To achieve the best possible homogeneity the specimens were compacted with the special vibratory compaction equipment, described in section 6.4.3 .

After compaction the specimens are cut to the correct height. Subsequently the specimen ends are covered with several layers of paraffin to ensure that there is no loss of water during the storing time. Following the curing period, the paraffin covers are removed and the specimens are placed in the testing equipment. During the first phase of the test, the specimens were allowed to soak up water from the lower ends where approximately 10 mm was immersed in water. This phase lasted for 20 days and the temperature of air and water was approximately 20°C. In conjunction with the soaking, a linear voltage displacement transducer (LVDT) is placed on each specimen. The temperatures are measured with copper-constantan thermocouples type T. In the first test series, the tops of the specimens were not covered at all and this resulted in freeze dehydration of the samples. This problem was solved by applying a silicone coating on top of each specimen that covered both the soil and the end of the PVC-tube. The silicone coating remained elastic even at very low temperatures.

The starting temperature was approximately 20°C and the temperature was then lowered approximately 2°C every 24 hours until the pre-determined minimum air temperature was reached. This was set at -17^{0}C and the temperature was then kept constant at this interval for approximately 2 weeks. After this period, the air temperature was increased by 8°C every 24 hours until it reached 20°C. When this cycle was completed, two specimens of each type were removed and replaced by dummies. The strength of the removed specimens was measured by unconfined compression tests. The second cycle freeze-thaw was then started and the temperature was lowered 8°C each 24 hours until it reached -14°C. This temperature was chosen in order to produce a second ice lens at a different location from that of the first ice lens. After the second cycle, four more specimens were removed and replaced with

dummies. The freeze-thaw test was stopped after three cycles. Both frost heave and temperatures were measured every 5 minutes to enable a detailed recording of the process.

During the formation of an ice-lens there is a rapid heat increase in the soil. This is called latent heating. The freeze-test equipment developed for this study is able to measure the heat increase in a specimen, see Figure 6.23.

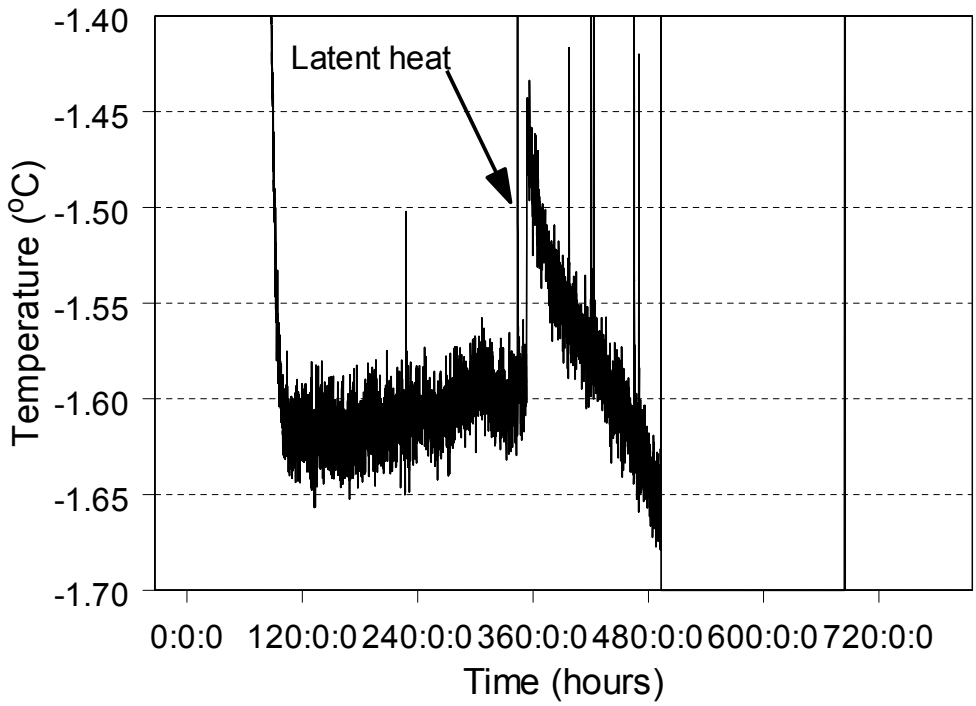


FIGURE 6.23 *Latent heat during the formation of a ice-lens.*

The plot shows a temperature increase of approximately 0.1°C during the formation of the ice-lens. After the initial increase, the temperature starts to decrease, cf. Figure 6.23.

6.6.3 Non-destructive tests

Non-destructive tests were performed in order to measure the changes in modulus and resitivity with time. This type of testing was only performed on stabilised soil samples.

6.6.3.1 GrindoSonic impact excitation

In order to collect more data, the non-destructive GrindoSonic impact-excitation test method was used to measure differences between different stabilising agents and changes with time. The impulse-excitation technique is a dynamic method based on the analysis of a transient vibration of the test object resulting from a mechanical impact. The method is normally used to evaluate the elastic modulus of the specimen, but in this case the geometry of the specimens did not allow this interpretation. Instead, the resonance frequency was used as a relative measure for a comparison between the specimens. A GrindoSonic MK5 "Industrial" measuring device from J.W. Lemmens N.V. was used to measure the natural frequency. The instrument is programmed to perform a time analysis of the signal to identify the fundamental component of the vibration. The test arrangement is presented in Figure 6.24.

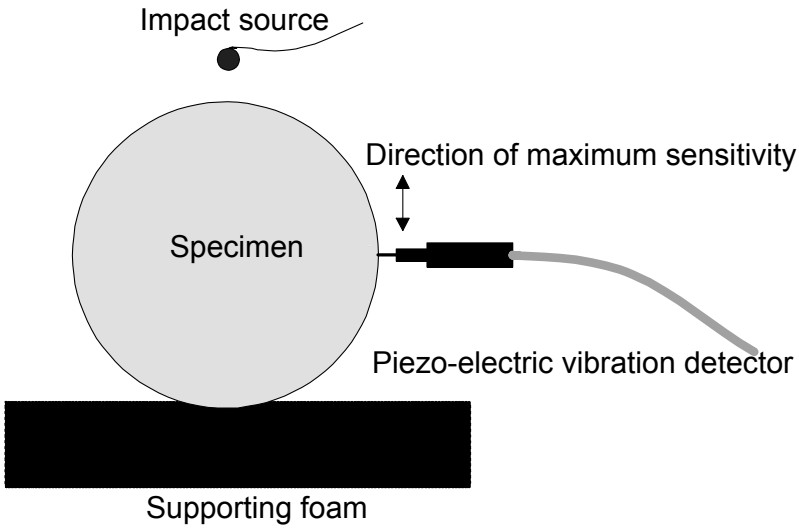


FIGURE 6.24 *Measuring arrangement with the GrindoSonic MK5 device.*

The equipment will eliminate any undesired effect of specimen movement and only retain the signal coming from its vibrations. Furthermore, it will monitor the decay of the signal and evaluate when the signal is in the linear range of the amplifier and at which moment processing should stop because the signal is about to fade out. The time measurements are stored in the memory and as soon as the minimum usable signal level has been reached, the natural frequency is computed. The result is presented as a constant with a 4-digit resolution in either frequency (Hz - kHz) or period (ms - ms) (Anon.).

6.6.4 XRD

X-ray diffraction was performed to identify the clay minerals. A soil sample was pre-treated with distilled water and a dispersant. The dispersant used in this case was natriumpyrophosphate. The treated soil was allowed to settle for approximately 7 to 8 hours. After this a smaller amount of the soil water solution was removed for specimen preparation. For each soil three specimens were prepared. The first specimen was scanned without any further treatment. The second specimen was treated with ethylene glycol to expand any swelling clay mineral. The third specimen was heated to 550°C as pre-treatment. This treatment was performed to identify kaolinite since this mineral disintegrates when heated.

6.7 Field tests

The field tests were performed to verify the effects found in the laboratory. Mainly established test methods were employed. However, non-destructive testing was also employed for validation of both the methods and the stabilised soils.

6.7.1 Plate-load test

The plate-load test were carried out according to the SNRA description no 606:1993 (Friborg and Hagert, 1993).

6.7.2 Pulverisation

Pulverisation tests were performed according to British Standard (BS 1924:1990). This is a method to control how the rotovator has managed to break up the soil. Indirectly, it also indicates how homogeneous the soil-binder blend is. The specimen with a mass of approximately 1 kg, is spread over a 5 mm sieve and shaken gently. The weight of the soil remaining in the sieve is then determined. Thereafter, all lumps are broken up until all material finer than 5 mm is separated. The sample is replaced on the 5 mm sieve and shaken until all fine material has passed through it, and the remaining material is weighed.

The degree of pulverisation (P, in %) can be determined by Equation 6.6.

$$P = 100 \cdot (m_1 - m_2)/(m_1 - m_3) \quad \textbf{(EQ : 6.6)}$$

where:

m_1 is the total mass of the sample
m_2 is the mass of the unbroken material retained on the sieve
m_3 is the mass of the material finally retained in the sieve

Specifications from Department of Transport (UK) require that at least 90% of the stabilised soil shall pass the 28 mm sieve and a minimum of 30% is shall pass the 5 mm sieve (Anon., 1990b).

6.7.3 Resistivity

The surveying was conducted as two-dimensional resistivity imaging, also called continuous vertical electrical sounding (CVES), which is presented as cross sections of the resistivity of the ground. The ABEM Lund Imaging System, a computer-controlled multi-electrode system was used for the data acquisition. Four electrode cables with 21 take-outs each were laid out in a line using an electrode spacing of 0.25 metre. The lines were extended using a roll-along technique (Dahlin, 1993) and the Wenner measuring array was used.

The data was processed using inverse numerical modelling (inversion), in which a finite difference model of the subsurface resistivities is automatically adjusted to minimise the residuals between the model response and the measured data. The software Res2dinv (Loke, 1999) was used for the inversion.

7 Results and discussion

7.1 Laboratory results - fine- and medium-grained tills

The tested soils are from different locations in Southern Sweden. All of them were excavated during the construction of roads or earthworks, in order to collect typical soils used in real projects. The amount of collected soil at each site varied from a few hundred kilograms to more than 1500 kg depending on the type of investigation. The soils were all fine-grained or medium-grained tills. The PBL (Pers blend i.e. the author's blend) soil was manufactured in the laboratory by mixing different soil types.

The main purpose of manufacturing PBL was to produce more material for a soil-stabilising test. However, it also gave the possibility to determine if a blended material behaved differently when compared to natural soils. During earthworks it is common for soils to be transported from different locations and placed in an embankment. Depending on the construction method, the fill material becomes more or less blended. Most of the excavations in Sweden are made with excavators in combination with articulated haulers.

The tested soils are presented in Table 7.1.

Table 7.1: *Soils tested in this study*

Excavation site	Denoted
Yttre Ringvägen, Malmö	Petersborg
Yttre Ringvägen, Malmö	Nordost
Yttre Ringvägen, Malmö	S7
Yttre Ringvägen, Malmö	S14
Lorensborgsgatan, Malmö	Lorensborg
Citytunneln, Malmö	Hyllie
Östra Torn, Lund	Östra Torn
Sturup	Sturup
E22, Flyinge Norra	E22FN
E22, Hurva	E22 Hurva
E22, Bromölla	E22 Bromölla
E18, Örebro	Örebro
Various	PBL

7.1.1 Particle size

The particle-size distributions for the tested soils are presented in Figure 7.1.

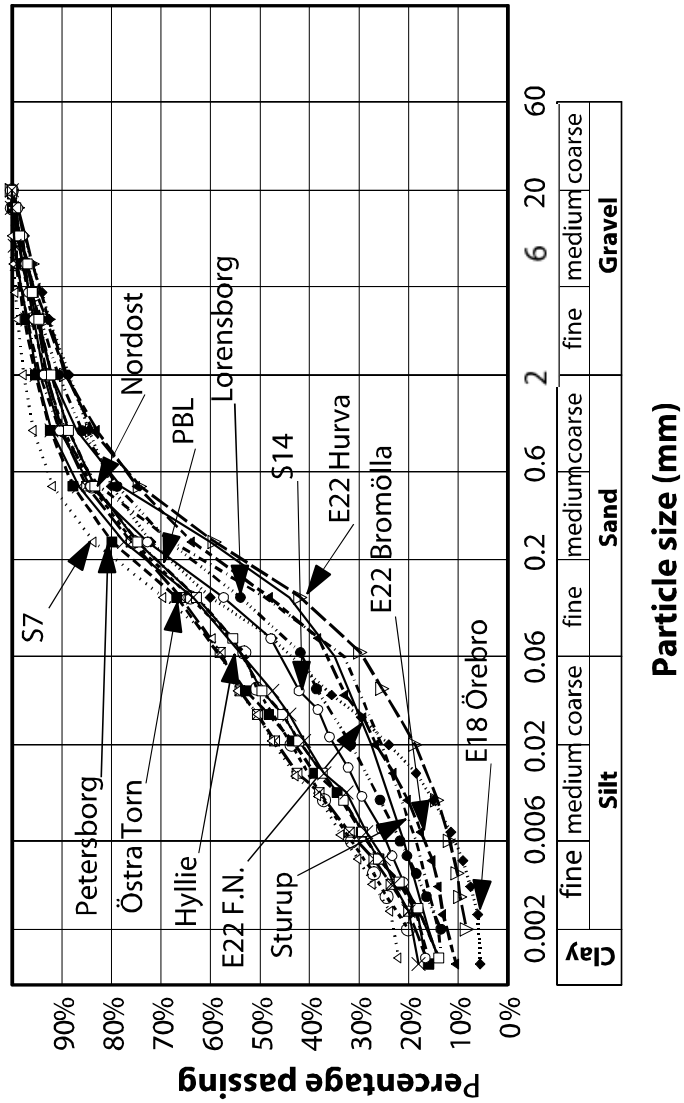


FIGURE 7.1 *Particle-size distribution for the tested soils.*

Table 7.2: *Soil classification*

Soil	Soil type	Fines (%)
Petersborg	silty clay till	59
Nordost	clay till	54
S7	clay till	59
S14	clay till	45
Lorensborg	clay till	42
Hyllie	sandy silty clay till	54
Östra Torn	sandy silty clay till	59
Sturup	clayey sand till	33
E22FN	clayey sand till	35
E22 Hurva	clayey sand till	30
E22 Bromölla	sandy silty clay till	36
Örebro	clayey sandy silt till	43
PBL	silty clay till	54

7.1.2 Water content

The natural water content for the tested soils is presented in Table 7.3. The natural water content is here defined as dug at the test pits.

Table 7.3: *Natural water for the tested soils.*

Soil	w_N (%)
Petersborg	14.6
Nordost	No data
S7	No data
S14	No data
Lorensborg	No data
Hyllie	15.1
Östra Torn	19.6
Sturup	14.4
E22FN	8.0
E22 Hurva	13.4[a]
E22 Bromölla	No data
Örebro	No data
PBL	"16.1"

a. After transporttation of the soil, excess water was expelled from the soil and was found on top in the the containers on top of the soil. When the excess water was removed w_N was 11.9 %.

The soils were stored in a climatic chamber to ensure equal conditions for the different tests. To study the variation of the natural water content for some of the soils, data for a statistical evaluation were compiled and are presented in Figure 7.2.

The variation in water content appears to be approximately normally distributed for all four soils that were studied. Figure 7.2 indicates that the variation in water content is approximately ±1% for each soil. A possible

explanation for the variation in water content, apart from the inaccuracy in the determination, can be the variation of the grain size distribution in the specimen. Each specimen in Figure 7.2 consists of approximately 200 g wet soil.

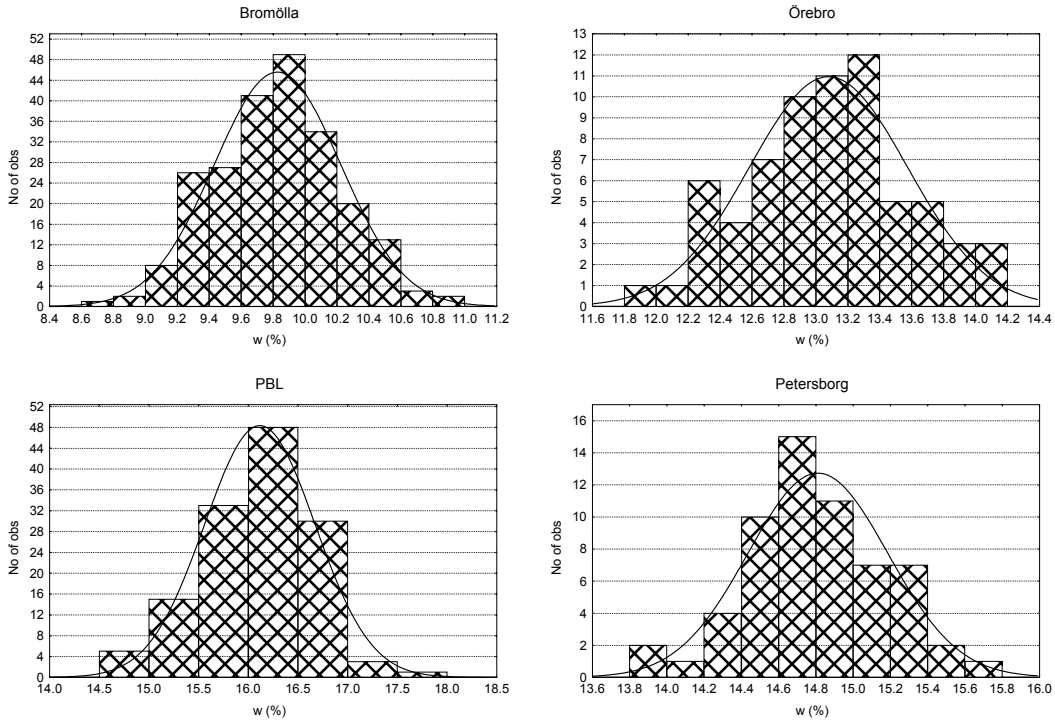


FIGURE 7.2 *Distribution of determined water-contents for the different soils, cf. Table 7.4.*

In this test series, the bowl used contained approximately 200 to 250g of moist soil. For each test two specimens were taken i.e. 400 to 500g were used. This does not fulfil the requirement in the Swedish Standard for >500g of moist soil. However, from Figure 7.1 it is clear that the D_{90} value is generally less than or very close to 2mm. According to the ISSMGE (Anon., 1998a) recommendations in Table 6.1 the minimum specimen mass should be 100g of moist soil for a D_{90} value of 2mm.

Table 7.4: *Mean water content and standard deviation for different soils (Lindh, 2000).*

Soil	Mean water content (%)	Standard deviation (%)
Bromölla	9.8	0.40
Örebro	13.1	0.50
PBL	16.1	0.56
Petersborg	14.6	0.29

7.1.3 Liquid limit and plasticity index

The liquid limit (w_L), plastic limit (w_P) and plasticity index (I_P) for the different soils are presented in Table 7.5 and plotted in Figure 7.3.

Table 7.5: *Index properties of the soils tested.*

Soil	Liquid Limit, w_L (%)	Plastic Limit, w_P (%)	Plasticity Index, I_P (%)
Petersborg	23.9	12.0	11.9
Nordost	No data	No data	No data
S7	No data	No data	No data
S14	No data	No data	No data
Lorensborg	No data	No data	No data
Hyllie	25.5	13.7	11.8
Östra Torn	25.0	16.0	9.0
Sturup	21.0	12.0	9.0
E22FN	17.0	10.0	7.0
E22 Hurva	16.1	No data	No data
E22 Bromölla	19.3	No data	No data
Örebro	No data	No data	No data
PBL	33.0	No data	No data

The plasticity chart in Figure 7.3 shows the data in relation to the Casagrade A-line and the T-line. The T-line represents the typical relation

for englacial debris and undisturbed lodgment till (Boulton and Paul, 1976).

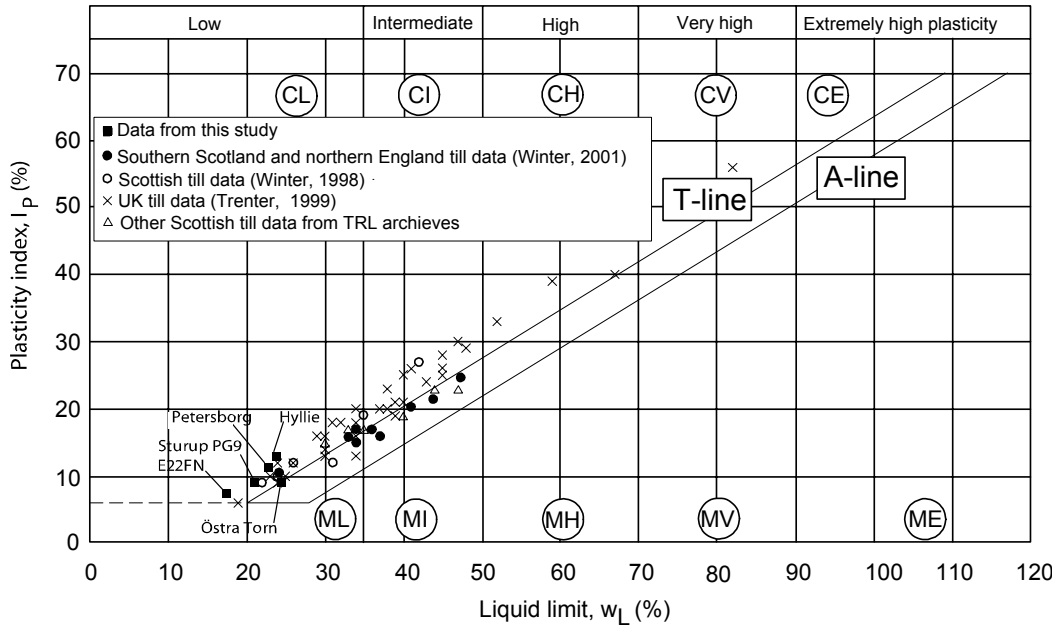


FIGURE 7.3 *Plasticity chart including data for till from southern Scotland and northern England. (After Winter, 2000).*

7.1.4 Compaction properties

Compaction is essential to achieve a good base for foundations for roads, railways and other constructions. To achieve a good result, the water content of the soil to be compacted must be within a certain range; in addition, sufficient compaction energy must be applied. Densification of fine-grained soils is about to overcome the soils' cohesion. The soils' apparent cohesion is the sum of cohesion and matrix suction. The soil-

matrix suction varies with water content and it is also depends on if the soil is in a wetting or drying phase. The soils' density increase during compaction is related to the applied compaction energy and the soils water content i.e. matrix suction.

In this study the compaction properties are determined by modified Proctor test and moisture condition apparatus (MCA). Of these two methods the modified Proctor is the most commonly used worldwide. The MCV method is used mainly in Great Britain and Sweden.

The densification of a fine-grained soil can also be problematic with too high water contents. The photograph in Figure 7.4 show a specimen compacted at different water contents. The first photo (A) shows a specimen with a MCV above 7 (w approx. 12%) and the second photo (B) (w approx. 14.5%) shows a specimen with a MCV about 3. The voids shown in Figure 7.4 (B) indicate that an adequate compaction could not be achieved at this water content.

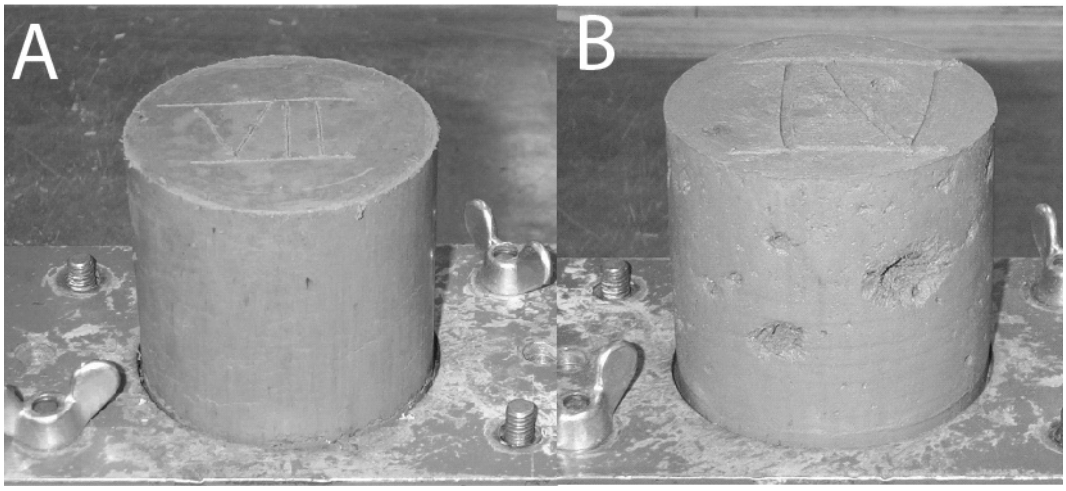


FIGURE 7.4 *Pictures of MCV specimens compacted at different water contents. Photography A shows no voids in the compacted specimen. The voids in photograph B indicate that the soil could not be sufficiently and homogeneously densified at this water content. Sturup material.*

7.1.4.1 Proctor-test results

A typical Proctor curve consists of three parts; an optimum moisture content/maximum dry density, a "wet leg" and a "dry leg". The wet leg is on the wet side of the optimum moisture content and the dry leg is on the dry side of it. The Proctor curves for all eight tested soils are presented in Figure 7.5.

The results indicate two soil groups with regard to compaction. The first group shows a higher maximum dry density and a lower OWC compared to the second group. The difference can to some extent be derived from the differences in the grain size distribution. The first group

consists of coarser soils than those in the second group, cf. Table 7.2. A relationship between the fine-particle content and maximum dry density has also been found by Jenkins and Kerr (1998). Another possible explanation for the difference between the two groups could be differences in the soil chemistry and mineralogy. This is commented on later, cf. Section 7.1.5.4 , page 202.

Figure 7.5 also shows the lines describing 0, 5 and 10% air voids. The wet leg lies between 0 and 5% air voids for all soils. The air-void lines are based on a particle density of 2.70 (Mg/m^3).

The curve fitting in Figure 7.5 is based on linear regression with a polynomial of the 4th order. The regression parameters are presented in Table 7.6. Determination of OWC according to Proctor is an empirical method, where water content is plotted versus dry density. Normally, the relation between water content and dry density is not mathematically described. The efforts that have been made to do this have mainly ended up with second, third- or fourth-order polynomial equations (Hilf, 1990; Howell *et al.,* 1997; Li and Sego, 2000). The use of linear regression with a fourth-order polynomial fits the tested soils in this study best. The general 4th order model can be expressed as;

$$Y = \beta_0 + \beta_1 \cdot x + \beta_2 \cdot x^2 + \beta_3 \cdot x^3 + \beta_4 \cdot x^4 + \varepsilon \qquad \textbf{(EQ : 7.1)}$$

where:

- $Y = \rho_d$
- $x = w$

This model has been criticised for several reasons (Howell *et al.,* 1997; Li and Sego, 2000). The main criticism is that the regression parameters (β_i) change by up to three-orders of magnitude or even from positive to negative values (Howell *et al.*, 1997). Another problem is that the regression model does not contain any parameter for compaction energy, i.e. the regression model will be different for Proctor compaction compared with modified Proctor compaction. These problems entail that it is not possible to establish a general model based on linear regression.

Li and Sego (2000) presented an alternative general model to describe the Proctor curve. However, linear regression and solving the roots of the equation to determine OWC together with engineering judgement have proved to be sufficient in this study to determine OWC. Contrary to Li and Sego (2000), no efforts have been made to mathematically describe the whole range in water content.

The main reason not to describe the whole compaction curve is that natural clay till does not became completely dry except in special circumstances. The only part of a fine-grained till that can naturally become completely dry is that at the very surface, dried by wind and sunshine. At a borrow site the volume of this dry soil is very small compared to the whole excavated volume, and therefore the method developed in this study should be sufficient for Swedish conditions.

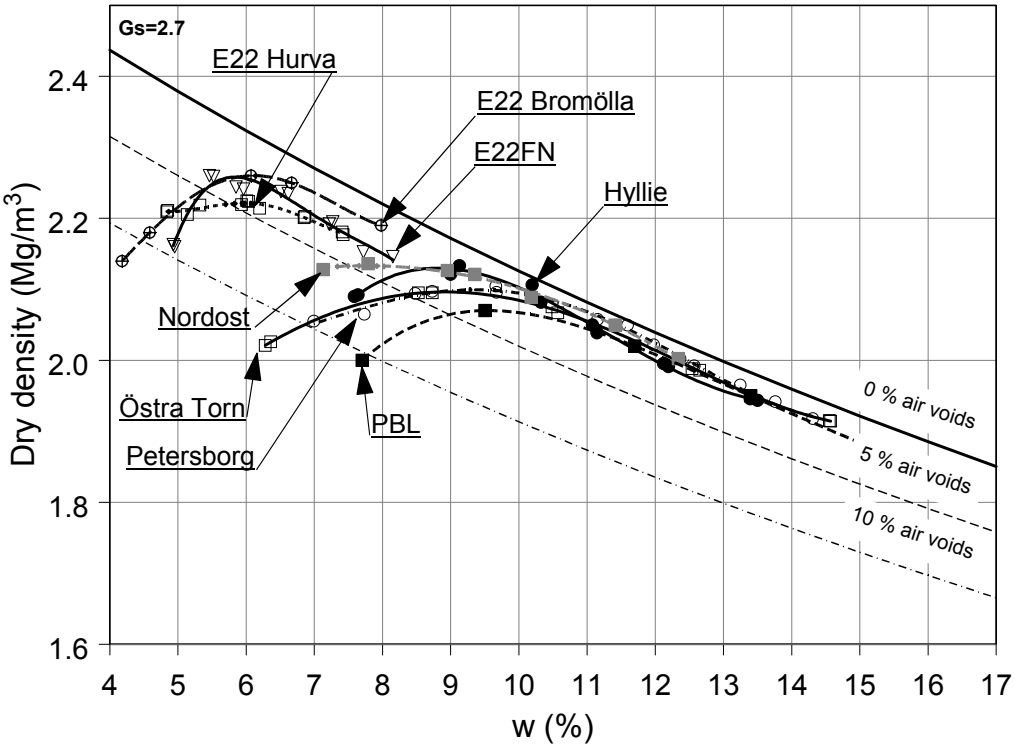


FIGURE 7.5 *Dry density as a function of water content according to modified Proctor for the tested soils.*

The modified Proctor curves in Figure 7.5 are presented in Table 7.6, cf. Equation 7.1.

Table 7.6: *Parameters for correlation between dry density and water content*

Soil	β_0	β_1	β_2	β_3	β_4
Petersborg	3.471	-0.7548	0.1369	0.1009	$25.71 \cdot 10^{-5}$
Nordost	-1.58	1.453	-0.2092	0.0133	$-32.19 \cdot 10^{-5}$
Hyllie	-2.509	1.429	-0.1509	0.006177	$-7.618 \cdot 10^{-5}$
Östra Torn	1.606	0.005164	0.02474	-0.002975	$9.284 \cdot 10^{-5}$
E22FN	-16.58	10.65	-2.234	2.062	-0.007109
E22 Hurva	11.14	6.06	1.519	-0.1664	0.006719
E22 Bromölla	3.716	-1.51	0.4721	-0.05874	0.002542
PBL	-2.622	1.447	-0.1614	0.007748	$-13.89 \cdot 10^{-5}$

In order to obtain the maximum dry density value from the Proctor regression model, the derivative of the regression equations with respect to w are first calculated. The calculated result for E22 Bromölla is presented in Equation 7.2.

$$Y' = -1.51 + 0.9442 \cdot w - 0.17622 \cdot w^2 + 0.010168 \cdot w^3 \qquad \textbf{(EQ : 7.2)}$$

The roots are then computed for the derivative, see Figure 7.6.

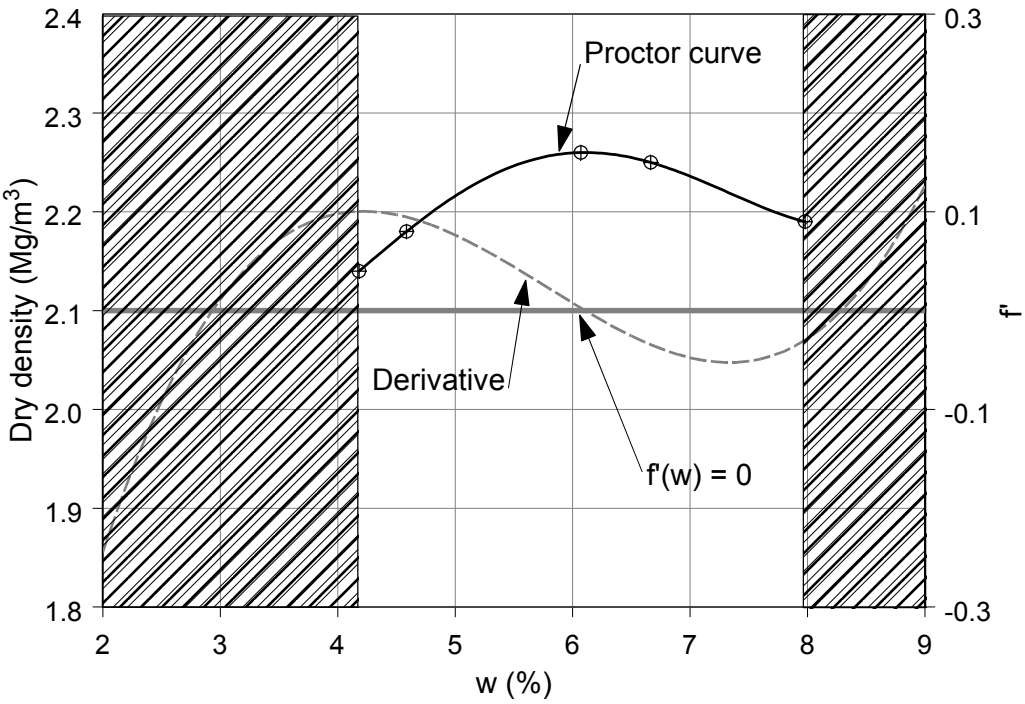


FIGURE 7.6 *Proctor curve and its derivative for the Bromölla material. Only the middle extremum is a possible solution for finding the optimum in the Proctor curve, since the other extrema are outside the Proctor-curve limits.*

The optimum moisture contents for the tested soils are based on the results in Table 7.6. The calculated results are presented in Table 7.7.

Table 7.7: *Natural water content and optimum moisture content. The OWC is based on the regression equations.*

Soil	Natural water content w_N (%)	OWC (%)
Petersborg	14.6	9.3
Nordost	No data	8.00
S7	No data	No data
S14	No data	No data
Lorensborg	No data	No data
Hyllie	15.1	8.8
Östra Torn	19.6	8.9
Sturup	14.4	No data
E22FN	8.0	5.8
E22 Hurva	13.4[a]	6.0
E22 Bromölla	No data	6.1
Örebro	No data	No data
PBL	"16.1"	9.6

a. After the transporting of the soil, excess water was freed from the soil and was found on top of the soil in the containers. When the excess water was removed w_N was 11.9 %.

The results presented in Table 7.7 correspond well to earlier data on the same soil types (Malmborg, 1983; Malmborg, 1996).

Blotz *et al.* (1998) developed a model to estimate the optimum moisture content and maximum dry density for compacted clays based on Proctor compaction. Their method is currently limited to clayey soils with $17\% \leq w_L \leq 70\%$ but it allows for different compaction energies. To estimate the optimum moisture content, liquid limit (w_L in %) and the compaction energy (E in KJ/m^3) must be known, see Equation 7.3.

$$w_{opt,e} = (12.39 - 12.21 \cdot \log w_L) \cdot \log E + 0.67 \cdot w_L + 9.21 \qquad \textbf{(EQ : 7.3)}$$

The applied compaction energy in this study was 2,482 kJ/m^3. The calculated optimum moisture content according to Equation 7.3 are compared to measured values in Table 7.8 for some of the tested soils.

Table 7.8: *Measured optimum moisture content and calculated optimum moisture content according to Blotz et al. (1998).*

Soil	w_L (%)	$w_{opt, calc.}$ (%)	$w_{opt, meas.}$ (%)	Delta w_{opt} (%)
Petersborg	23.9	10.10	9.33	0.77
Hyllie	25.5	10.05	8.8	1.25
E22FN	17	11.66	5.78	5.88
PBL	33	10.43	9.56	0.87

The precision in predicting OWC according to the model presented by Blotz *et al.* (1998) is also shown in Table 7.8. Some of the predictions are very close to the actual values, whereas some results differ considerably. This indicates that further studies with Swedish soils are needed.

Another attempt to predict the compaction characteristics of fine-grained soils from the plastic limit was made by Gurtug and Sridharan (2002). However, their study only included standard Proctor tests and could therefore not be used for comparison in this case.

7.1.4.2 MCV results

MCV calibration lines (cf. Figure 6.5) for some of the soils are presented in Figure 7.7. The same two different groups as found in the Proctor compaction can be identified. The calibration lines for the E22 Hurva, E22FN and Lorensborg materials show a lower slope compared to the other soils, cf. Figure 6.5. One important observation from Figure 7.7 is that the E22FN and Sturup materials have widely different calibration lines, even though their grain size distributions are very similar.

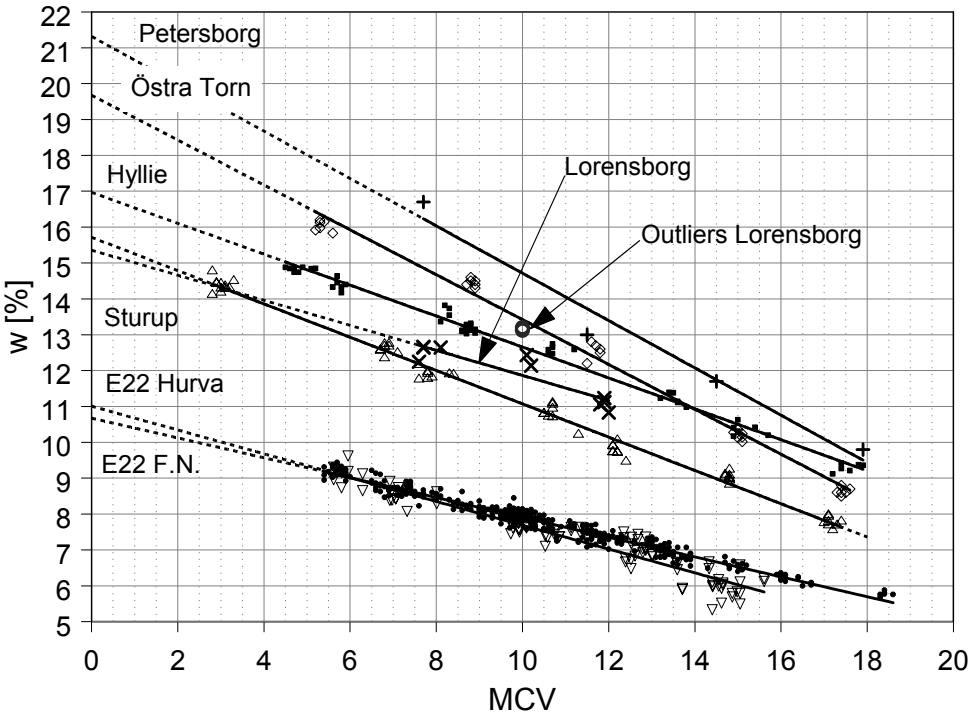


FIGURE 7.7 *MCV calibration lines for seven different soils. The Petersborg line is only based on four tests. The Lorensborg data showed unusually large scatter and some of these results were therfore excluded.*

Matheson and Winter (1997) developed a classification method to identify the most problematic soils owing water susceptibility. The slopes of the MCV calibration line is plotted against the intercept to identify the most problematic soils, see Figure 7.8. From Figure 7.7 the slopes and intercepts of the MCV calibration lines were determined and plotted in Figure 7.8. The results show that the coarser soils have a sensitivities between very high and high. The soils with higher clay contents have between high and moderate sensitivities.

This implies that for fine-grained soils with a larger amount of coarse material are more sensitive to wetting than fine-grained soils with higher fine contents. However, if a soil with very high sensitivity , cf. Figure 3.8, becomes too wet, it is also much easier to dry than a soil with moderate sensitivity.

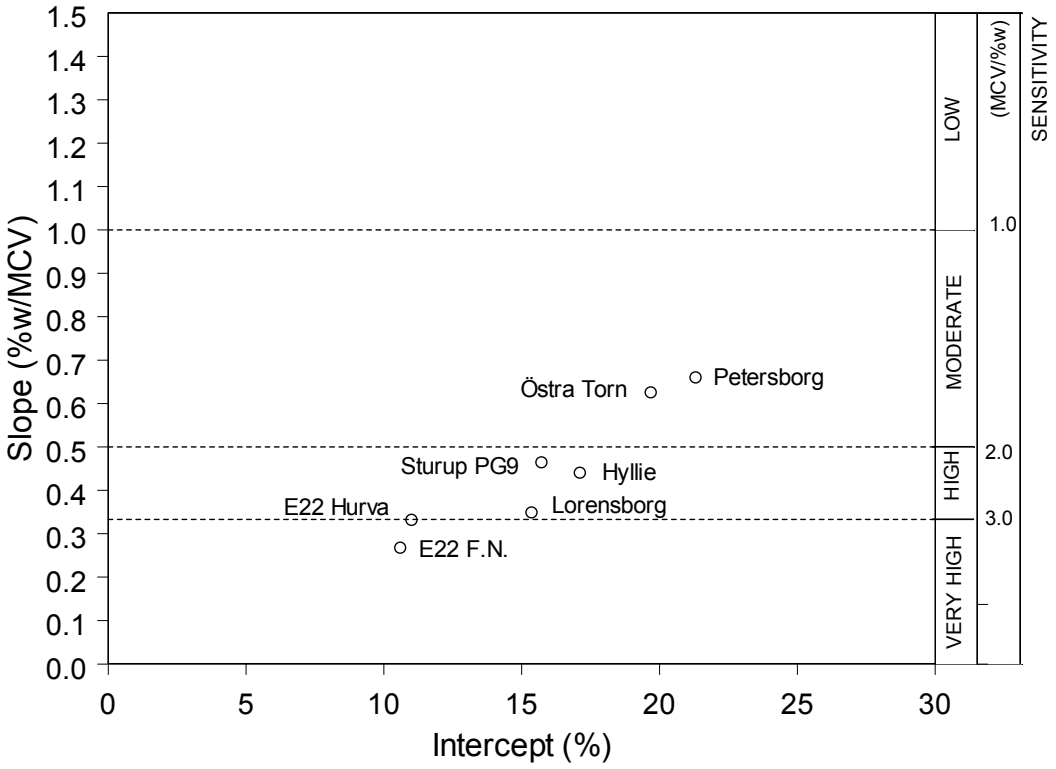


FIGURE 7.8 *Soil sensitivity according to Matheson and Winter (1997). Soils tested within this project are shown in the model.*

The MCV can also be related to the dry density of the soil after compaction in the MCA. Such a correlation between MCV and dry density is shown in Figure 7.9. The correlation is very strong as long as the soil is compacted on the wet side of optimum. Specimens compacted on the dry side of optimim do not connect to the linear equation. The

regression line for the E22FN material is based on more than 300 observations and has a coefficient of determination of 0.97 (R^2). For the Sturup material R^2 is 0.98.

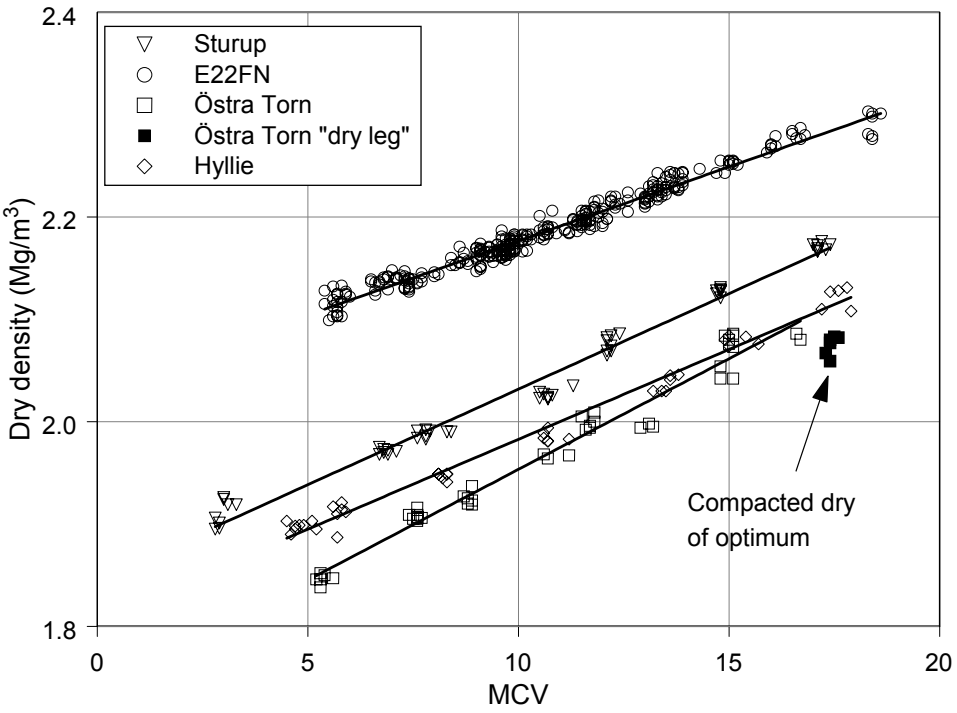


FIGURE 7.9 *Dry density as a function of MCV. The Östra Torn specimens compacted dry of optimum are presented in the plot but excluded from the regression analysis.*

7.1.4.3 Comparison between Proctor and MCV results

Proctor compaction is used worldwide and therefore regarded as a reference test in this study. Four different soils were used for comparison with MCA compaction; two medium-grained and two fine-grained soils. The results are presented in Figure 7.10. For the two coarser soils, the MCA compaction gives the highest dry density compared to Proctor compaction at the same water content. The most probable explanation for this is, the higher compaction energy for the MCA compacted specimens. The drier the soil is the more blows (i.e. higher compaction energy) are applied until optimum compaction is reached.

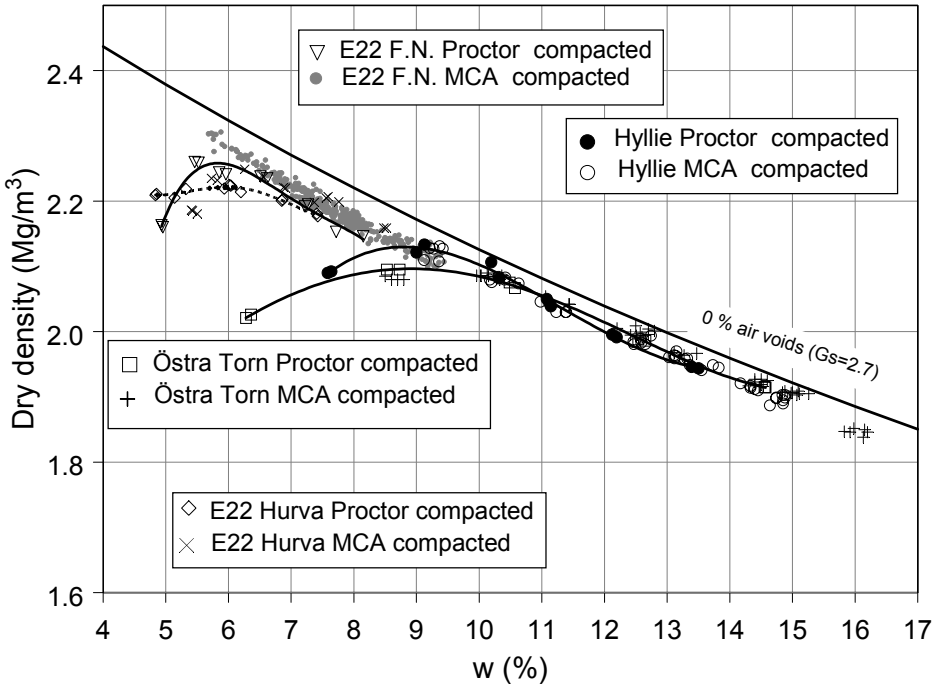


FIGURE 7.10 *Dry density as a function of water content for modified Proctor and MCV compacted specimens.*

However, these results do not fully correspond to previous experience. Murray *et al.* (1992), cf. Figure 7.11, suggested that the optimum compaction achieved during MCV compaction lies between standard Proctor and modified Proctor. In this study , the difference in dry density between modified Proctor and MCV is less pronounced and the dry density of MCV compacted specimens is as high as or higher than of those compacted according to modified Proctor.

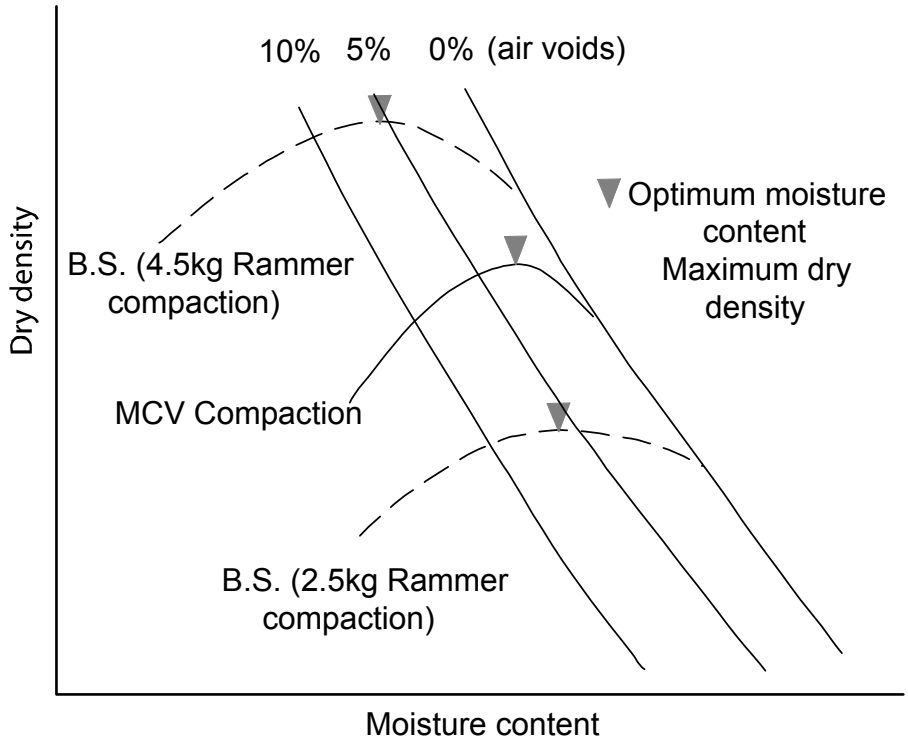


FIGURE 7.11 *Typical compaction curves according to Murray et al. (1992) (after Murray et al., 1992).*

For the fine-grained soils, the dry density is very similar for both compaction methods. However, for dry of optimum fine-grained soils Proctor compaction seems to crush the dry lumps of soil better and thereby result in higher dry density than MCA compaction. Due to the design of the MCA, the compaction energy is applied to the whole area of the specimen, and dry lumps could better resist crushing. This observation is only based on the E22FN material and more soil types have to be tested with the MCA. The water content should then be dry of optimum.

Another difference is the pore-pressure response from compaction. For coarse soils there are no problems with excess pore pressure in either of the compaction methods, owing to the high air and water permeability of the soil. However, in a fine-grained soil wet of optimum, compaction in a MCA mould will gradually be taken up by the pore pressure until no further densification of the soils can be achieved and the applied compaction energy from the rammer only results in a bounce from the pore water. This means that the load is applied to the noncompressible water rather than to the skeleton of solid particles resulting in the rammer bouncing off the specimen without deforming it.

In the Proctor compaction the rammer only hits a small area at a time resulting in local high pore pressure and shear failure in the soil and thereby soil movement and kneeding of the material in the mould.

For the E22FN material, the MCA compaction results in higher dry density for the whole tested range of water content. The E22 Hurva material obtains a higher dry density at MCA compaction up to approximately OWC and then a lower dry density compared to Proctor compaction for water contents dry of optimum. No obvious reason for this was found but a possible explanation could be the high calcium content for the E22FN material and the agglomeration of the soil that could be beneficial for the MCA compaction.

The theoretical compaction energy for modified Proctor is 2482 kJ/ m^3. However, in a Proctor test a collar is used above the mould to allow some excess material. This excess in height is then trimmed off to give the specimen the same height as the Proctor mould. Depending on the operator's skill, this soil excess could vary and thereby also the applied compaction energy in relation to soil volume.

For modified Proctor the compaction effort is independent of water content. On the other hand, for the MCA compacted specimens each blow represents 17.2 J but the number of blows decreases with increasing water content. Since different specimens become different heights at the MCV compaction, energy per volume soil differs and can only be calculated as a mean value for each different soil.

The Östra Torn material was chosen to visualise the number of blows in an ordinary Proctor plot, see Figure 7.12. The MCV method is only defined to an MCV value of 18. The MCV calibration was only performed to an MCV of 17.6 which corresponds to 256 blows for the Östra Torn material. To allow a full comparison between test results from Proctor and MCV compaction another set with drier soil with a MCV > 18 should have been made, but unfortunately this was not the case.

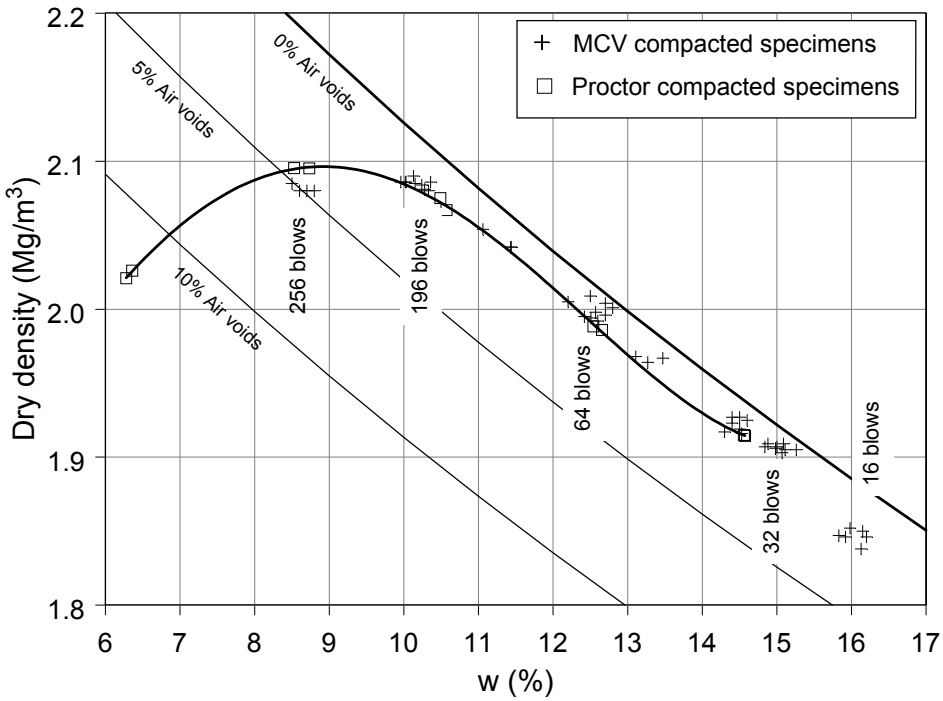


FIGURE 7.12 *Applied number of blows for MCV compaction at different water contents for the Östra Torn material. Modified Proctor compacted specimens are included for comparison.*

The result in Figure 7.12 shows no difference in dry density between the modified Proctor and the MCV result.

It should be noted that the soil E22FN behaves differently from the Östra Torn soil. The E22FN soil with the highest MCV, i.e. compacted with 256 blows, shows a significantly different behaviour compared to the Proctor-compacted specimens, cf. Figure 7.10. This indicates that the

relation could be soil-dependent, which in turn could explain the discrepancy with the results from Murray *et al.* (1992).

Another comparison between MCV and modified Proctor compaction is presented in Figure 7.13. The linear regression in Figure 7.13 for the Östra Torn material implies that 94.8 blows in the MCV method correspond to the energy per unit volume applied in a modified Proctor test, cf. Equation 7.4 and Equation 7.5.

$$Energy_{OstraTorn} = -36.48 + 26.56 \cdot (Blows) \quad \textbf{(EQ : 7.4)}$$

$$Energy_{E22FN} = -49.94 + 27.95 \cdot (Blows) \quad \textbf{(EQ : 7.5)}$$

where the energy is in kJ/m^3.

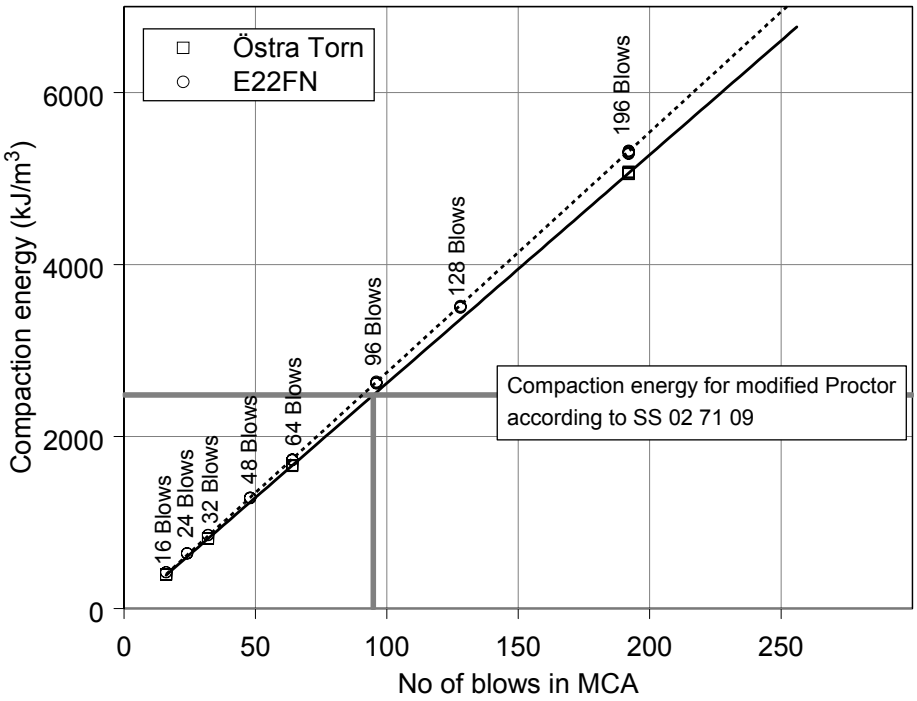


FIGURE 7.13 *Compaction energy as a function of blows in MCA compaction. The result is based on the Östra Torn and E22FN materials. Each number of blows consists of six or more observations.*

In an MCV compaction test series, the nearest number of blows is 96 to approximately reach the same compaction energy as modified Proctor. Using the data from Östra Torn and E22FN, the number of blows can be plotted against MCV, se Figure 7.14. The regression equation is presented in Equation 7.6 and Equation 7.7.

$$MCV_{OstraTorn} = -6.4 + 4.24 \cdot \ln(Blows) \qquad \textbf{(EQ : 7.6)}$$

$$MCV_{E22FN} = -5.5 + 4.00 \cdot \ln(Blows) \quad \textbf{(EQ : 7.7)}$$

The R^2 values for the equations are 0.986 and 0.983 respectively. The difference between the soils lies in the resulting volume of the specimens after compaction, i.e. the height of the specimens. Data from more soils is needed to make a general equation.

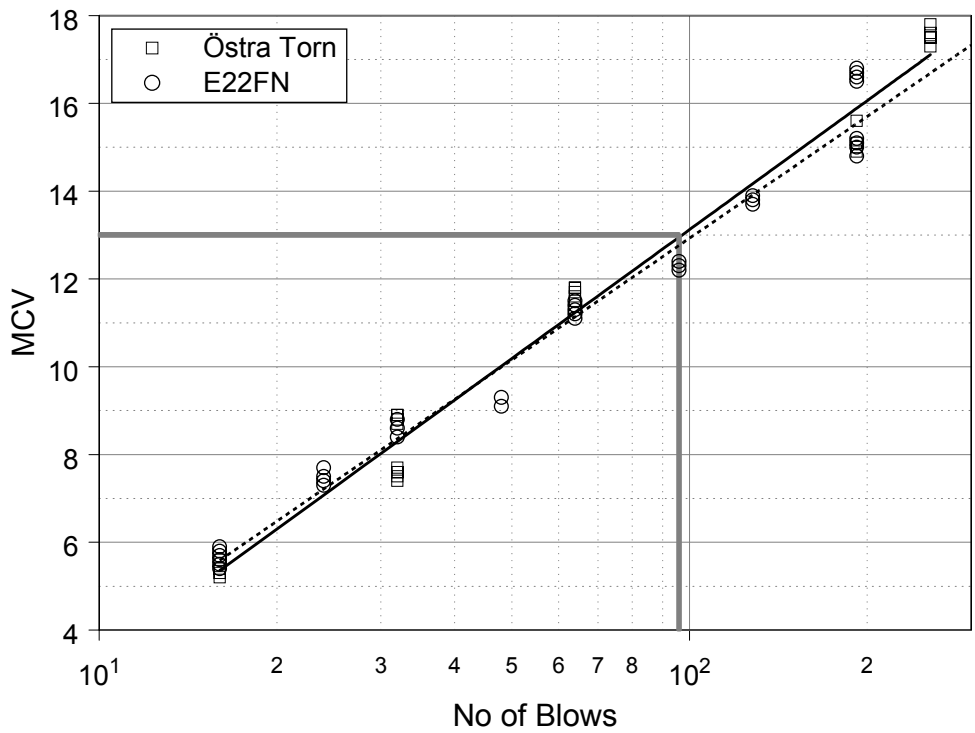


FIGURE 7.14 *MCV plotted against number of blows for the Östra Torn and E22FN materials.*

For both the Östra Torn material and the E22FN materials the compaction energy from modified Proctor corresponds approximately to an MCV of 13, cf. Figure 7.14.

However, from Table 7.7 the OWC for the Östra Torn material is 8.92%. A water content of 8.92% inserted in Figure 7.12 shows that the corresponding number of blows for MCV compaction is approximately 256. This will in turn give an MCV of approximately 17, cf Figure 7.14, which corresponds to more than twice the compaction energy of modified Proctor. This exercise shows that the relation between the methods is much more complicated than comparing compaction energies. A further complicating factor is the difference in energy losses between the two methods. The results above indicate larger energy losses for MCA compaction.

In Proctor compaction, each blow results in higher density until maximum density for the actual water content is reached. Each time the ram pressure is removed, the expansion of the soil is resisted by negative pore pressures in the soil (Olson, 1963). In the MCV compaction each blow hits the whole specimen surface and the expansion of the soil after the blow is prevented by a combination of both negative pore pressures and friction between the soil and the MCV mould.

However, the results from the different compaction methods show approximately the same achieved dry density on the wet side of optimum. The main difference between the methods is found on the dry side of optimum.

7.1.5 Shear strength based on unconfined compression test

The shear-strength tests in this study have been performed mainly on MCA compacted specimens. Since an MCA-compacted specimen varies in height between approximately 75 and 90 mm they were tested in pairs stacked on top of each other. This increases the slenderness ratio from approximately 0.75:1 to 1.5:1. For comparisons Proctor compacted specimens have also been tested in unconfined compression tests.

Parsons and Boden (1979) studied the correlation between remoulded undrained shear strength and MCV. They tested a wide range of soil types, which they divided into six different groups. The correlation equations are presented in Table 7.9.

Table 7.9: *Correlations between remoulded shear strength and MCV (Parsons and Boden, 1979).*

Soil type	Equation of relation	No obs	r
Clay - high plasticity	Log c_u = 0.74 + 0.111(MCV)	40	+0.94
Clay - intermediate plasticity	Log c_u = 0.77 + 0.107(MCV)	44	+0.96
Clay - low plasticity	Log c_u = 0.91 + 0.112(MCV)	14	+0.89
Silt - high plasticity	Log c_u = 0.70 + 0.105(MCV)	15	+0.97
Silt - intermediate plasticity	Log c_u = 0.80 + 0.100(MCV)	2	-
Silt - low plasticity	Log c_u = 0.91 + 0.120(MCV)	4	-

For a given MCV, silt of low plasticity gives the highest and silt of high plasticity the lowest shear strength in relation to the MCV, cf.Figure 7.15. The correlation equation for silt of low plasticity can be rewritten as:

$$c_u = 8.13 \cdot e^{0.276 \cdot MCV} \quad \textbf{(EQ : 7.8)}$$

and that for silt of high plasticity as:

$$c_u = 5.01 \cdot e^{0.248 \cdot MCV} \quad \textbf{(EQ : 7.9)}$$

However, the data in Table 7.9 are based on shear strength determined by means of laboratory vane, while the shear strength in this study was determined by unconfined compression tests.

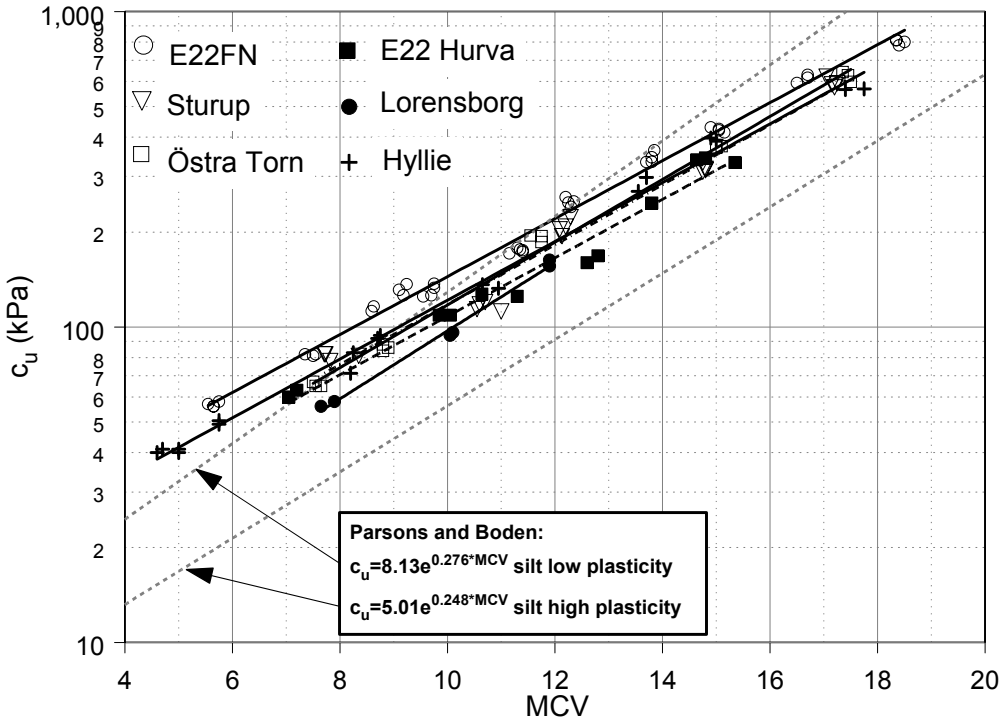


FIGURE 7.15 *Shear strength as a function of MCV for six different soils. Two correlations from Parsons and Boden (1979) is also included in the figure.*

Based on Figure 7.15, the relation between the undrained shear strength, cu and MCV can be expressed in the following general equation;

$$c_u = \alpha \cdot e^{\beta \cdot MCV} \qquad \textbf{(EQ : 7.10)}$$

where α and β are material parameters.The undrained shear strength c_u is measured in kPa. In this study β turned out to be a more or less soil independent constant.

The correlation results from the data in Figure 7.15 are presented in Table 7.10.

Table 7.10: *Parameters for correlation between MCV and shear strength for double height specimens, cf. Equation 7.10.*

Soil	α	β	No of observations	R^2
E22FN	17.83	0.21	41	0.994
E22Hurva	12.78	0.21	12	0.972
Sturup	13.2	0.22	20	0.976
Östra Torn	11.86	0.23	15	0.995
Hyllie	14.21	0.22	12	0.990
Lorensborg	8.13	0.25	6	0.994
All data	**14.1**	**0.22**	**106**	**0.746**
Petersborg[a]	*18.69*	*0.20*	*6*	*0.761*

a. Based on recalculated MCV data cf. Equation 6.2. Not presented in Figure 7.15.

The β parameter has a value between 0.21 and 0.23 for all soils in Table 7.10 except for the Lorensborg soil. A major difference between the Lorensborg soil compared to the others is the depth of the trial pit. At Lorensborg the soil was only excavated above the frost limit. Another difference is the regression limits. The determined MCV varies only from 8 to 12, and this could affect the slope β as well as the α parameter for a larger MCV interval. However, the Lorensborg data corresponds well with the results presented by Parsons and Boden (1979), cf. Equation 7.9.

If all results in this study are merged, see Figure 7.16, the regression line determines over 74% of the variation in the data. Some of the low

MCV data were excluded from the regression analysis due to the different failure patterns, cf. Figure 6.12 and Figure 7.17 (outliers Hyllie).

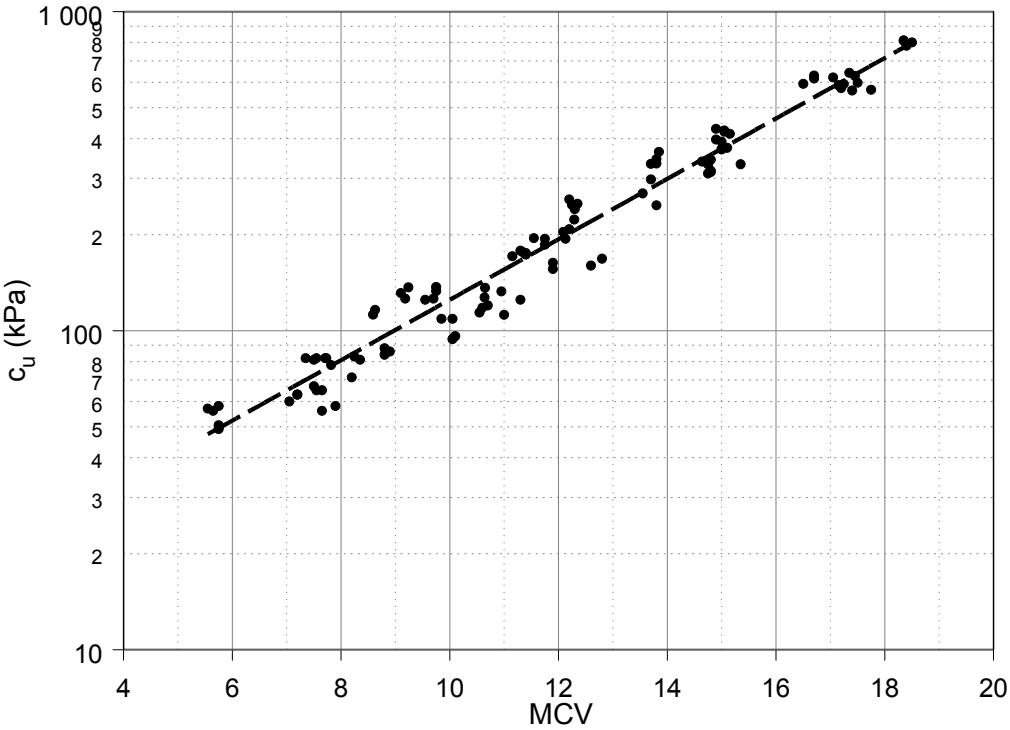


FIGURE 7.16 *Shear strength (c_u) as a function of MCV for all tested soils. The result is based on 106 tests, cf. Table 7.10.*

Results from previous studies, by Malmborg (1992), are shown in Table 7.11. They show the same β-value as this study. However, the α-values differ by a factor of 2 or more. A probable explanation for this difference is the effect of different sample sizes (cf. Chap. 6). Malmborg (1992) used 1800g of soil in the MCV test. This supports the assumption

that the β-value is independent of soil type and could be regarded as a constant.

Table 7.11: *Parameters for correlation between MCV and shear strength presented byMalmborg (1992). Cf.Equation 7.10.*

Soil	α	β
Ramp 3	40.5	0.21
Ramp 4	34.6	0.21
Ramp 3+4	40.3	0.2

7.1.5.1 The effect of sample height on shear strength

Since the tested MCV specimens are shorter than the normal 2:1 ratio, the effect of sample height on measured shear strength was studied. This study was performed with four different soils to ensure that the result was not random or valid for one specific soil type only. The short specimens consisted of single MCV specimens and the slenderness ratio was then approximately 0.85:1. This value is low even compared to the Danish standard (Jacobsen, 1970), which uses a slenderness ratio of 1:1. The null hypothesis of the test was that there should be no difference in shear strength between single- and double-height specimens. The null hypothesis was rejected i.e. a significant difference in shear strength was found.

The same load frame and instrumentation was used for single- and double-height specimens. The deformation rate was 2 mm/min for single height specimens and for double height specimens it was 4 mm/min.

The specimens were prepared at several different water contents, which gave different MCVs.

The results are presented in Figure 7.17. The relations between MCV and shear strength follow the same pattern as for higher specimens, but are somewhat displaced with different α-parameters. The differences in shear strength are mainly related to the boundary conditions i.e. the friction at the ends of the specimens has a significant influence on the measured shear strength. The regression results for the single-height shear-strength tests are presented in Table 7.12.

Table 7.12: *Parameters for correlation parameters between MCV and shear strength measured in single-height specimens. Δc_u is the relative increase in shear strength measured in single-height specimens compared to that in double height specimens. Cf. Table 7.10*

Soil	α	β	No of obs.	R^2	Δc_u (%)[a]
E22FN	22.98	0.21	26	0.987	+29
E22 Hurva	20.59	0.20	8	0.994	+46
Hyllie	16.68	0.22	8	0.986	+17
Östra Torn	13.25	0.21	15	0.993	-9

a. Calculated at MCV = 10.

The relative shear-strength increase, see Equation 7.11 ,is here denoted as Δc_u.

$$\Delta c_u = \frac{c_{u(single)} - c_{u(double)}}{c_{u(double)}} \qquad \textbf{(EQ : 7.11)}$$

The influence of specimen height shows a wide scatter between the soils. The fine-grained soils appear to be less affected by changes in the slenderness ratio, and for the Östra Torn material the measured strength even decreased with lower ratio. For the Hyllie material the strength increase is below 20%. The coarser soils are more affected by the changes in the slenderness ratio with strength increases upto almost 50%. A possible explanation for this is that the coarser soils have a higher angle of friction. They are thereby more affected by the friction between the soil sample and the top and bottom plates since this will affect larger parts of the failure planes. Another factor could be that the lubrication of the end plates is more effective for the fine-grained soils.

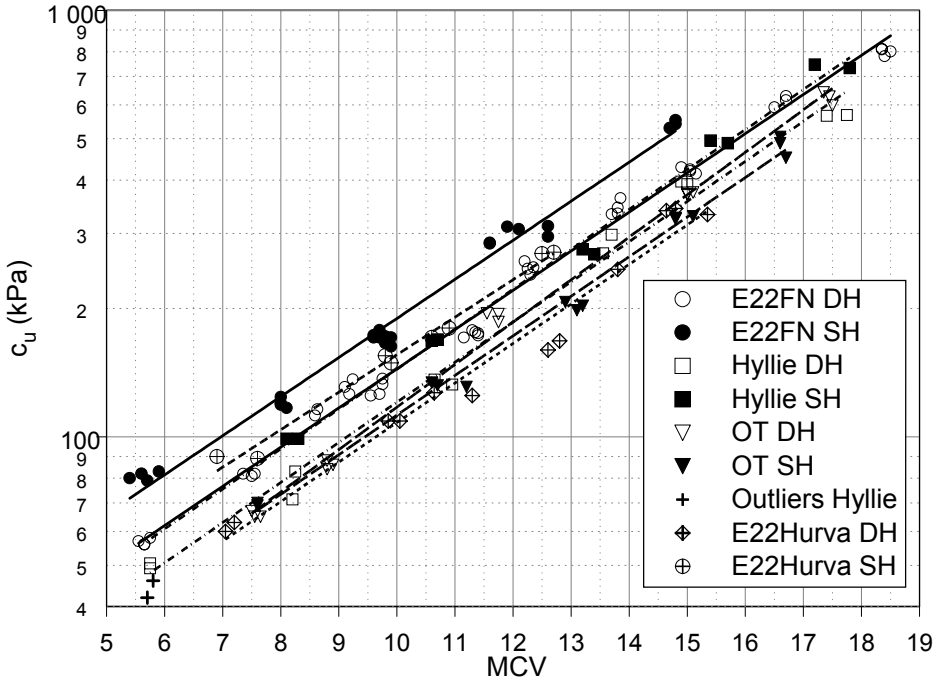


FIGURE 7.17 *Shear strength as a function of MCV for single-height (SH) and double-height (DH) specimens. Low MCV values for the Hyllie material were excluded from the linear regression due to barrel effects, cf. Figure 6.12.*

For clarity, the results for the E22FN material are also presented in Figure 7.18.

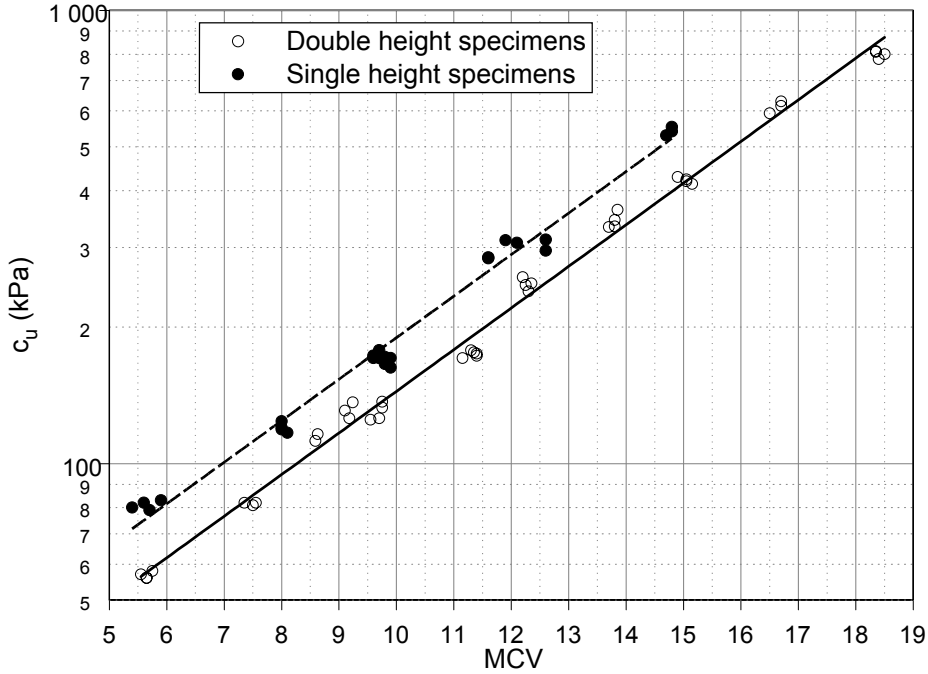


FIGURE 7.18 *Shear strength as a function of MCV for single height and double height specimens for the E22FN material.*

7.1.5.2 Influence of water content on the shear strength

The water content plays a major role in the behaviour of a fine-grained soil. Traditionally, the shear strength has often been presented as a function of plasticity index or liquid limit (Skempton and Northey, 1952; Dennehy, 1979; Forde and Davis, 1979; Arrowsmith, 1979; Black and Lister, 1979). Since the water content is more convenient to determine; and in saturated soils also reflects the void ratio, correlations between this parameter and shear strength have also been performed

(Hartlén, 1974; Dennehy, 1979; Moroto 1993; Jenkins and Kerr, 1998; Kumar and Wood, 1999).

Hartlén (1974) presented undrained shear strength as a function of water content, see Equation 7.12. The results were derived from unconfined compression tests on compacted specimens from clay till. The specimens had a diameter of 50mm.

$$c_u = 2.45 \cdot 10^{-5} \cdot w^{-6.64} \qquad \textbf{(EQ : 7.12)}$$

where c_u is in kPa and w is in decimal number.

Dennehy (1979) presented a compilation of shear strength as a function of moisture content for 22 different soils. The tested specimens were prepared from remoulded and recompacted soil at pre-determined water contents. The result from this study does not fully correspond to the results presented by Dennehy (1979). Dennehy proposed a linear relationship between log shear strength and log moisture content.

Jenkins and Kerr (1998) presented a linear relationship between shear strength and water content. The results presented by Kumar and Wood (1999) showed a linear relationship between log shear strength and water content. Kumar and Wood used fall cone tests to determine the shear strength.

The results from this study show a linear relation ship between log shear strength and water content, see Figure 7.19.

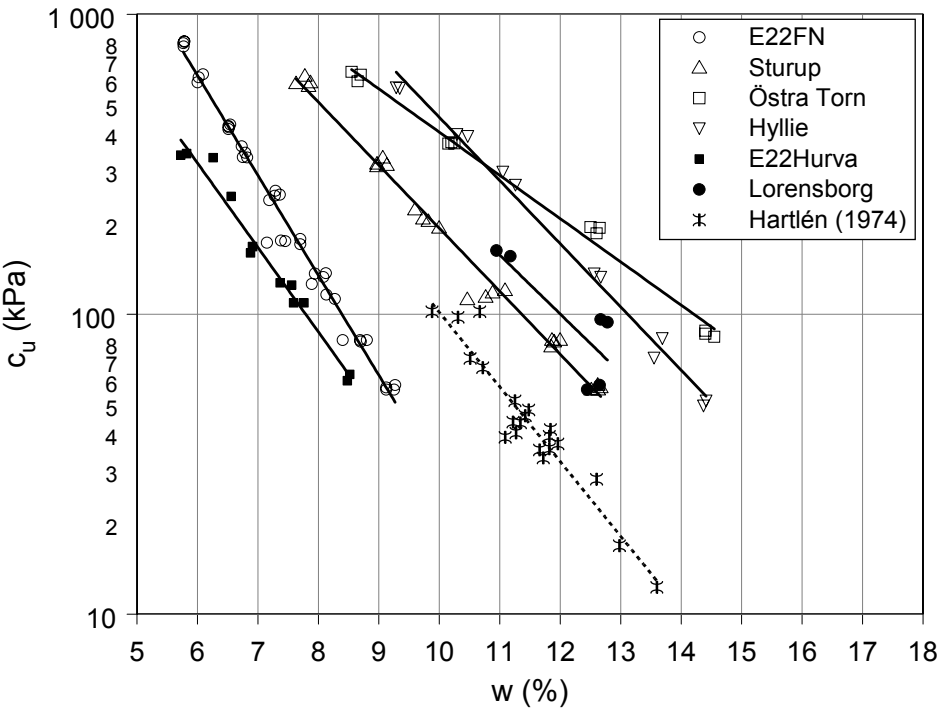


FIGURE 7.19 *Shear strength as a function of water content for MCA compacted specimens. The figure also includes data from Hartlén (1974). Note that Hartlén did not use MCA compaction.*

The general relationship between undrained shear strength and water content used in this study is presented in Equation 7.13.

$$c_u = \kappa \cdot e^{(\lambda \cdot w)} \qquad \textbf{(EQ : 7.13)}$$

where c_u is in kPa.

The results from Hartlén (1974) include shear strength as low as 13 kPa, which is very low compared to the soils used in this study. Another difference between this study and the study presented by Hartlén (1974) is the diameter of the specimens. This study uses a diameter of 100 mm compared to 50 mm for Hartlén (1974). The maximum grain size allowed in an unconfined strength specimen is $1/5^{th}$ of the sample diameter. This difference in particle- and specimen size between the studies could explain some of the differences. Another effect could be differences in applied compaction effort and also differences in compaction equipment i.e. homogenity of the specimens. The MCA compacted specimens has limitations regarding wet samples, cf. Figure 6.12. The soil type used by Hartlén (1974) is very simila to those used in this study.

Table 7.13: *Shear strength as a function of water content for compacted specimens. The specimens prepared in this study were compacted with MCA whereas Hartlén (1974) used a different compaction method.*

Soil	κ	λ	No of obs.	R^2
E22FN	62300	-0.767	38	0.984
E22Hurva	15610	-0.648	12	0.974
Sturup	24420	-0.484	24	0.987
Östra Torn	11170	-0.332	12	0.991
Hyllie	57920	-0.485	12	0.990
Lorensborg	22840	-0.452	6	0.656
Hartlén (1974)	*31250*	*-0.573*	*21*	*0.909*

The results from Hartlén fall between the two groups in Figure 7.19.

7.1.5.3 Shear strength as a function of dry density

A dens subsoil is very important to achieve a firm and durable base for a construction. The soil stiffness is often related to the achieved dry density. In Figure 7.20 the dry density is plotted against the shear strength of the soil for MCV-compacted specimens. For the E22FN material data from both the double-height specimens and the single-height specimens are presented in the plot.

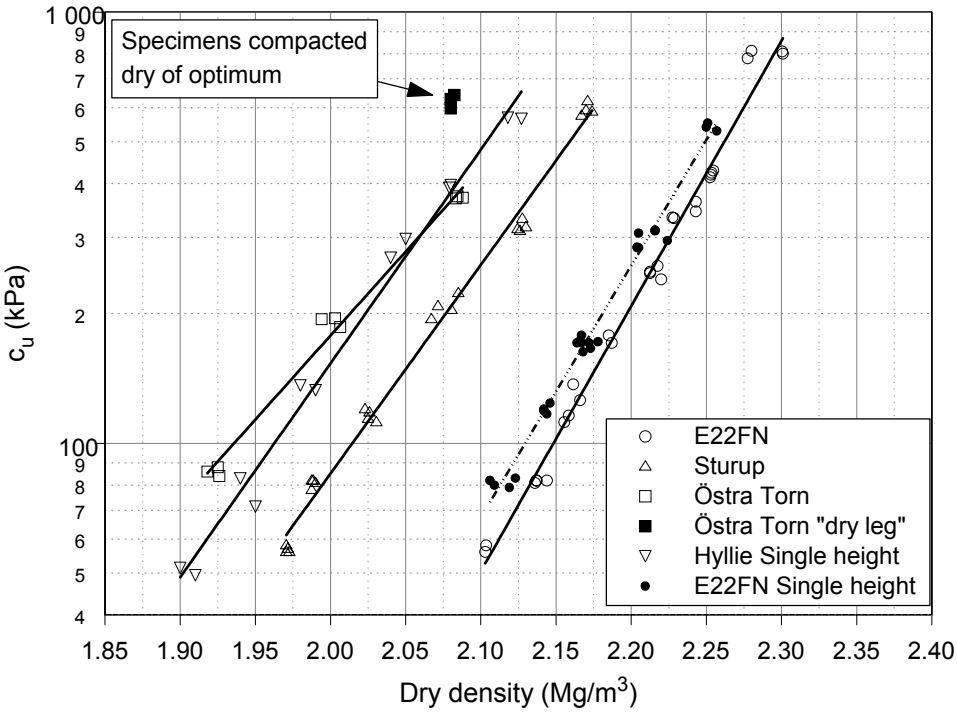


FIGURE 7.20 *Shear strength as a function of dry density for MCA compacted specimens. The square filled symbols show Östra Torn data not used in regression due to compaction on the dry side of optimum, cf. Figure 7.5.*

The general regression equation is presented in, Equation 7.14.

$$c_u = A \cdot e^{B \cdot \rho_d} \qquad \textbf{(EQ : 7.14)}$$

where:

- c_u = undrained shear strength (kPa)
- A and B material constants

A and B is material constants and c_u is in kPa.

The results from the regression analyses are presented in Table 7.14.

Table 7.14: *Undrained shear strength as a function of dry density for MCV compacted specimens. The Hyllie and E22FN SH data are from single height MCV specimens.*

Soil	A	B	No of obs.	R^2
E22FN	$6.18 \cdot 10^{-12}$	14.16	28	0.825
Sturup	$1.79 \cdot 10^{-8}$	11.14	24	0.884
Hyllie	$1.84 \cdot 10^{-8}$	11.42	12	0.936
Östra Torn	$2.90 \cdot 10^{-6}$	8.97	9	0.976
E22FN SH	$3.72 \cdot 10^{-6}$	13.44	26	0.887

The effect of compaction on the dry side of the Proctor curve is illustrated for the Östra Torn material. The undrained shear strength is then about 200 kPa higher than in the same soil compared to the same dry density on the wet side, cf. Figure 7.20.

The same type of analysis was also performed for Proctor-compacted specimens, see Figure 7.21. Due to the difference in compaction procedures between the MCV and Proctor methods, the effect of compaction on the dry side is even more pronounced for the Proctor-compacted specimens.

As can be seen in Figure 7.21, specimens with a dry density of 2.16 Mg/m^3 can have shear strengths of both 100 kPa and 500 kPa, depending on the water content at the Proctor compaction, cf. Figure 7.5.

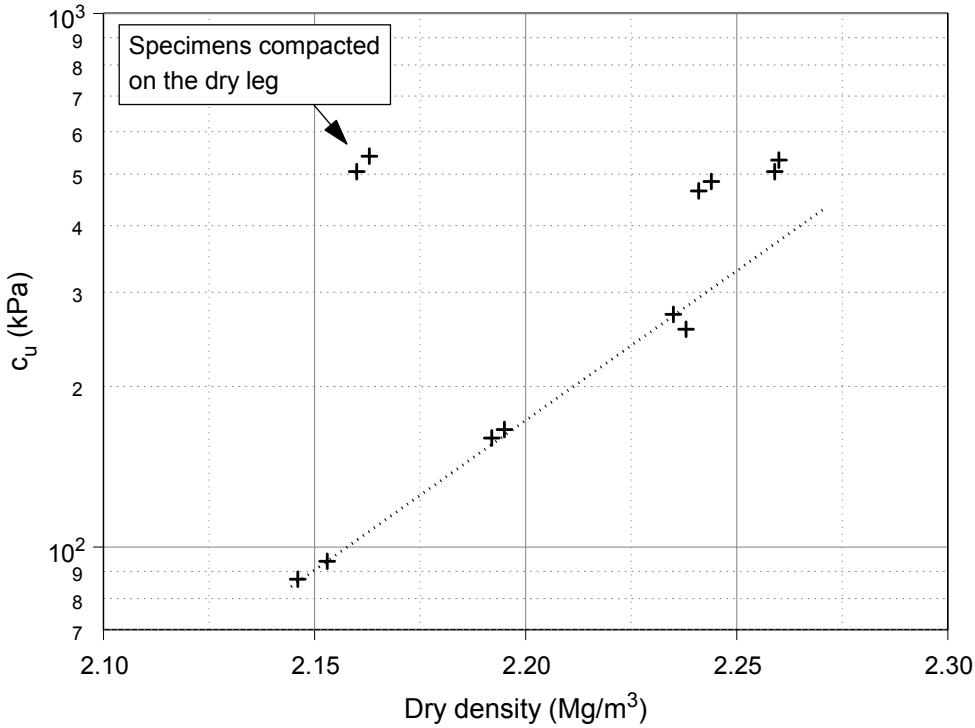


FIGURE 7.21 *Undrained shear strength as a function of dry density for the E22FN material after Proctor compaction. The dotted line is only a subjective guideline.*

The specimens with a shear strength of 100 kPa have approximately 3% air voids whereas the specimens with a shear strength of 500 kPa have approximately 9% air voids, see Figure 7.22. This figure also shows that specimens with the same shear strength can have very different amounts of air voids. The specimens with approximately 9% airvoids are compacted on the dry side of OWC and the structures of their grain skeletons are likely to collapse if they are saturated under stress.

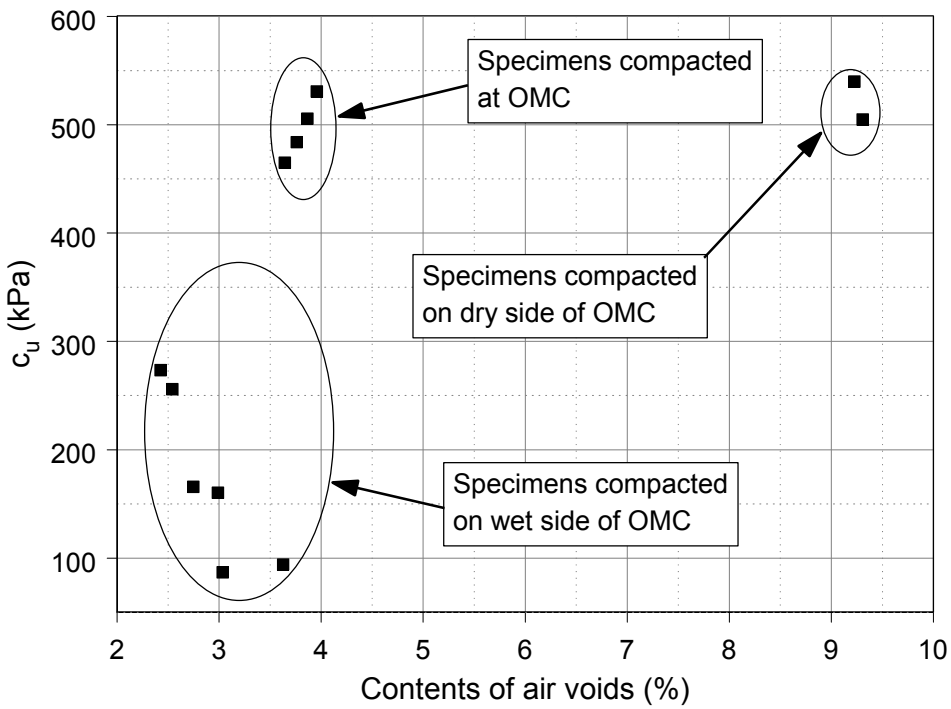


FIGURE 7.22 *Shear strength vesrsus air voids for the E22FN material cf. Figure 7.5.*

The Proctor-compacted specimens contain 2 to 4% air voids when compacted on the wet leg of the Proctor curve. A decrease in water content of 0.5% increases the air void volume from approximately 4% to above 9% with only minor effect on the shear strength.

7.1.5.4 Shear strength as a function of void ratio

The dry density is related directly to the porosity and the void ratio. The strength as a function of void ratio has been studied by Jacobsen (1970) and by Hartlén (1974). Jacobsen (1970) proposed a correlation between the undrained shear strength and void ratio (e_k) in Danish till, see. Equation 7.15.

$$c_u = 10 \cdot \exp(0.77 \cdot e_k^{-1.2}) \qquad \textbf{(EQ : 7.15)}$$

where:

- c_u = undrained shear strength (kPa)
- e_k = void ratio

This correlation is based on plate load tests, unconfined compression tests and vane tests. According to Jacobsen (1970), this equation will give conservative estimates of the shear strength, cf. Figure 7.23.

The correlations for shear strength in Swedish clay till proposed by Hartlén (1974) are based on clay content, degree of saturation and void ratio in the till, see Equation 7.16.

$$c_u = 18.0 \cdot w_0^{-2.02} \cdot e_0^{-1.88} \cdot l_c^{2.66} \qquad \textbf{(EQ : 7.16)}$$

where:

- c_u = undrained shear strength (kPa).
- l_c = clay content.
- w_0 = water content.
- e_0 = void ratio.

The validity of this correlation was limited to clay contents between 17 and 32%, water contents between 7 and 20% and void ratios between 0.29 - 0.82. These values refer to material < 20 mm. Further the validity of the equation was restricted to a maximum undrained shear strength of 200 kPa owing limitations of existing data at higher strengths. The grain density was set to 2.67 Mg/m^3 for all tills as simplification, cf. Figure 7.24.

The void ratio data presented in this study were measured after compaction, i.e. before the strength test. No effort was made to saturate the specimens before testing. The shear strength was evaluated according to Equation 6.3 and the results are presented in Figure 7.23.

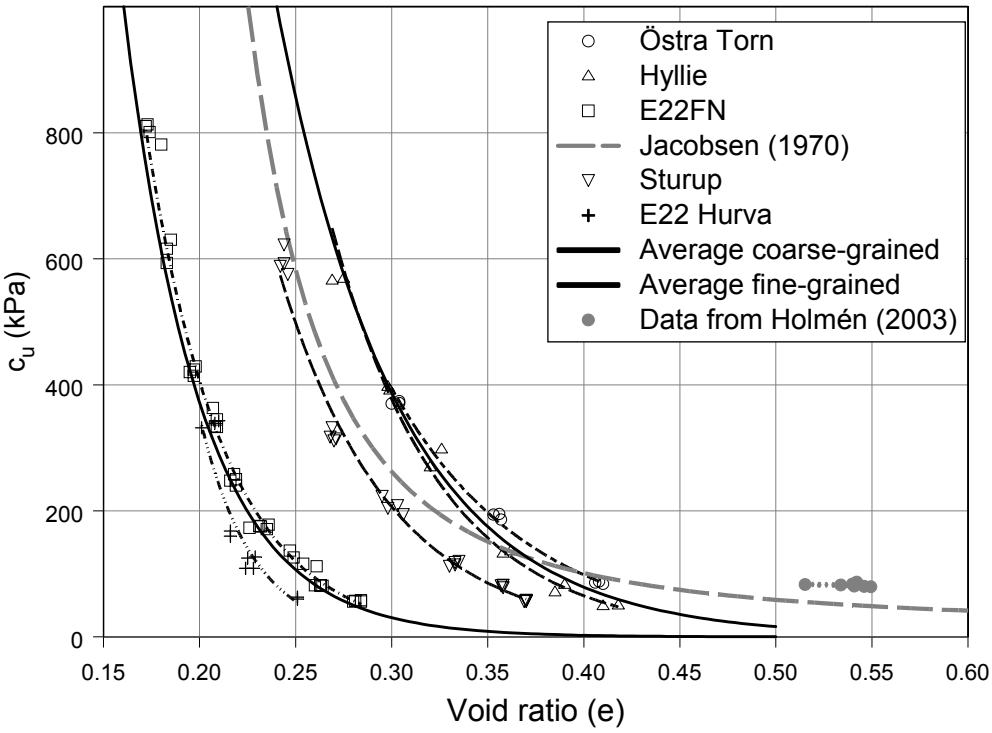


FIGURE 7.23 *Shear strength as a function of void ratio for MCA compacted specimens compared to the Jacobsen (1970) equation. The plot also includes two different average regression lines for fine- and medium-grained tills. The data from Holmén (2003) refer to specimens that have been vibro-compacted.*

The linear regressions for the data presented in Figure 7.23, are assumed to have the general equation.

$$c_u = \alpha_e \cdot e^{(\beta_e \cdot e)} \qquad \textbf{(EQ : 7.17)}$$

where:

- α_e and β_e are material constants
- e = void ratio

The regression equations are presented in Table 7.15.

Table 7.15: *Shear strength as a function of void ratio.*

Soil	α_e	β_e	No of obs.	R^2
Östra Torn	$2.54 \cdot 10^4$	-13.88	9	0.987
Hyllie	$7.24 \cdot 10^4$	-17.53	12	0.961
E22FN	$5.37 \cdot 10^4$	-24.43	36	0.892
Sturup	$4.09 \cdot 10^4$	-17.64	24	0.908
E22 Hurva	$5.04 \cdot 10^5$	-36.39	12	0.754
Average medium-grained	$\mathbf{5.55 \cdot 10^4}$	**-25.03**	47	0.834
Average fine-grained	$\mathbf{4.55 \cdot 10^4}$	**-15.89**	21	0.953

The regression data in Table 7.15 show a wide scatter on the parameters a_e and b_e. The results from E22Hurva have the lowest coefficient of determination and the regression parameters differ from the others than the average. For the more general regression models based on the average data from medium-grained and fine-grained soils the regression models fit quite well, cf. Figure 7.23.

The soil-dependent parameter a_e varies from $5.55 \cdot 10^4$ to $4.55 \cdot 10^4$ for coarse-graned and fine-grained tills respectively. The b_e parameter varies from -25.03 to -15.89 for medium-grained and fine-grained tills respectively.

However, the Sturup material seems does not seem to fit either the medium-grained model or the fine-grained model but belongs somewhere in-between. This difference in behaviour compared to the other soils can also be seen in Figure 7.7, Figure 7.9 and Figure 7.19.

The Jacobsen (1970) equation corresponds quite well to the measured values for the fine-grained tills and particularly for the Sturup material, see Figure 7.23. However, if it is used for more medium-grained tills the undrained shear strength becomes overestimated.

Another way to express the shear strength in relation to void ratio can be made according to Equation 7.18, (referred to as new equation or modified Ryshkewitch equation, cf. Figure 7.24).

$$c_u = K \cdot e^{-(L \cdot e)} \qquad \textbf{(EQ : 7.18)}$$

where:

- c_u = UCS/2
- K =a material dependent constant
- L = a material dependent constant
- e = void ratio

The new equation is based on an equation from Ryshkewitch (1953). Ryshkewith (1953) studied the relationship between strength and porosity for porous sintered alumina. The equation presented by Ryshkewitch (1953) uses capillary porosity insted of void ratio and the K parameter in Equation 7.18 is in the Ryshkewitch equation described as the compressive strength of the solid phase.

The new equation is compared to the Jacobsen (1970) equation and the Hartlén (1974) equation in Figure 7.24.

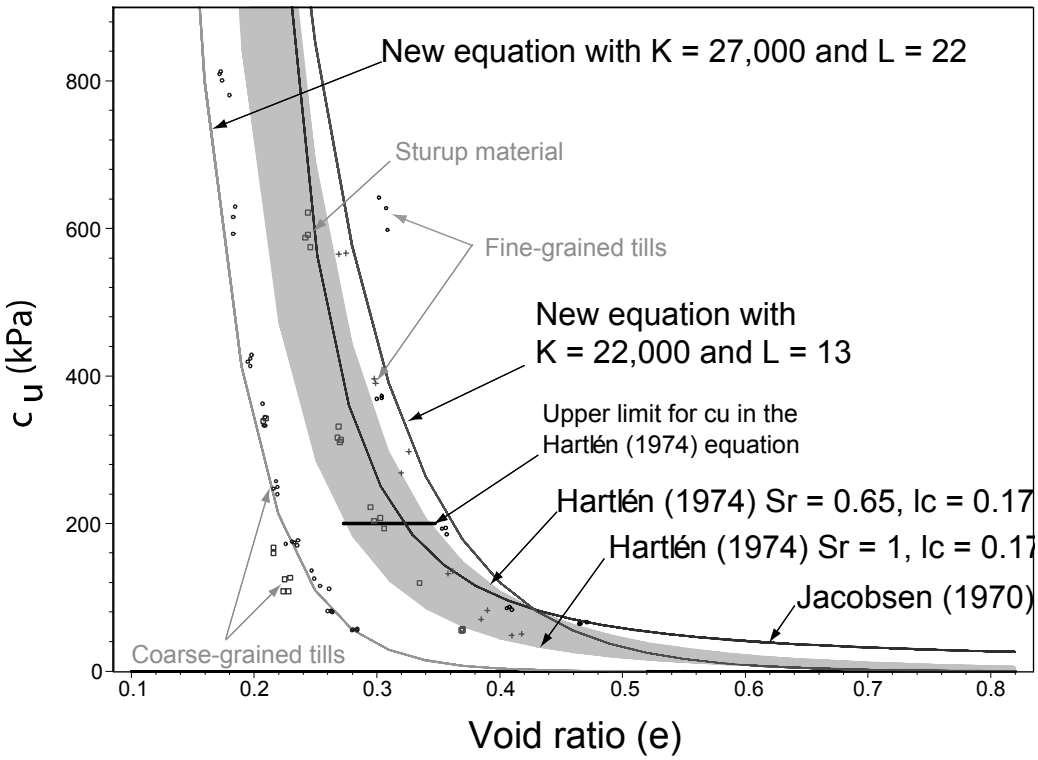


FIGURE 7.24 *Shear strength as a function of void ratio for MCA compacted specimens. The modified Ryshkewitch (1953) Equation 7.18, Jacobsen (1970) Equation 7.15and the Hartlén (1974) Equation 7.16 are also presented. The clay content in Equation 7.16 was set to 17%.*

Figure 7.24 shows the undrained shear strength as a function of void ratio for MCA-compacted specimens. The Hartlén (1974) equation covers a wide range since this is the only equation that covers different degrees of saturation (Sr). From a void-ratio of 0.4 and lower the Jacobsen (1970) equation and the Hartlén (1974) equation for a Sr between 0.70 to 0.75, give approximately the same result. It could be

noted that the Jacobsen (1970) equation and the Hartlén (1974) equation give fairly good approximations of the undrained shear strength for the fine-grained soils and the Sturup material. However, for the more medium-grained soils the new equation with parameters adapted to this kind of soil gives the best estimation of the undrained shear strength.

From the results presented in Figure 7.23 and Figure 7.24 there seems to be a transition zone in undrained shear strength as a function of void ratio for fine-grained tills to medium-grained tills. The Sturup material appers to be in this transition zone.

A possible explanation for the behaviour in this transition zone could be found in a study by Kumar and Wood (1999). They studied the behaviour of clay - gravel mixture with varying clay contents and found that the gravel starts to influence on the soil's behaviour when the gravel fraction reaches about 45% of the volume. Kumar and Wood (1999) also investigated the influence of the shape of the granular particles but no significant difference was dicovered between the E22FN and the Sturup material in this study.

7.1.5.5 The influence of compaction method on shear strength

The different compaction methods used in this study generate different dimensions of the specimens regarding heights and diameters. For the remoulded unstabilised fine-grained soils in this study, modified Proctor and MCV compaction has mainly been used. These compaction methods give, for some soils, different dry densities at the same water content and the effect of this on the shear strength has also been studied.

In Figure 7.25, the modified Proctor-compacted specimens show a lower undrained shear strength compared to MCA-compacted specimens at the same dry density for the E22FN material.

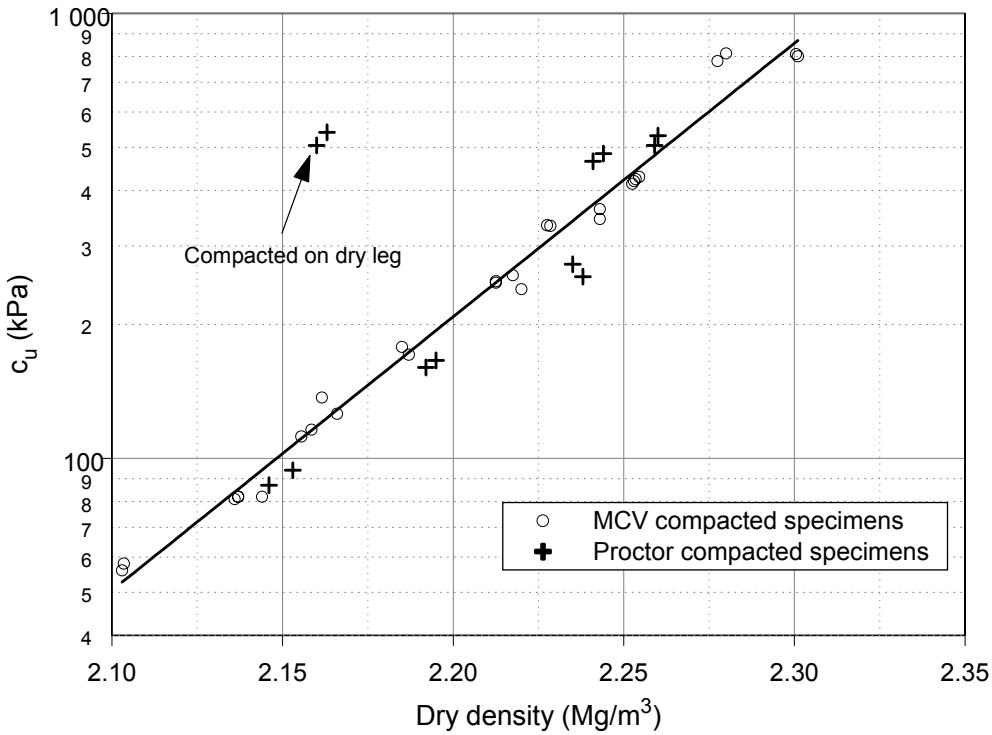


FIGURE 7.25 *Shear strength as a function of dry density for Proctor and MCA-compacted specimens (E22FN material). Note the results from the specimens compacted on the dry leg of the Proctor curve.*

This result was not expected since the modified Proctor-compacted specimens have a lower slenderness ratio, cf. the result presented in Table 7.12 and Figure 7.17 for the E22FN material. According to those results, the Proctor compacted specimens could be expected to have a slightly higher strength. A possible explanation could be that the different compaction methods yield different soil structures but so far there is no

evidence for this. However, the Proctor compacted specimens compacted close to OWC show a higher undrained shear strength compared to the MCA compacted specimens.

7.1.6 Shear strength based on triaxial tests

The triaxial tests were performed to examine the influence of the confining pressure on the shear strength at different initial MCVs. Efforts have been made previously to compare shear strength from MCV specimens and shear strength determined by field-vane tests (Brorsson and Petersson, 1997), see Figure 7.26. The authors attributed the difficulty in comparing these two types of shear strength values to the time lap between the different testing occasions.

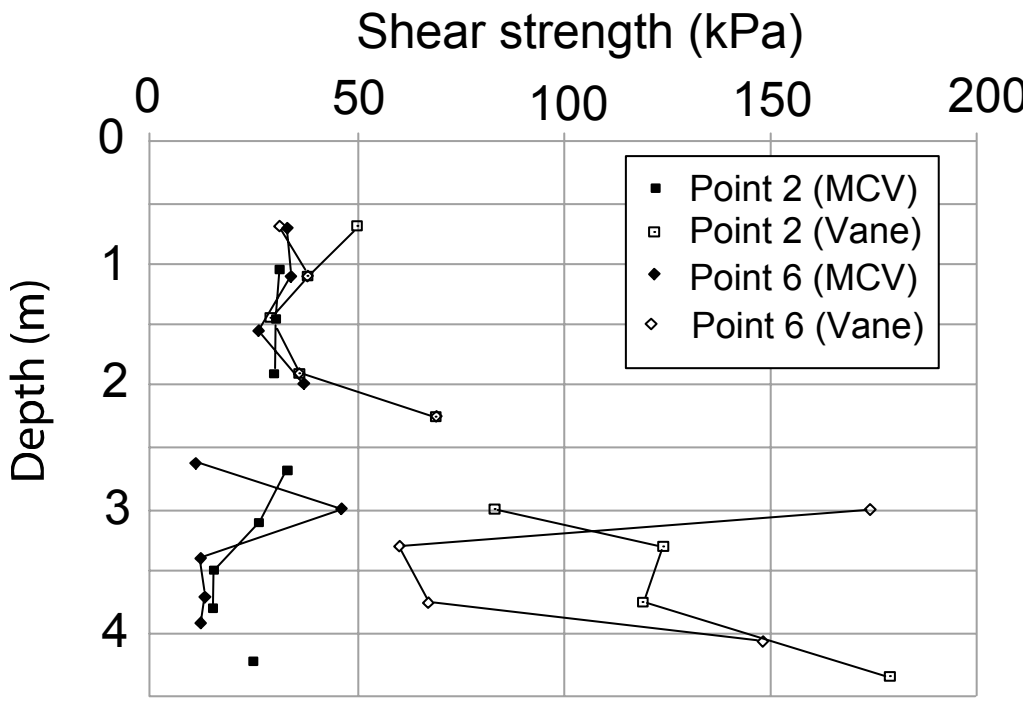


FIGURE 7.26 *Shear strength determined based on MCV and field vane tests. (After Brorsson and Petersson, 1997).*

However, the time lap could only explain a shear strength increase due to consolidation and possibly a small portion of cementation. The confining pressure is normally a substantial contributor to the shear strength in soil. To account for this, triaxial tests should be performed on the MCV specimens to simulate the stress conditions at different depths in the embankment.

Three different cell pressures were used for the triaxial tests performed in this study : 20 kPa, 80 kPa and 200 kPa. Three different types of triaxial tests were performed: Unconsolidated undrained tests, consolidated undrained tests and consolidated drained tests. The

undrained tests were performed to investigate the soil's shear strength at relatively rapid loading, e.g. loading from traffic and also to give comparable results to the vane test. The drained tests were performed to investigate the soil's long-term shear strength.

7.1.6.1 Unconsolidated, undrained triaxial tests

The unconsolidated, undrained triaxial tests (UU-test) were performed to examine the influence of different total confinement pressures. The results are shown in Figure 7.27. The plot shows that the total confinement pressure has its largest relative effect on specimens with low MCV, which should be expected. For specimens with MCV 15 there is no difference in shear strength between tests with no confinement pressure and tests with a confinement pressure of 20 kPa, which confirms the stiffness at much high MCV.

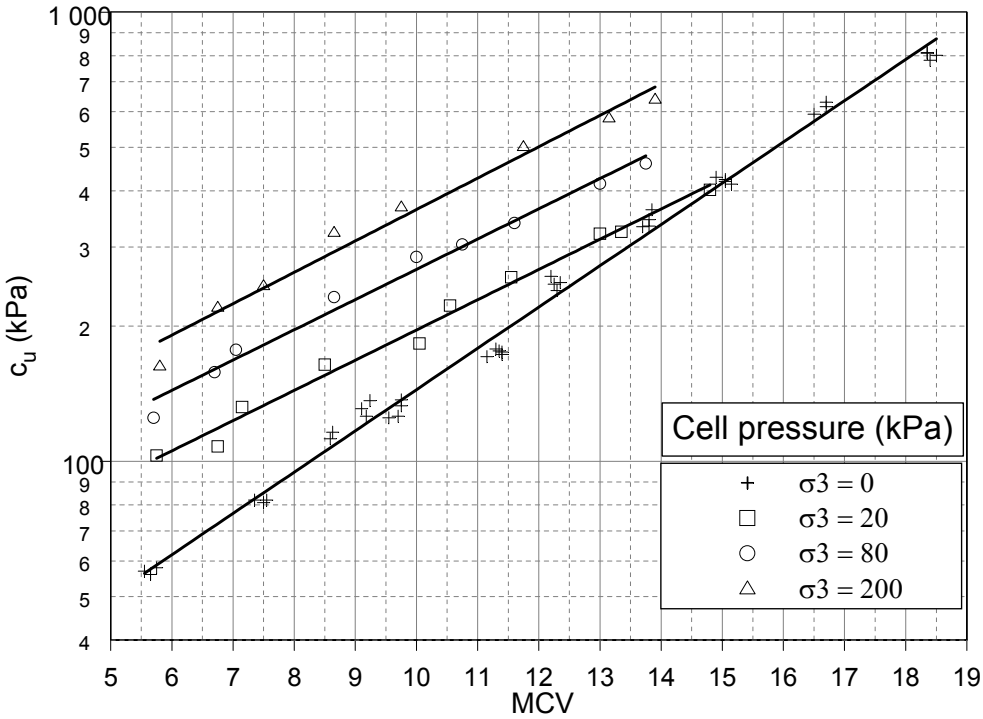


FIGURE 7.27 *Unconsolidated undrained shear strength as a function of MCV for different cell pressures, cf Figure 7.15. E22FN material.*

The same general regression equation used as for the unconfined compression tests was applied to the UU-test data, cf. Equation 7.10. The correlation parameters are presented in Table 7.16

.

Table 7.16: *Parameters for correlation between MCV and unconsolidated, undrained shear strength at different confinement pressures, cf. Table 7.10. Sr* < 1.

Cell presure (kPa)	α	β	No of obs.	R^2
0	17.83	0.21	41	0.994
20	41.66	0.155	10	0.913
80	56.77	0.155	9	0.960
200	72.75	0.161	8	0.974

The plot in Figure 7.27 shows that a confinement pressure considerably increases strongly the undrained shear strength at low MCVs and reduces the slopes of the regession lines, compared to the results of unconfined compression tests, cf. Table 7.10.

7.1.6.2 Consolidated undrained triaxial tests

The results from the consolidated undrained triaxial tests are presented in Figure 7.28. The plot shows the same pattern as for the unconsolidated undrained triaxial tests, i.e. the confinement pressure has the largest effect on specimens with low MCV.

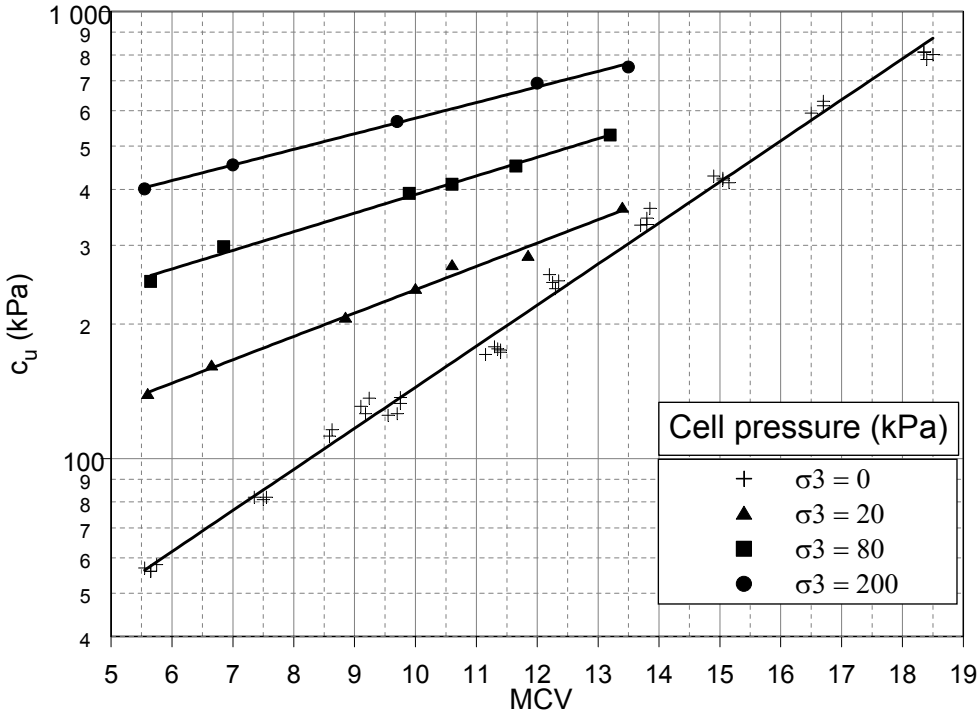


FIGURE 7.28 *Consolidated, undrained shear strength as a function of MCV for different cell pressures, cf. Figure 7.15. E22FN material.*

The results from the consolidated ,undrained tests do not cover MCVs higher than 13.5. However, the results for the 20kPa cell pressure and the unconfined compression test appears to have an intersection at MCV about 15, cf. Figure 7.27.

The regression results for the tests in Figure 7.28 are presented in Table 7.17.

Table 7.17: *Parameters for correlation between MCV and consolidated, undrained shear strength at different confinement pressures, cf. Table 7.10 and Table 7.16.*

Cell presure (kPa)	α	β	No of obs.	R^2
0	17.83	0.21	41	0.994
20	71.7	0.120	7	0.923
80	145.0	0.097	6	0.964
200	248.9	0.085	5	0.974

7.1.6.3 Differences in results between consolidated and unconsolidated, undrained triaxial tests

The data from the consolidated and unconsolidated undrained triaxial test are presented together in Figure 7.29. The results show that the consolidated specimens have higher shear strengths compared to the unconsolidated ones at the same MCV. The regression lines tend to intersect at a very high MCV. The intersection increases with cell pressure i.e. higher cell pressure gives intersection at higher MCV. For a cell pressure of 20 kPa, the regression lines seems to intersect at MCV = 15. A possible explanation is that the applied compaction energy at MCV = 15 has densified the specimen so the cell pressure does not result in any further change in volume of the specimen and thereby no consolidation. It requires a very high confinment pressure to significantly increase the undrained strength of such a specimen. For a plot in a linear - linear relation there will be no intersection rather a merge into one regression line.

The slope β of the regression line for unconsolidated tests appears to be fairly constant at approximately 0.16. However, the corresponding slopes for the consolidated tests show a wider scatter and the β value has here a tendency to decrease with increasing cell pressure. A probable explanation is the changes in water content because of the consolidation.

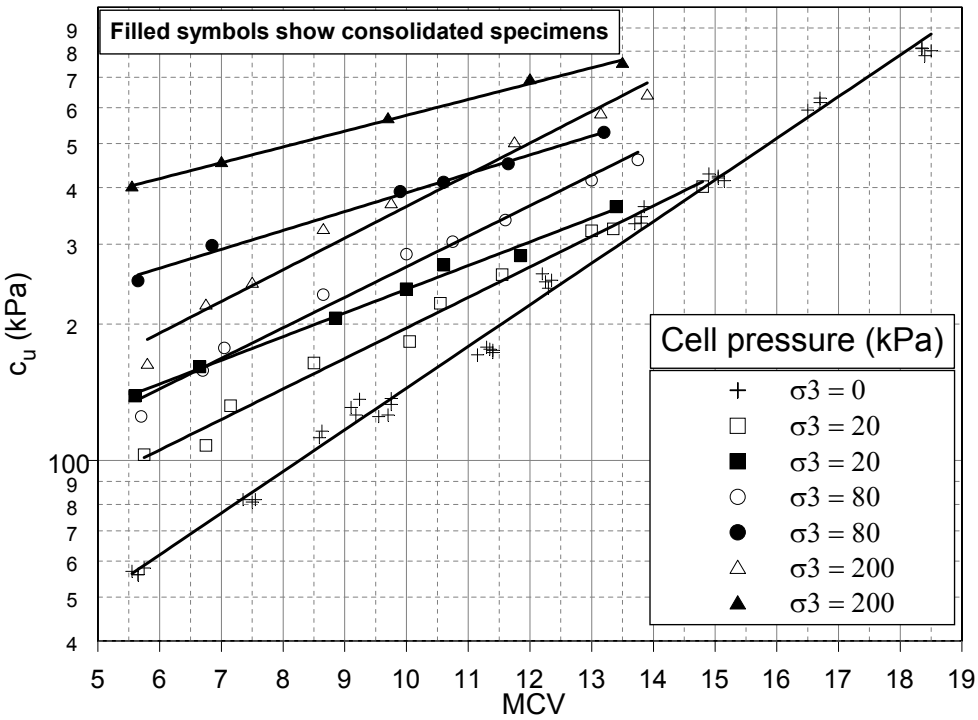


FIGURE 7.29 *Consolidated and unconsolidated, undrained shear strength at different cell pressures and MCVs. E22FN material.*

7.1.6.4 Drained consolidated triaxial tests

Since the cost of drained, consolidated tests is high, the test program was reduced to deal only with three different MCVs. The selected MCVs were 7, 9 and 13, which cover the most common range. The results from these tests are presented in Figure 7.30.

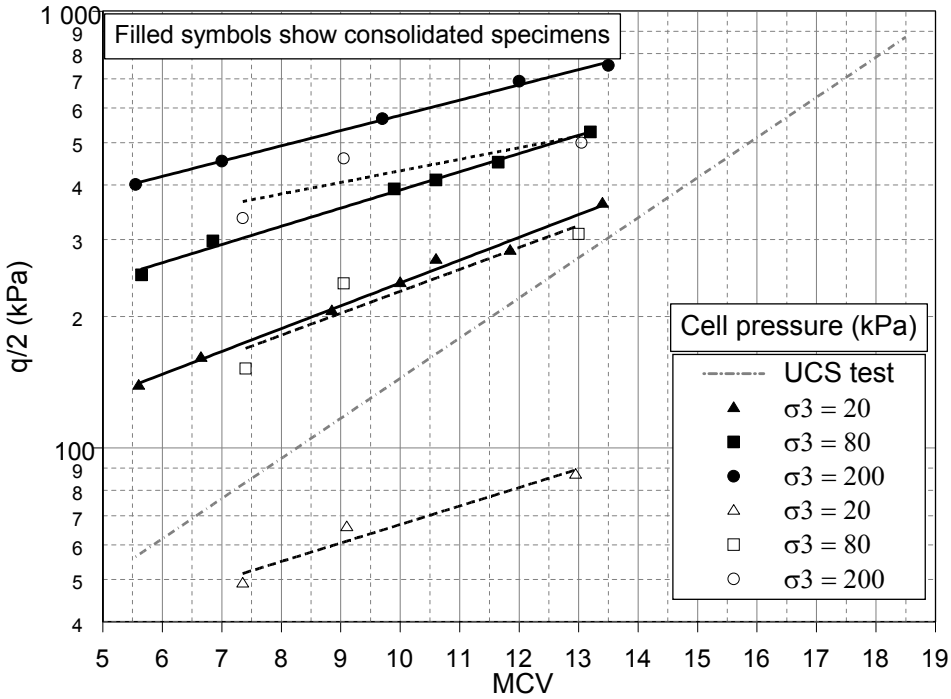


FIGURE 7.30 *Drained and undrained, consolidated triaxial shear strengths at different cell pressures and MCVs. E22FN material.*

The general regression equation for the drained strength tests used in this study is:

$$q/2 = \alpha' \cdot e^{(\beta' \cdot MCV)} \quad \textbf{(EQ : 7.19)}$$

where:

where α' and β' are drained material parameters, cf Equation 7.10. The drained shear strength q/2 is measured in kPa. In this study β' tends to scatter with different confinment pressures.

The regression results for the drained tests in Figure 7.30 are presented in Table 7.18. The plot shows similar slopes for both drained and undrained tests.

Table 7.18: *Parameters for correlation between MCV and consolidated drained shear strength for different confinement pressures, cf. Table 7.10, Table 7.16 and Table 7.17.*

Cell pressure	α'	β'	No of obs.	R^2
20	25.21	0.097	3	0.994
80	71.88	0.1156	3	0.953
200	234.1	0.061	3	0.815

The plot in Figure 7.30 shows a reduction in shear strength for all drained specimens compared to the undrained specimens. A small part of the reduction of the shear strength can be found in the difference in strain rate. However, during an undrained triaxial test on a dense compacted specimen negative pore water pressure is developed and increases the shear strength. In a drained test no negative pore pressure is developed that could contribute to the shear strength.

Using the regression parameters in Table 7.17 and Table 7.18 shows that for a confining pressure of 20 kPa and a MCV = 10 there is a drop in shear strength from 238 kPa for the undrained test to 66 kPa for the drained test.

7.1.7 Ageing effect on the shear strength

Malmborg (1992) and Möller & Malmborg (1995) reported a considerable strength increase due to the ageing of compacted clay till. They found an increase of 100% during the first 24 hours and an additional 15 to 30% during the first month after compaction. However, later studies by Lindh (2000) showed no strength increase with time in stored compacted clay till specimens. To further study the ageing effect, two different soils, E22 F.N. and Hyllie, were used. The Hyllie material is a clay till very similar to that studied by Malmborg (1992), Möller & Malmborg (1995) and Lindh (2000). The specimens were compacted with MCA according to BS 1377 (Anon., 1990b). Some of the specimens were removed from their moulds and covered with paraffin. Others were stored within their MCV moulds and also covered with paraffin. After the storing period, a check was carried out to ascertain that there was no loss in weight during the storing i.e. no loss in water content. The strength-increase with time in these specimens is shown in Figure 7.31.

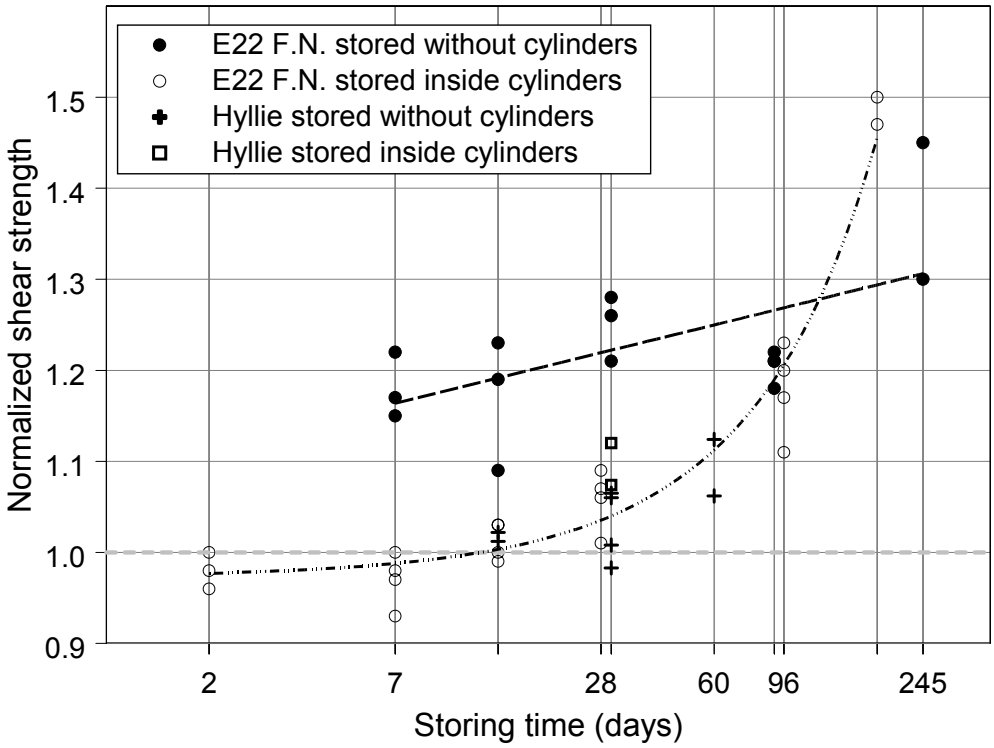


FIGURE 7.31 *Normalized shear strength for the stored MCV specimens. The normalization is performed withresults from the regression analysis presented in Table 7.10, which corresponds to tests performed directly after compaction of the specimens. (Lindh, 2003).*

Compared to the E22 F.N. material, the Hyllie material was found to be more inert and showed a strength increase of less than 15% during 60 days of storing. A possible explanation for some of the high shear-strength increases reported by Malmborg (1992) and Möller & Malmborg (1995) is drying of the specimens during storing. The reported increase in strength was 100% after 24 hours and this is more difficult to explain.

Another factor that may be important is the mould type. In this study, the storage mould was made of stainless steel. In the previous studies showing large time effects, the cylinders were made of ordinary steel. In such cylinders there may be corrosion, which in turn may cause friction between the mould and soil. This may also increase the force required to push the specimens out of the mould and may possibly cause an extra compression and a higher shear strength. In the study performed by Lindh (2000) the specimens were compacted and stored in PVC-tubes.

This study shows that there is a strength increase with time in compacted fine-grained tills. However, this strength increase is marginal compated with the strength increase developed by a stabilising agent i.e. lime or cement and could thereby be ignored in soil stabilising. However, for embankments of unstabilised fine-grained till this effect could be beneficial.

The ageing process is not fully understood but the soil chemistry may be an important factor. Further studies of ageing effects must be performed, preferably with full volume control during the storing time.

7.1.8 The effect of sample-preparation techniques

According to the literature it is very common that laboratory work involves drying of the soils and there after wetting to desired water contents (Olson *et al*, 1965; Barden & Sides, 1970; Börgesson, 1981; Brown *et al.*, 1987; Rogers *et al.*, 1999; Fleureau *et al.*, 2002; Cunningham *et al.*, 2003).

The differences between specimen-preparation in Sweden and in Britain, for example, mainly consist of the methods used to achieve the

range of moisture contents required to obtain an MCV calibration line. In Sweden, full drying of clayey soils before any type of testing is avoided whenever possible, since drying and rewetting is considered to significantly alter the structure of the soil. Swedish practice thus requires drying or wetting of a soil from its natural moisture content to achieve the desired range of moisture contents. The British Standard method (Anon., 1990c) uses an air-dried soil that is wetted to achieve the desired range of moisture contents. Both methods are developed to deal with the hysteresis effect between drying and wetting of a soil (i.e., the soil properties are different between the drying phase and the wetting phase), albeit using opposite approaches; one starts with a wet soil and the other with a dry soil. For both the Swedish and the British Standard methods the soil needs to rest for some time to achieve homogeneous conditions in the soil samples (i.e., even distribution of the water within the soil). In BS (Anon.,1990c) the minimum storage time for cohesive soils is 24 hours in an airtight container. This methodology was also performed by Fleureau *et al.* (2002). In the tests in the current study, the minimum storage time was one week for both partially dried soil and wetted soil. To compare the effect of different drying methods, both air-dried and oven-dried soils were prepared. The oven-dried soils were dried for at least 24 hours and then cooled to room temperature before re-wetting.

The Swedish method is similar to that proposed by Jones and Greenwood (1993). Their method essentially involves one test at the natural moisture content and further tests on samples that have been wetted-up or partially dried to higher and lower moisture contents respectively. Winter (2001) reported the results of an extensive testing programme to compare the standard method, as presented by Matheson and Winter (1997), and that proposed by Jones and Greenwood (1993). No large and statistically significant differences were found in the results

obtained from the two methods (e.g., Figure 7.32), but operational difficulties were experienced with the Jones and Greenwood (1993) method and this was not recommended for further routine use. There are, however, two key differences between the Jones and Greenwood (1993) method and the Swedish method. These are as follows:

1. Natural moisture content MCV data are not incorporated in determination of the calibration line using the Swedish method. Winter (2001) found the incorporation of these data to be particularly difficult with fine-grained soils owing to the differences in the residual soil structure created by the different sample preparation methods (see Winter and Matheson, 1997).

2. In Sweden, the high natural moisture contents of the soils normally entail that all tests are carried out on air-dried specimens selectively to the required moisture contents and that none of them have had their moisture content increased. This effectively minimises the effects of hysteresis on the wetting-drying curve. It does, however, rely upon encountering wet soils in the field. A drier condition could be compensated for in the laboratory by first wetting the soil sample, but this increases the sample preparation time.

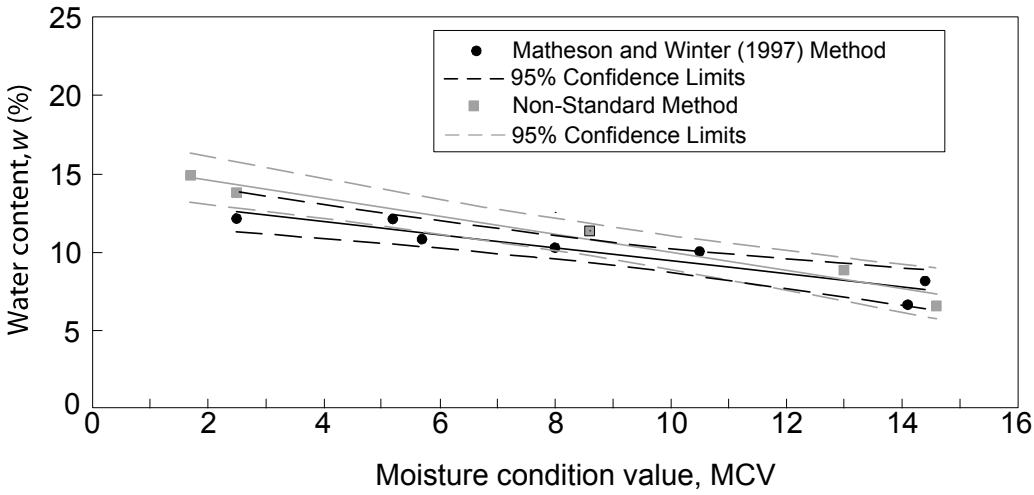


FIGURE 7.32 *MCV calibration lines obtained using the standard and non-standard methods of specimen preparation (after Winter, 2001).*

The results presented in Figure 7.33 (E22) and Figure 7.34 (Östra Torn) show that the differences in sample preparation affect the MCV calibration lines to a greater degree for soils with a higher clay and silt content (i.e., Östra Torn). This indicates that the clay and silt particles are most affected by drying and wetting. The results also indicate that it is the drying of the soils that makes the main difference and not how the soils are dried (i.e., air-drying or oven-drying).

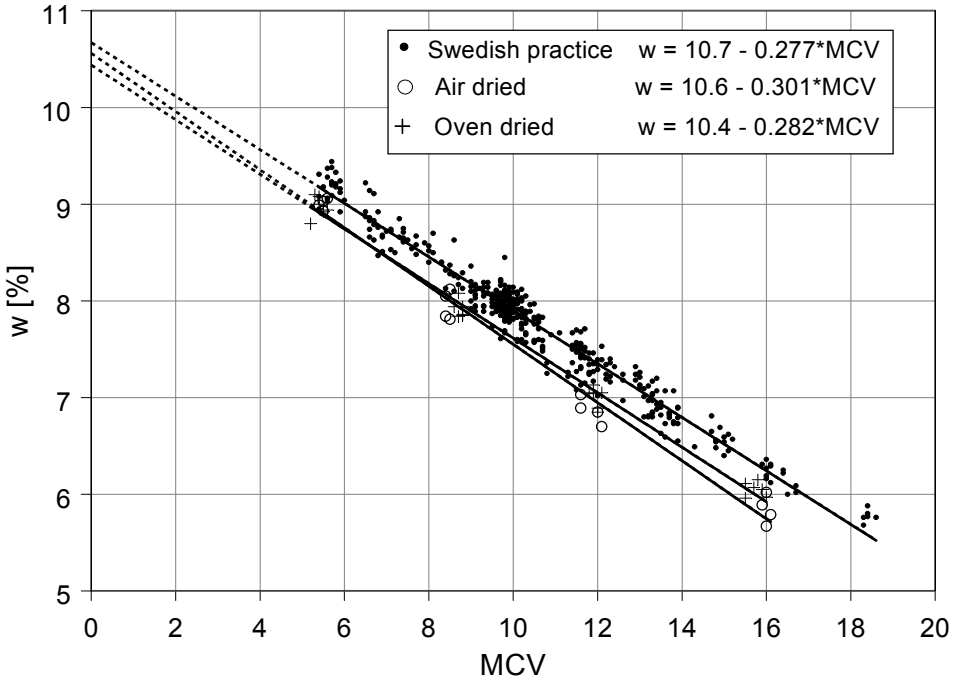


FIGURE 7.33 *MCV Calibration lines for the E22 material treated in three different ways (Lindh and Winter, 2003).*

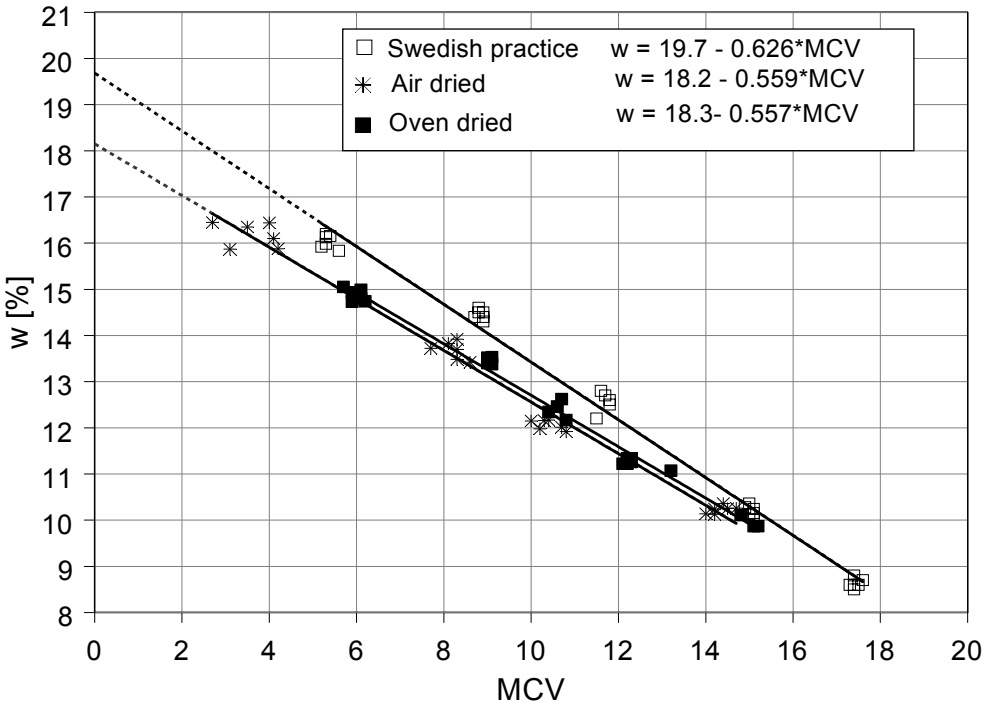


FIGURE 7.34 *MCV Calibration lines for the Östra Torn material treated in three different ways (Lindh and Winter, 2003).*

The differences in soil preparation between Sweden and Britain do affect the MCV calibration line. However, the Swedish method can be very time-consuming because of the need for controlled drying. To achieve the best quality in MCV calibration, the differences in moisture content between samples in the calibration should be equidistant. To obtain this with the Swedish method, the drying procedure needs to be carefully controlled. The drying process creates agglomerats with a wet core surrounded by a drier shell. This difference in moisture content

between the surface and core could affect the MCV calibration because of high suction on the surface of the soil agglomerations. To overcome this, the soil must achieve equilibrium and this can take several days for a clayey soil. The problem with uneven distribution of moisture in the soil can also arise when the soil is wetted. However, for the completely air-dried and oven-dried soil most of the bonds in the agglomerations are broken and the moisture appears to become more evenly distributed when wetted.

The effect of air- or oven-drying of a soil does not only affect the MCV calibration line, it also affects the correlation between water content and undrained shear strength, cf. Figure 7.35.

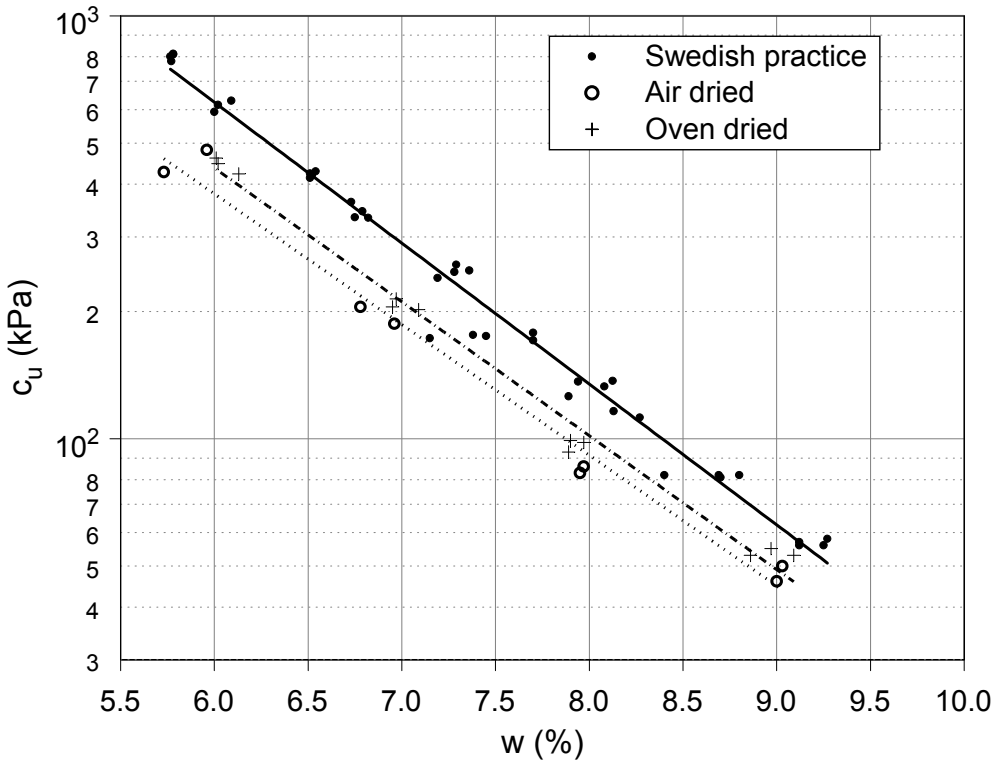


FIGURE 7.35 *Shear strength as a function of water content for MCA-compacted E22FN soil treated in three different ways. The air- and oven-dried materials show lower undrained shear strength compared to partially dried soil compacted and tested at the same water content.*

The drying of the soil results in permanent changes in soil structure. These take place in the clay-size fraction of the soil. One of these effects is that no water menisci are formed at very low water contents and the lack of water menisci makes the soil behave like a cohesionless, dusty material (Olson, 1963).

This permanent change in soil structure could be an explanation for the difference between undrained shear strength for the same water contents in the specimens treated according to Swedish practice, versus specimens that are dried. These changes in soil structure also affect the undrained shear-strength calibration with MCV, cf. Figure 7.36. The dried specimens show, for the same MCV, a lower undrained shear strength compared to the specimens treated according to Swedish practice. However, this is in most cases on the safe side and does not affect the soils acceptability in an earthwork except in special cases.

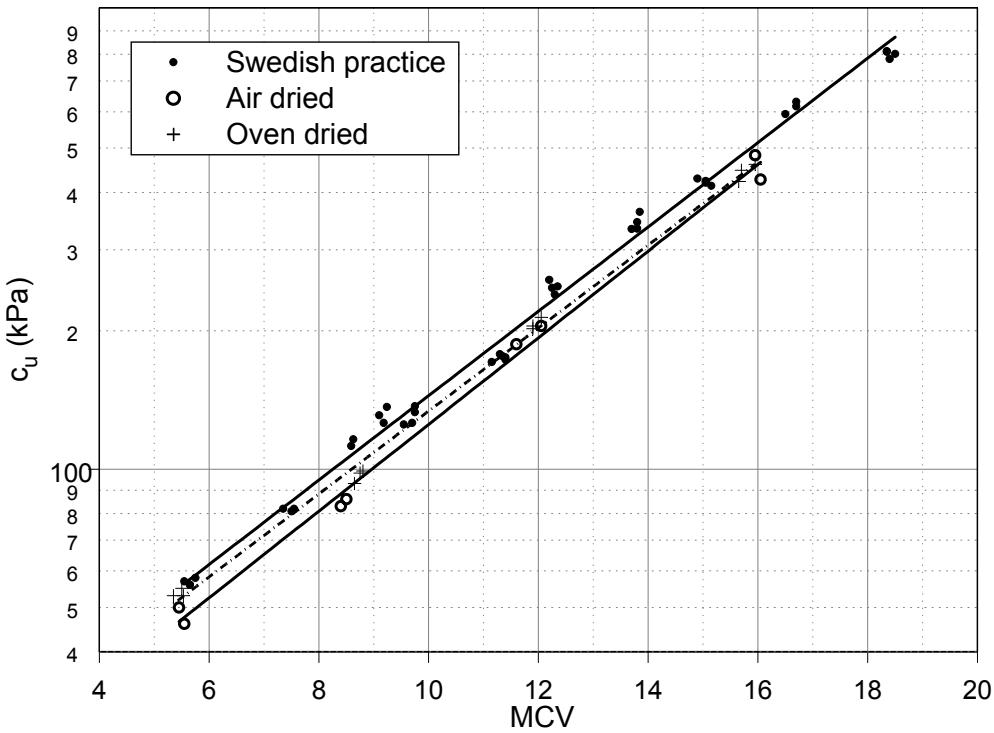


FIGURE 7.36 *Undrained shear strength for different MCV's and for different specimen-preparation techniques for the E22FN material.*

7.1.9 Results from chemical analysis

Tests were performed to determine the chemical composition of the soils. The tested properties were: pH according to SS-ISO 10390 (Anon., 1994b), Organic content according to SS 027107 (Anon., 1990a), Calcium carbonate according to SGI report 27E (Larsson *et al.*, 1987), and Free lime according to prEN1744-1:1994 (Anon., 1994c). The results are presented in Table 7.19.

Table 7.19: *Chemical properties of the tested soils*

Soil	pH-H_2O	Org. content	$CaCO_3$ (%)	Free lime (%)
Hyllie	8.6	<0.1	27.3	<0.1
Östra Torn	8.6	<0.1	13.5	<0.1
Sturup	8.2	<0.1	3.1	<0.1
E22FN	8.4	<0.1	20.1	<0.1
E22Hurva	8.9	<0.1	13.2	<0.1

The results in Table 7.19 show almost identical pH values, no free lime and no organic content. However, the calcium carbonate content varies between the different soils, probably because of the different genesis of the tills. The E22 F.N. and Sturup soils have similar grain size distributions, cf. Figure 7.1 but their $CaCO_3$ content is different.

The main clay minerals are smectite-illite (mixed layer), illite, chlorite and kaolinite.

7.1.10 Recommendations

Densification is essential to obtain a stable fill that can support roads, railways or buildings besides its own weight. A densification can be achieved during construction or by a surcharge afterwards. If the alternative of densification of the fill during construction is chosen, the compaction criteria should be based on sufficient data to support the designer's engineering judgment. The pre-investigation should preferably contain a graph similar to Figure 7.37. These data should be sufficient for a contractor to decide the type of earth moving plant and capacities for different conditions of the fill. It could also aid decisions of when soil modification/stabilisation needs to be considered.

If these data are combined with field data regarding the variation of the soil's natural water content during the year, a good prediction of the soil's workability, and possible actions to improve this by soil modification can be planned at an early stage.

The MCV method is a very useful method for predicting the soils water sensitivity, workable range regarding water contents and predicting the soil's undrained shear strength after compaction. The MCV method is no compaction control but rather a method to determine if a soil can be compacted sufficiently at the actual water content. The MCV method has several advantages compared to the Proctor method for fine-grained soils. The Proctor method on the other hand, has only one advantage over the MCV method, which is the geographical spread and the long-time experience of the method.

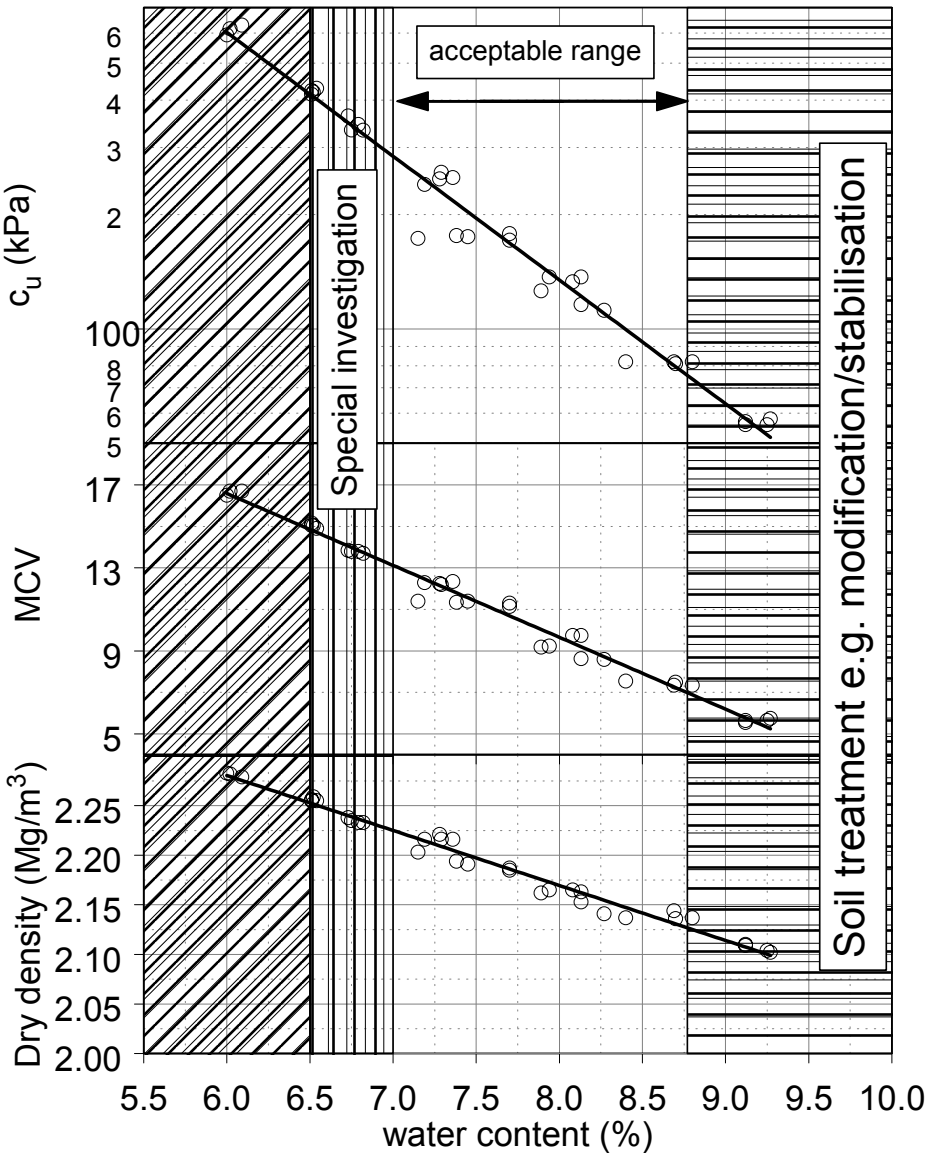


FIGURE 7.37 *Relationship between water content; dry density, MCV and shear strength for the E22FN material.*

7.2 Laboratory results - stabilised soil

Soil stabilisation and soil modification change the structure of the soil and thereby change the soil's engineering properties. One of the main means to achieve a high quality soil stabilisation is compaction. The compaction properties of the soil-binder mixture are different from those of the unstabilised soil and are important to determine. Other important parameters are the undrained shear strength directly after compaction and the strength increase with time.

7.2.1 Changes in compaction properties

The change in compaction properties depends on binder type and the total amount of binder or binders. The compaction properties are often presented in the form of achieved dry density versus water content at compaction.

7.2.1.1 Changes in compaction properties for the PBL material

Another way to present the compaction characteristics for stabilised soils is to plot dry density against binder content, see Figure 7.38. In this type of plot, the effect of the binder content can be illustrated for several types of binders.

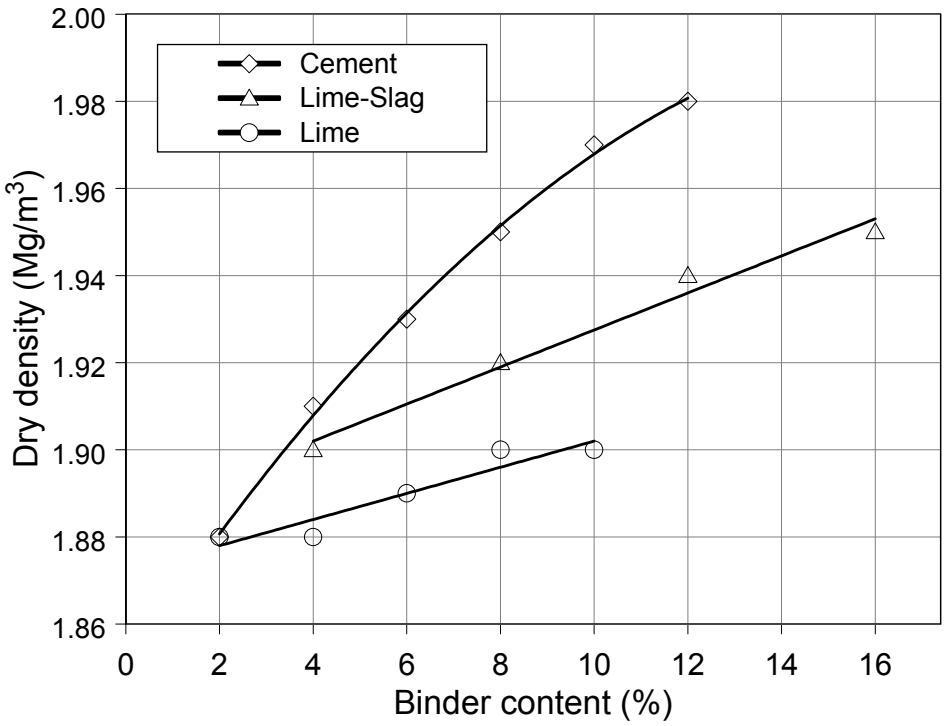


FIGURE 7.38 *Dry density as a function of binder content for the PBL soil. All specimens are compacted according to modified Proctor one hour after mixing the soil and binder. (Lime = hydrated lime).*

The achieved dry density is a result of several factors. Since the binders have different specific densities both compared to each other and compared to the soil itself, this will naturally affect the dry density. Another factor is the change in water content because the binder absorbs some of the soil water. Different binders affect the water content differently. In Figure 7.38, the results show that cement affects the dry density most and hydrated lime has the least effect on dry density. The effect of the blended binder consisting of 50% lime and 50% slag is somewhere in-between that of cement and that of hydrated lime.

The reduction in water content from the same data set is shown in Figure 7.39.

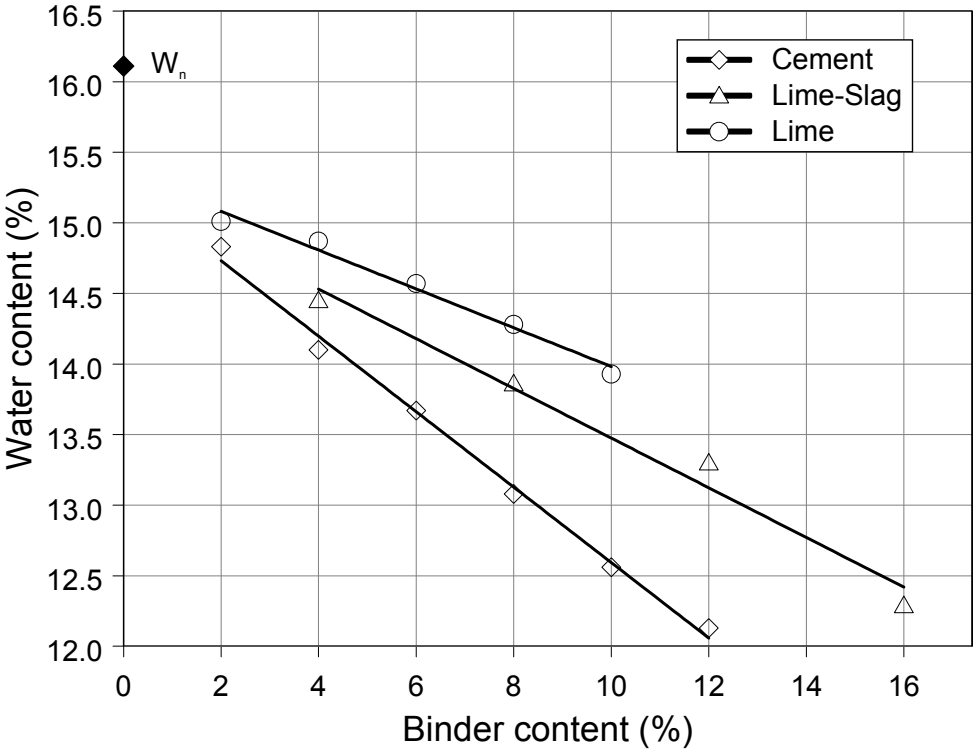


FIGURE 7.39 *Water content as a function of binder content for the PBL soil type. The natural water content for unstabilised soil is 16.1% and the optimum moisture content 9.6%.*

The pattern in Figure 7.38 is also reflected in Figure 7.39. Cement has the greatest effect on the water content of the tested binders and lime has the lowest. The natural water content for the PBL soil is 16.1%, cf. Figure 7.2. Table 7.7 shows that the optimum moisture content is 9.6% for this soil without any stabilising agents. Mixing a soil with a binder will

reduce the dry density and increase the optimum moisture content. The initial reduction in water content is approximately 1% with 2% binder and the reduction then decreases, depending on the type and amount of binder. The 1% reduction in water content caused by the binder is small, but due to the simultaneous change in soil structure, it will affect the compaction characteristics much more than a corresponding reduction in a natural soil.

The second compaction method used in this study, MCV, presents another way to compare how different binders affect compaction. In contrast to the Proctor method, the compaction energy varies, which results in MCV - binder relations as shown in Figure 7.40.

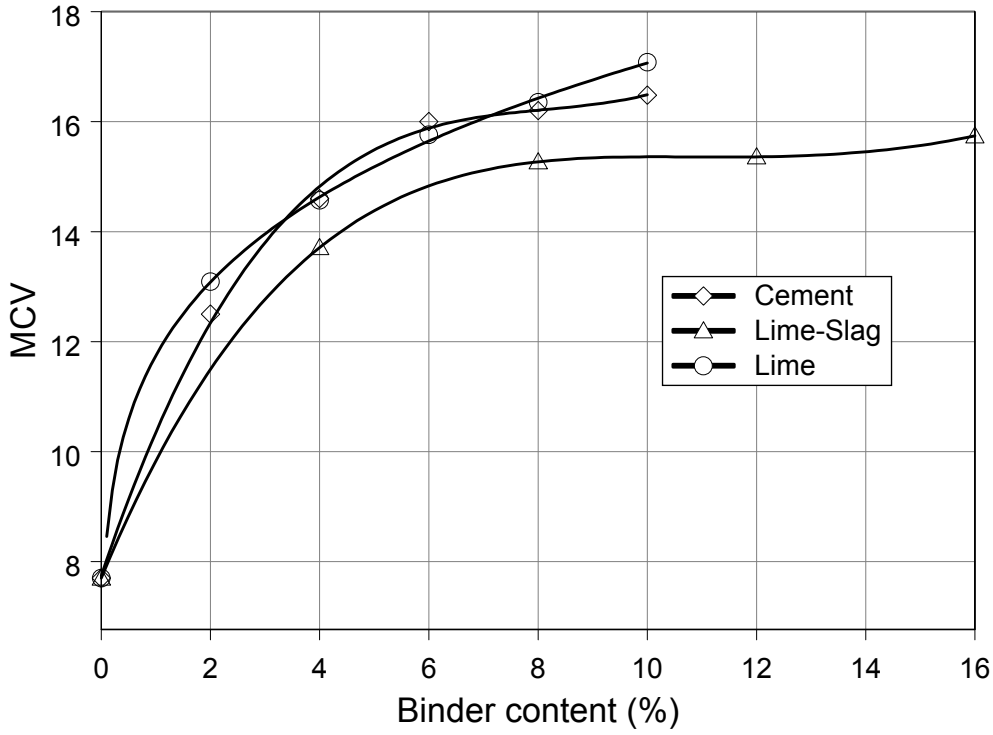


FIGURE 7.40 *MCV as a function of binder content for three different binders mixed with PBL soil. All specimens were compacted one hour after mixing.*

The results in Figure 7.40 indicate that cement and lime affect the MCV in approximately the same way. However, this is not the case for the results from the Bromölla and Petersborg soils, see Figure 7.41 and Figure 7.42. The lime-slag binder gives the lowest increase in MCV and this is also found in the results from the Bromölla and Petersborg soils. The Petersborg and PBL soils are very similar and no ready explanation can be found why the relation between the effects of lime and cement are different for the two types of soil. Another condition that needs to be commented upon is the "MCV 14 limit", cf. Section 6.4.3.1 , page 121.

At approximately MCV 14 the compaction energy corresponds to that of the modified Proctor method. Approximately 3% lime or cement can be used without additional water if this energy is set as an upper limit for compaction. If additional water is needed, this may require remixing of the soil and an extra compaction. The MCV for the unstabilised soil at its natural water content was 7.7 and this is just above the lower limit used in Great Britain for acceptance as fill material (Perry *et al.*, 1996). However, an MCV of 7.7 is accepted for Swedish conditions.

7.2.1.2 Changes in compaction properties for the Bromölla material

For the Bromölla and Petersborg soils the adopted procedure and evaluation technique was different owing to the large number of different binders and combinations that were tested. The test series were preceded by an experimental design. In contrast to the tests on PBL soil, only one fixed binder content of 2.5% was used. For the Bromölla soil a boundary design with six different binders and binder combinations was chosen. Double replicates were performed for all specimens and a 5% significance level was used. The response surface with MCV as response variable is presented in Figure 7.41.

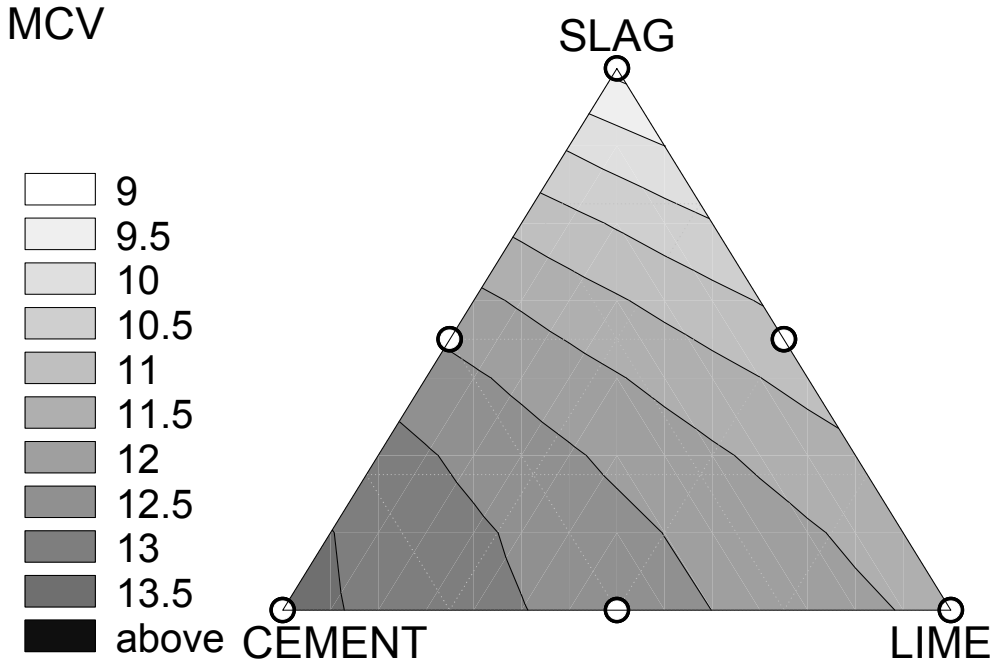


FIGURE 7.41 *MCV response surface for stabilised Bromölla soil. Binder content 2.5%. The delay between mixing and compaction was one hour. R^2=0.999, adj. R^2=0.999.*

The results presented in Figure 7.41 show that cement increases the MCV much more than lime does. Slag gives the lowest increase in MCV and a blend of lime and slag affects the MCV somewhere between lime and slag.

7.2.1.3 Changes in compaction properties for the Petersborg material

For the Petersborg soil , which is a silty clay till, the results are similar to those for the Bromölla soil, see Figure 7.42.

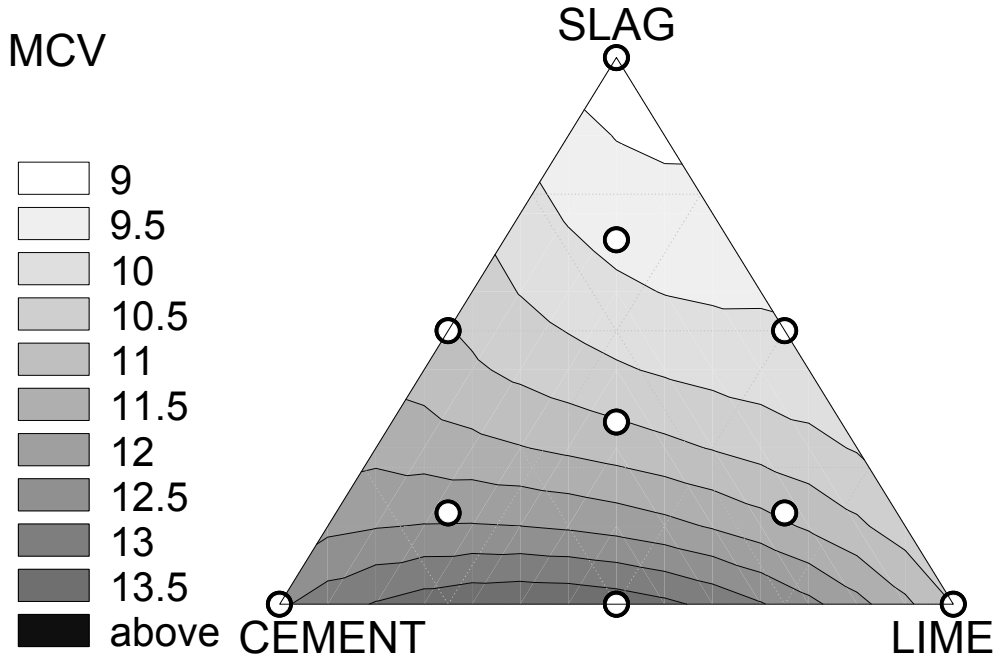


FIGURE 7.42 *MCV response surface for stabilised Petersborg material. Binder content 2.5%. The delay between mixing and compaction was one hour.* R^2*=0.875, adj* R^2*=0.843.*

Figure 7.42 indicates a significant interaction effect between cement and lime. This interaction results in an even higher MCV than for cement only. Without this interaction, a blend of cement and lime would give an MCV somewhere in between the MCVs of cement and lime respectively. The Petersborg soil was also tested with two delay times, 3 and 5 hours respectively. The results are presented in Figure 7.43 and Figure 7.44 respectively.

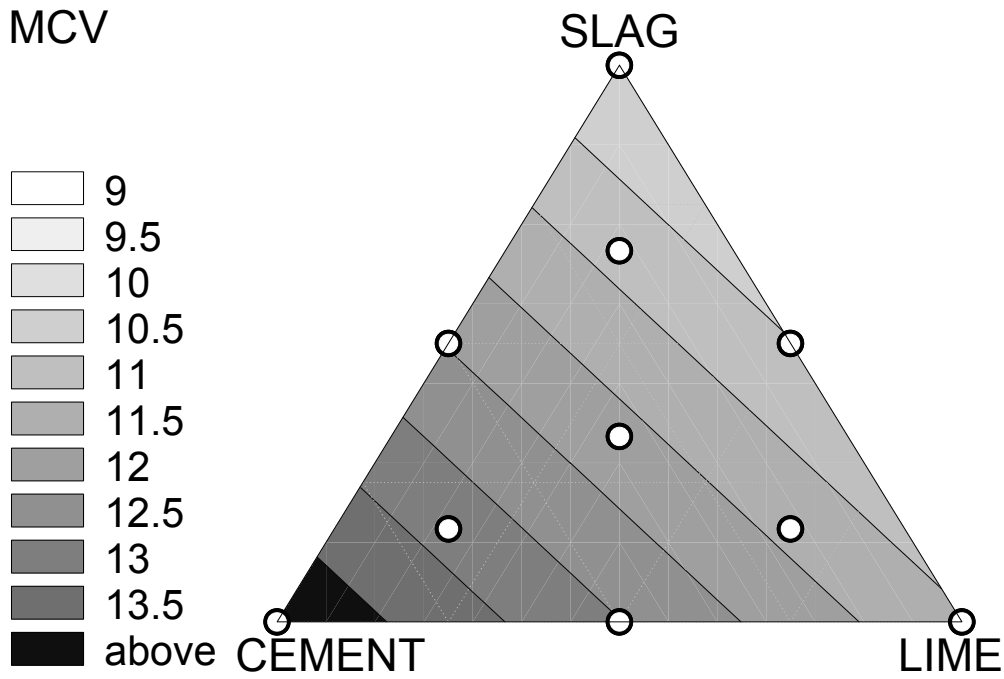


FIGURE 7.43 *MCV response surface for stabilised Petersborg material. Binder content 2.5%. The delay between mixing and compaction was 3 hours. R^2=0.748, adj R^2=0.718.*

In Figure 7.43, the main pattern remains from Figure 7.42. However, in this case only the main effects of the different binders remain significant. This gives the straight contour lines in the plot. No interaction effects could thus be found in this test series. Figure 7.44 shows the results from the test with 5-hour compaction delay.

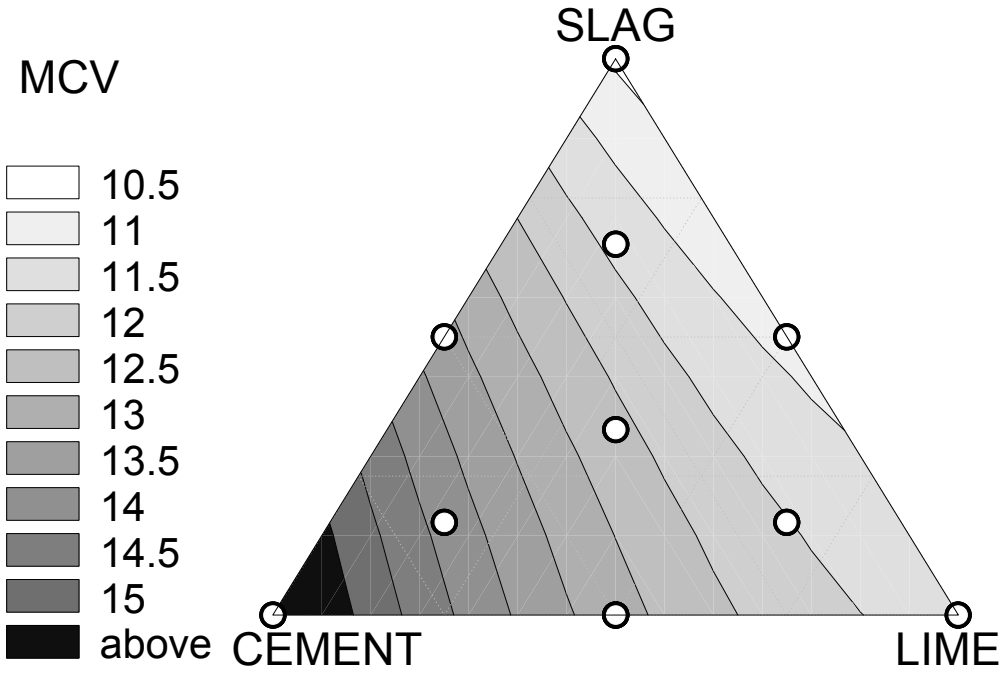


FIGURE 7.44 *MCV response surface for the Petersborg material. Binder content 2.5%. The delay between mixing and compaction was 5 hours. R^2=0.871, adj R^2=0.857.*

In Figure 7.44, the main pattern remains from Figure 7.43. However, in this case the main effects together with a small interaction could be found significant. This causes nearly straight contour lines in the plot.

The results of 6 tf the original 10 different recipes in this test series are plotted as MCV versus time delay between mixing and compaction in Figure 7.45.

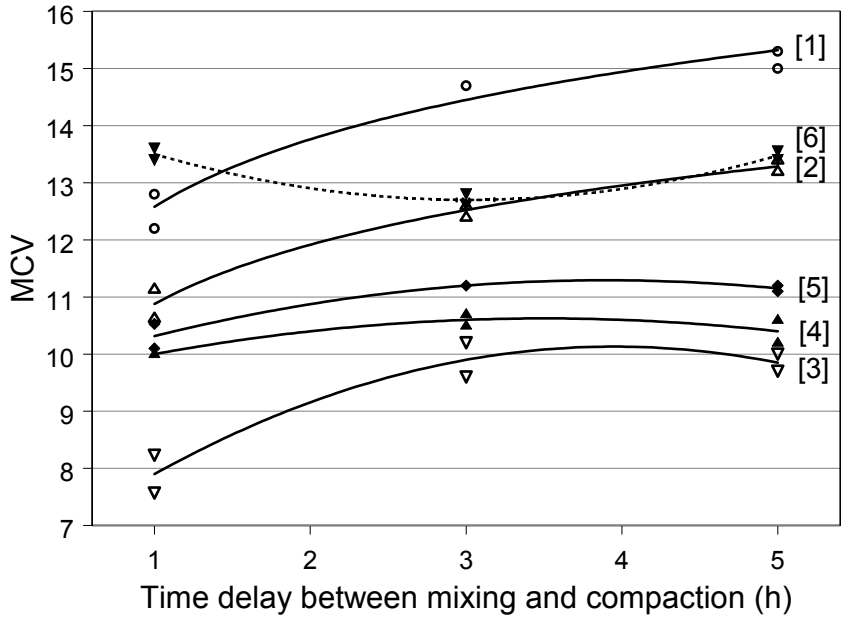


FIGURE 7.45 *MCV as a function of time delay between mixing and compaction for the Petersborg material. Binder content 2.5%. [1] Cement, [2] 50% Cement + 50% Slag, [3] Slag, [4] 50% Slag + 50% Lime, [5] Lime, [6] 50% Lime + 50% Cement.*

Three types of binders show the largest increase in MCV with time delay. These are cement, cement-slag and slag. However, the slag soil mixture had a low initial MCV and the negative effect of the delay is therefore balanced. For cement, the increase between one and three hours involves going from an acceptable MCV to an unacceptable MCV. The blend containing lime-cement yields the highest initial MCV and then a small decrease in MCV after three hours. No plausible explanation

for this could be found. Additional tests were performed but these confirmed the results. It is notable that slag-lime and lime mixtures are very little affected by the time delay. This result should be compared to the results presented by Dumbleton (1962), cf. Figure 5.19. Dumbleton also found that cement-stabilised soil had a larger decrease in dry density with delay time before compaction compared to lime-stabilised soil.

The general effect of soil modification as measured with the MCV method is illustrated in Figure 7.46. The Figure shows MCV calibrations for the Petersborg material. Unfortunately, the MCV tests were made early in this project and therefore based on the wrong soil mass. However, the results can still serve as a comparison between stabilised and unstabilised soils.

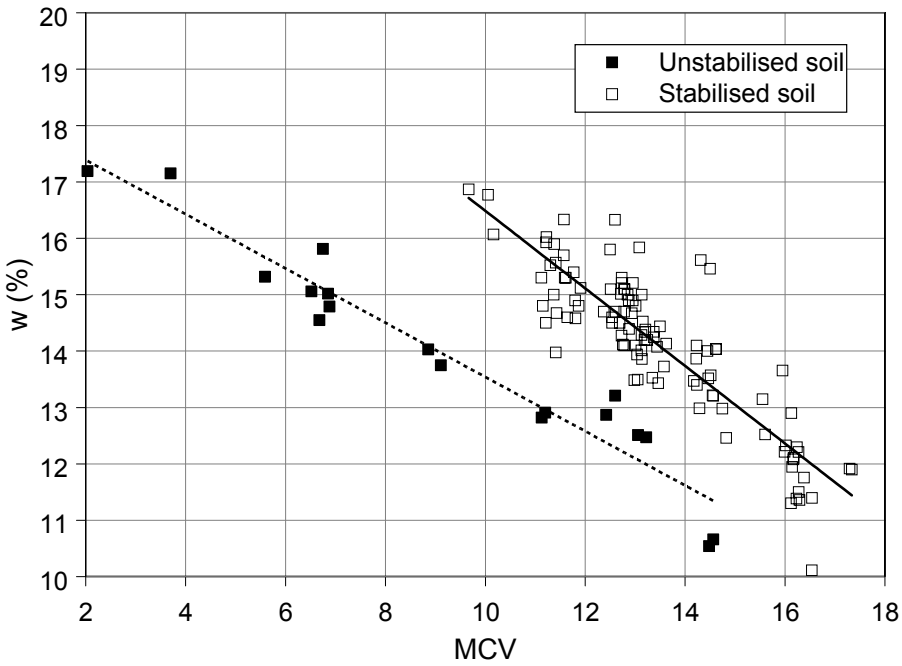


FIGURE 7.46 *MCV calibrations for stabilised and unstabilised Petersborg soil. The soil was stabilised with different types of binders.*

7.2.1.4 Changes in compaction properties for the E22FN material

As for the PBL material, the compaction properties have here been studied with modified Proctor. This test was a subtest to investigate the binder's working period. The test consisted of 32 specimens in a CCD. The independent factors were binder content (Cement 1, 3 and 5%), delay time between mixing and compaction (1, 3 and 5 hours) and curing time between compaction and strength tests (1, 15 and 29 days). The response variables were dry density and unconfined compressive strength.

The dry density was determined after compaction and before storing. This means that the curing time does not affect the result. The result from the ANOVA test is presented in Figure 7.47.

The Figure 7.47 shows only the significant effects, the other effects have been put in the error term, cf. Appendix A. As shown in Figure 7.47 only three effects are significant; delay time, cement content and the interaction between delay time and cement content. The delay time has the largest impact on dry density followed by the interaction term and cement content. It is notable that the interaction term has more impact on dry density than the binder content.

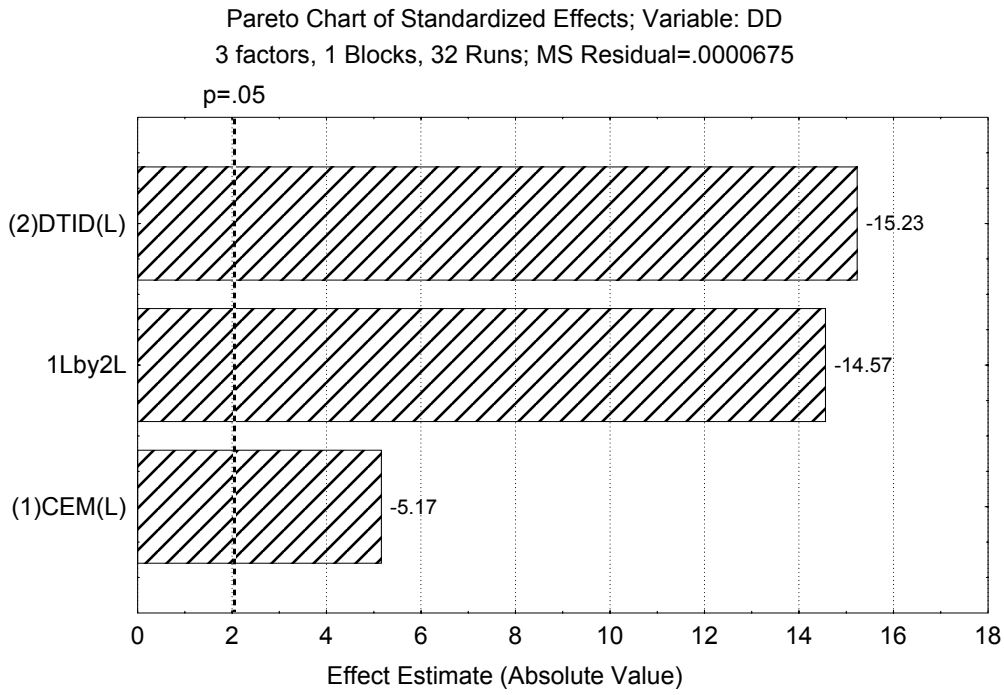


FIGURE 7.47 *Pareto chart showing the significant effects on dry density.*

The response surface of the test is shown in Figure 7.48. The effect of delay time is very pronounced for high binder content, i.e. for a binder content of 5% the dry density is very affected by the delay time. This should be compared to a binder content of 1% where the delay time gives a small increase of the dry density. It should also be noted that for a delay time of 1 hour the dry density increased with binder content. The same effect is also observed for the PBL material, cf. Figure 7.38. If the delay time increases to three hours the negative effect on dry density could be seen for increased binder content.

The reason for the decrease in dry density depends on the strength increase in the stabilised soil owing the cement reactions. This means that

some of the compaction energy is used to break the bonds created by the cement reactions and thereby less compaction energy is used for densification. This leads to lower dry density and increased void ratio.

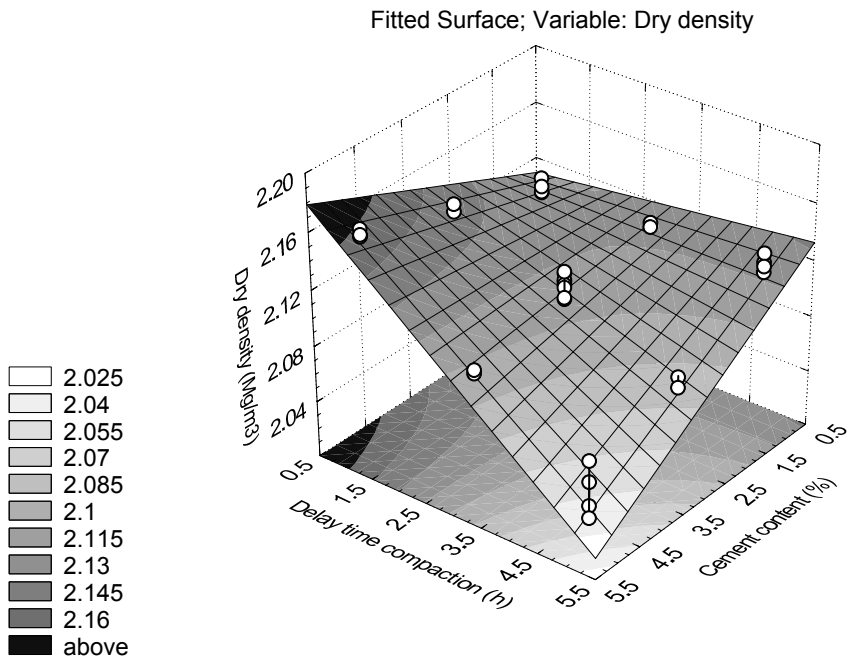


FIGURE 7.48 *Dry density as a function of cement content and delay time between mixing the soil with cement and compaction with modified Proctor. Non significant effects removed from regression. E22FN material.*

Figure 7.48 indicates that the binder's working period is between one and three hours without affecting the soil's dry density in a negative way.

7.2.2 Changes in strength properties

In stabilised soils, the strength is dependent on binder type, total amount of binder and compaction. Since different binders have different reaction times, the strengths were studied during three different curing periods. The first curing period studied is 7 days after compaction. This time lapse was chosen to show the early strength that can have a great importance for practical road-stabilising purposes. The second time lapse, 28 days, was chosen because of the cement binder. At this time, most of the alite component in the cement has reacted and the strength growth is decreased. The third and last time lapse was 90 days and was intended to give the pozzolanic reactions time to develop.

7.2.2.1 Strength properties for PBL material

The Proctor-compacted specimens from Figure 7.38 were also tested for unconfined compressive strength (UCS). The results are presented in Figure 7.49.

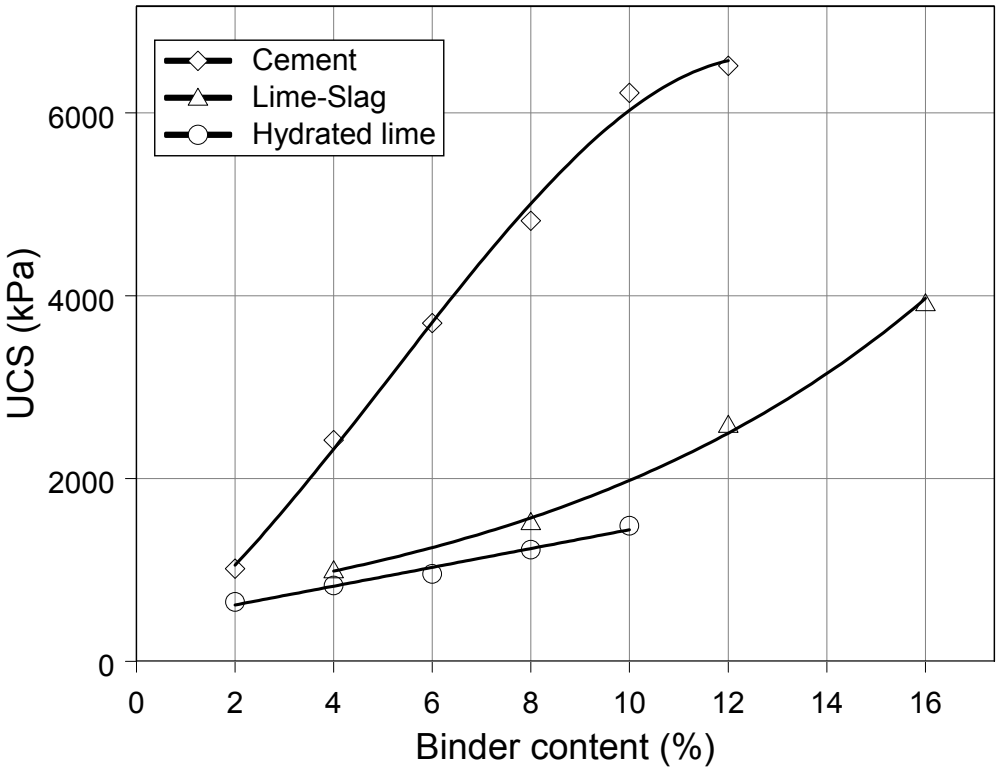


FIGURE 7.49 *UCS as a function of binder content for the PBL material. All specimens were compacted according to modified Proctor one hour after mixing. Curing period for all specimens was 7-days (Lime = hydrated lime)*

The dry density relations to binder content in Figure 7.38 are very similar to the UCS relation to binder content in Figure 7.49. This indicates that there is a relation between UCS and dry density which is relatively independent of binder type, see Figure 7.50. The dry density could be used as a rough strength indicator within reasonable binder contents and with a specific curing time.

For blended binders such as lime-slag the relation may be a little more complicated. The error obtained with an assumption of a linear relation could still be acceptable for some applications. The relationships shown in Figure 7.50 are based on 7-days' UCS. Since the different binders develop strength with time in different ways, these relations are not valid for other curing periods.

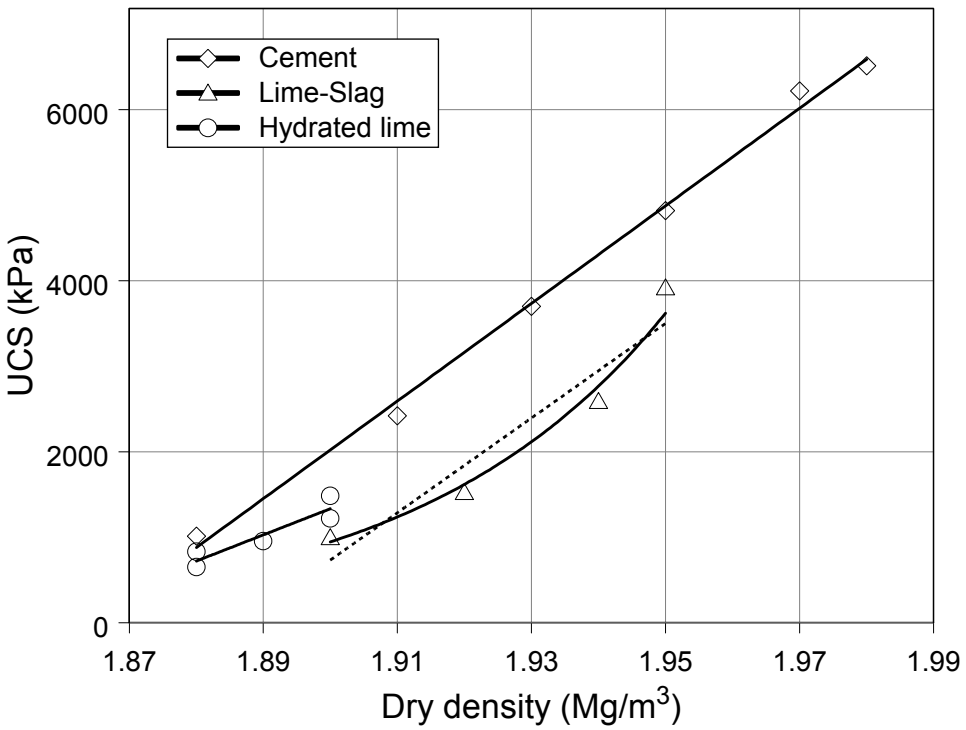


FIGURE 7.50 *UCS as a function of dry density for the PBL material. All specimens are compacted according to modified Proctor one hour after mixing. Curing period for all specimens was 7-days.*

Compared to field conditions, the samples were cured at constant temperature (20°C) without possibility for drying or wetting. This will mostly not be the case in field conditions. Changes in both temperature and water content will affect the strength.

7.2.2.2 Strength properties for Petersborg material

In the test series on Petersborg material a comparison is made between different binders. These specimens were compacted with the vibro compacting unit. The size of the specimens was 206 mm in height and 103 mm in diameter. They were compacted in PVC tubes to remain under horizontal stress and paraffin was used to prevent drying of the specimens.

The same binder recipes as in Figure 7.42 were used in preparation of the specimens for unconfined compressive strength tests. The specimen were cured at 7, 28 and 90 days, respectively. The 7-days' UCS is presented in Figure 7.51.

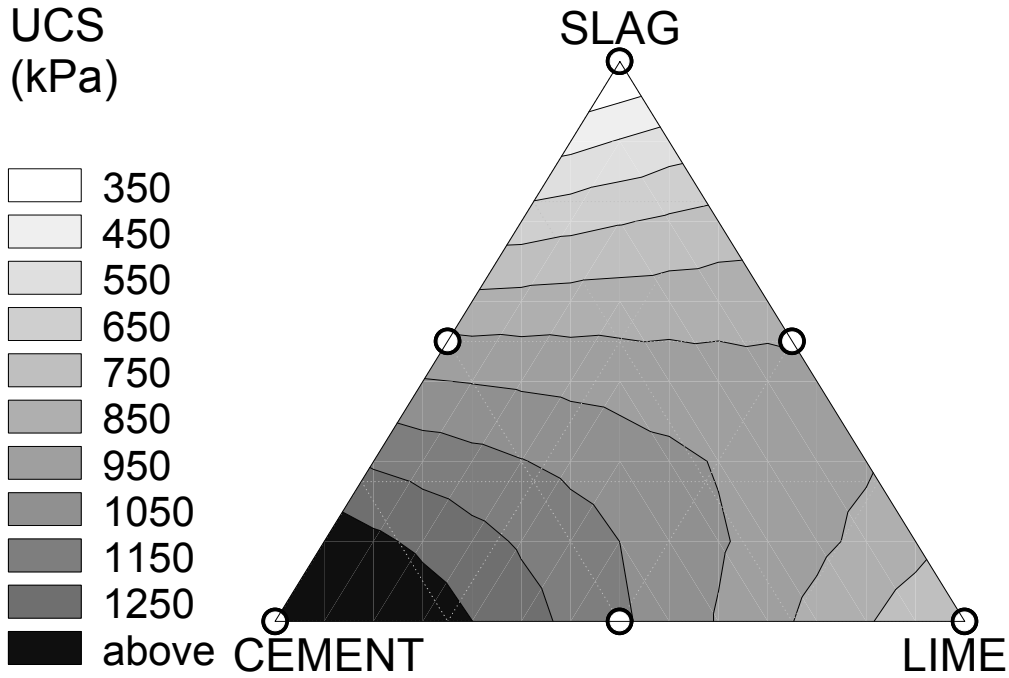


FIGURE 7.51 *UCS for the Petersborg material after 7-days of curing at 20^{0}C. All specimens were vibrator-compacted one hour after mixing.*
R^2=0.989, adj R^2=0.985.

Figure 7.51 clearly indicates interaction effects between lime and slag. Without interaction between lime and slag, the UCS for lime - slag should be the mean value of the UCS for lime and the UCS for slag. In this soil type, the mix of lime and slag is very beneficial for the development of strength. Cement gives the highest strength and slag the lowest.

The results after 28-days of curing are presented in Figure 7.52.

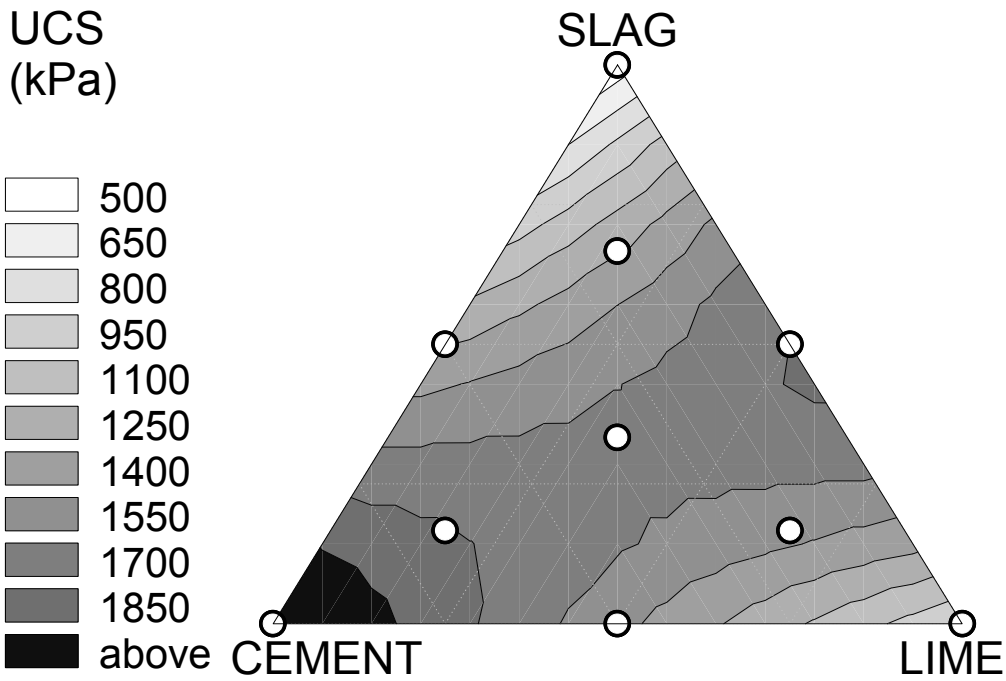


FIGURE 7.52 *UCS for Petersborg material after 28-days curing at 20^{0}C. All specimens were vibrator-compacted one hour after mixing.*
R^2=0.956, adj R^2=0.948.

Here, the lime-slag interaction is even more pronounced. In Figure 7.53, the results after a 90-day curing period are presented.

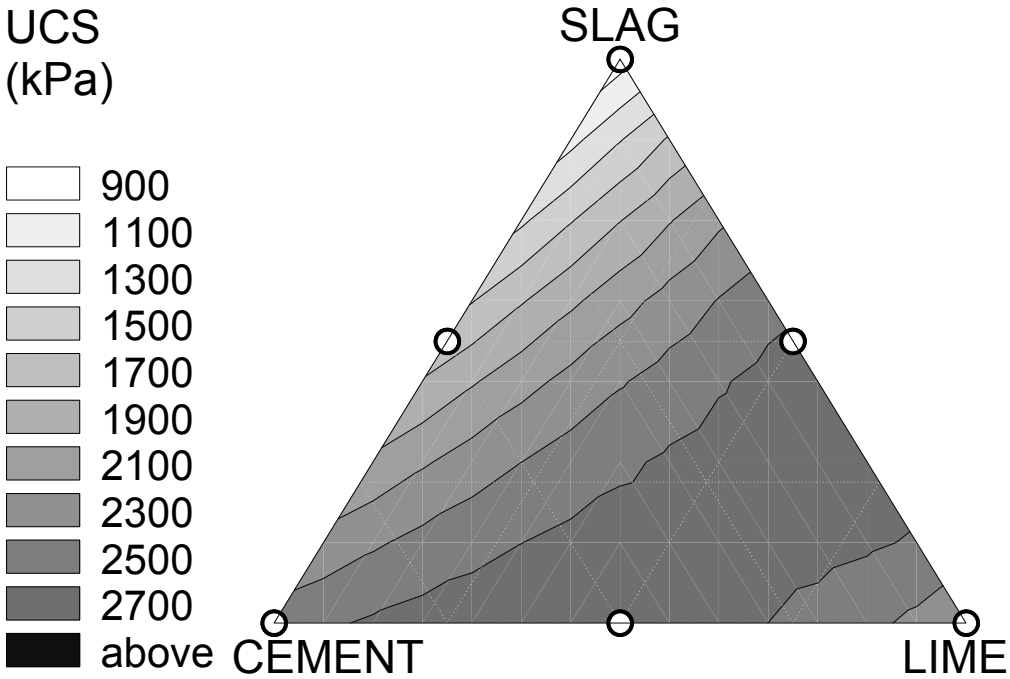


FIGURE 7.53 *UCS for Petersborg material after 90-days of curing at 20°C. All specimens were vibrator-compacted one hour after mixing.*
R^2=0.972, adj R^2=0.955.

After this curing period, most of the cement reactions have been developed but the pozzolanic reactions in the lime-stabilised soil are continuing. With respect to strength, the optimum binder recipe contains lime-slag, lime-cement or a combination of all three binders. However, if strength is combined with compaction characteristics (Figure 7.42) the optimum recipe contains only lime and slag. An optimum recipe should contain 50-60% lime and 40-50% slag. Another beneficial factor with this blended binder is that the working period is significantly longer than for any binder containing cement.

Indirect tensile tests were performed in connection to the test series presented in Figure 7.45. The object was to test how the delay time between mixing and compaction affected this type of strength. The result from the indirect tensile test is presented in Figure 7.54. For the cement-stabilised soil, there is a tensile-strength reduction of approximately 16% between one and five hours' compaction delay. In the same time the compaction effort has increased from MCV 12.5 to 15.2, cf. Figure 7.45.

For the blended binders that contain cement, no such reduction could be found. A possible explanation for this could be the dilution of the cement. For the cement-lime blend the different pattern compared to other binders remains. In this case it could be explained by the fact that the specimens delayed three hours had less compaction energy compared to the specimens with one and five hour delays. Slag has the greatest increase in MCV from one to three hours and a small increase in indirect tensile strength could be spotted in Figure 7.54.

When it comes to slag-lime blend the interaction between the binders is clearly visible. If no interaction was involved the tensile strength should be somewhere between the strength of lime and that of slag. The slag-lime blend outperforms the single binders by more than 100% and performs approximately like the lime-cement blend. However, lime-cement has a higher MCV compared to slag-lime and is thereby more difficult to compact. Lime was, as expected, fairly insensitive to a time delay between mixing and compaction.

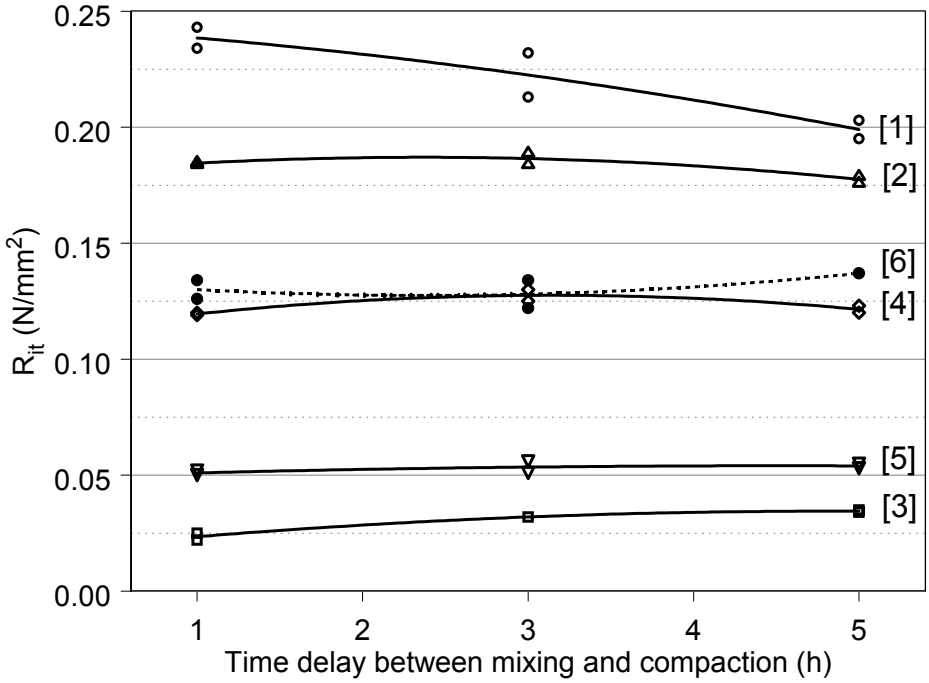


FIGURE 7.54 . *The indirect tensile strength (R_{it}) as a function of time delay between mixing and compaction for Petersborg material. Binder content 2.5%. [1] Cement, [2] 50% cement + 50% Slag, [3] Slag, [4] 50% Slag + 50% Lime, [5] Lime, [6] 50% Lime + 50% Cement.*

7.2.2.3 Strength properties for Örebro material

The test series on the Örebro material was designed to test how a slag binder without any activator would performed regarding strength. Sherwood (1993 and 1995) stated that slag needs to be activated with cement or lime to react. According to the results in Figure 7.51, Figure 7.52 and Figure 7.53, pure slag develops strength with time without any activator.

To check this, four different specimen batches were manufactured with nine replicates in each. The binders were 2% cement, 2% slag, 4% cement and 4% slag. The specimens were compacted in plastic tubes with a diameter of 50 mm. Each specimen was compacted with the "hand Proctor" with 25 blows in five layers. The samples were trimmed to a height of 100 mm, covered with paraffin, and stored within the plastic tubes. The curing period was 90-days for all samples. After the curing period, the samples were tested for unconfined compressive strength. The null hypothesis was chosen, assuming that no difference between 2% slag and 4% slag would be found. The cement-stabilised series were used as control samples. T-tests for independent samples were performed to reject this hypothesis at a p-level of 5%. The null hypothesis was rejected.

Figure 7.55 shows that with 2% binder content the slag-stabilised soil has a lower strength compared to cement-stabilised soil. However, with 4% binder content the slag-stabilised soil reached a higher strength compared to cement-stabilised soil.

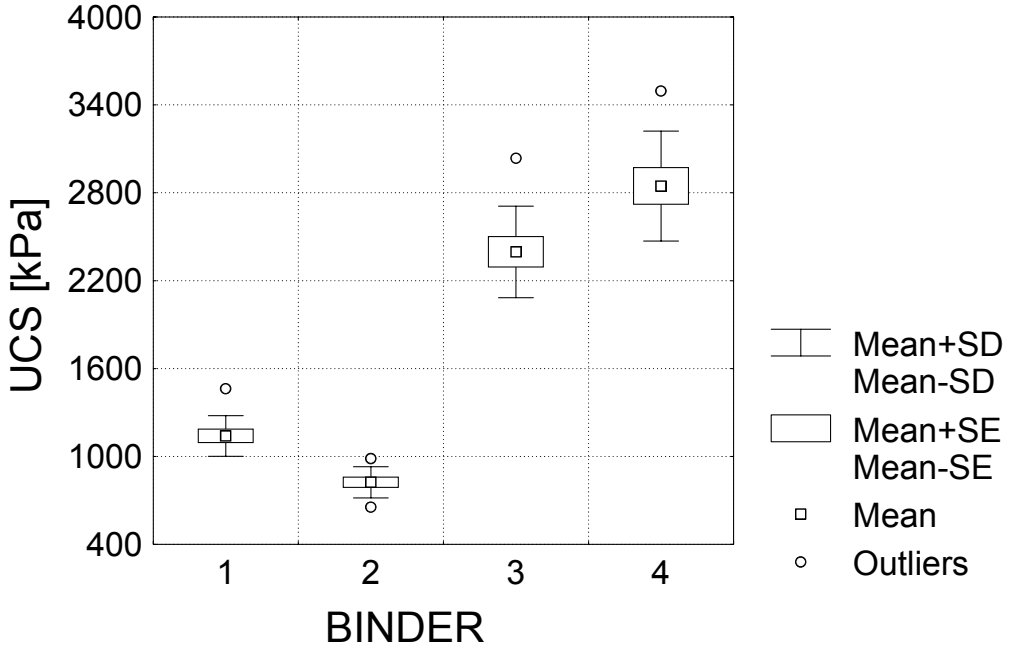


FIGURE 7.55 *Unconfined compression strength for Örebro material stabilised with: [1] 2% Cement, [2] 2% Slag, [3] 4% Cement, [4] 4% Slag. Curing period for all specimens was 90-days at 20°C. SD = standard deviation, SE = standard error.*

7.2.2.4 Strength properties for Hyllie material

The Hyllie material is a sandy silty clay till and this type of soil responds very well to lime treatment. The test series for this soil was designed to study the undrained shear strength after 2, 7, 14 and 28 days' curing. Both the unsoaked and the soaked undrained shear strength was tested. The specimens were stabilized with 2% hydrated lime and

compacted with MCA one hour after mixing. After compaction the specimens were removed from the mould and covered with paraffin.

The soaked specimens were cured in the same way as the unsoaked until 3 days before testing. Then the paraffin was removed from the lower part of the specimen, which was then immersed in water. The specimens were allowed to soak water for 3 days whereafter the strength was tested.

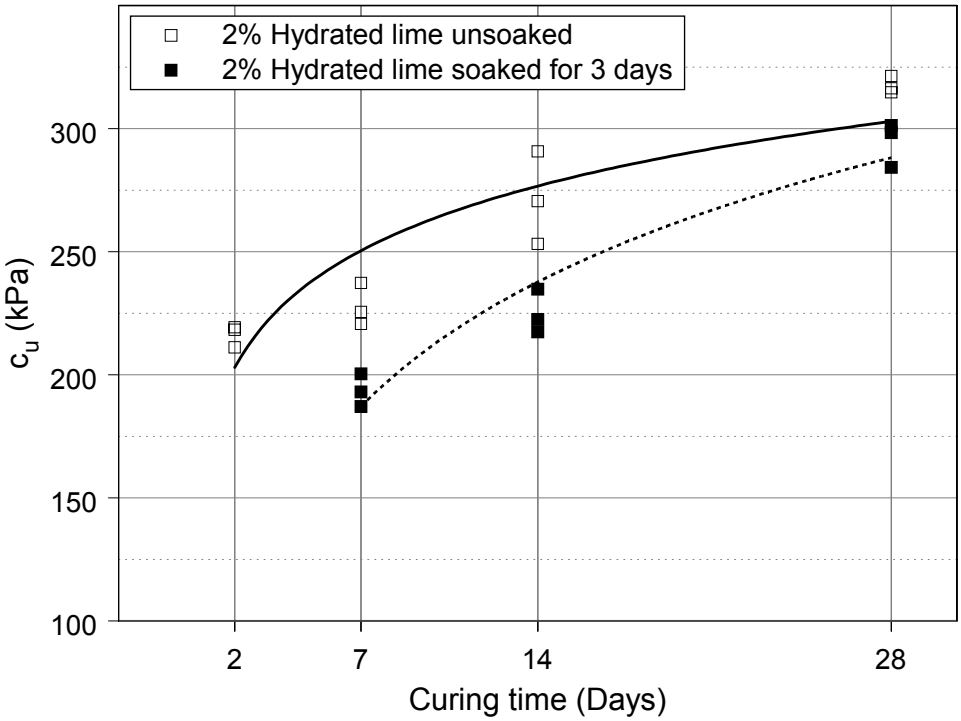


FIGURE 7.56 *Undrained shear strength as a function of curing time for unsoaked and soaked specimens.*

The result shows a strength increase from approximately 200 kPa after two days of curing to approximately 300 kPa after 28 days of curing, see Figure 7.56. The soaked specimens show lower undrained shear strength compared to the unsoaked specimens. However, for the specimens cured 28-days the difference in undrained shear strength has decreased compared to specimens with shorter curing time.

7.2.2.5 Strength properties for E22FN material

This test was performed to verify the MCV and cu relation for cement modified soil. The six specimens were made of soil with natural water content mixed with 2% cement and MCA compacted one hour after mixing. The sub specimens were put together to form three specimens for UCS. The MCV increased from approximately 9 to approximately 17.

The results from the evaluated shear strength are presented in Figure 7.57.

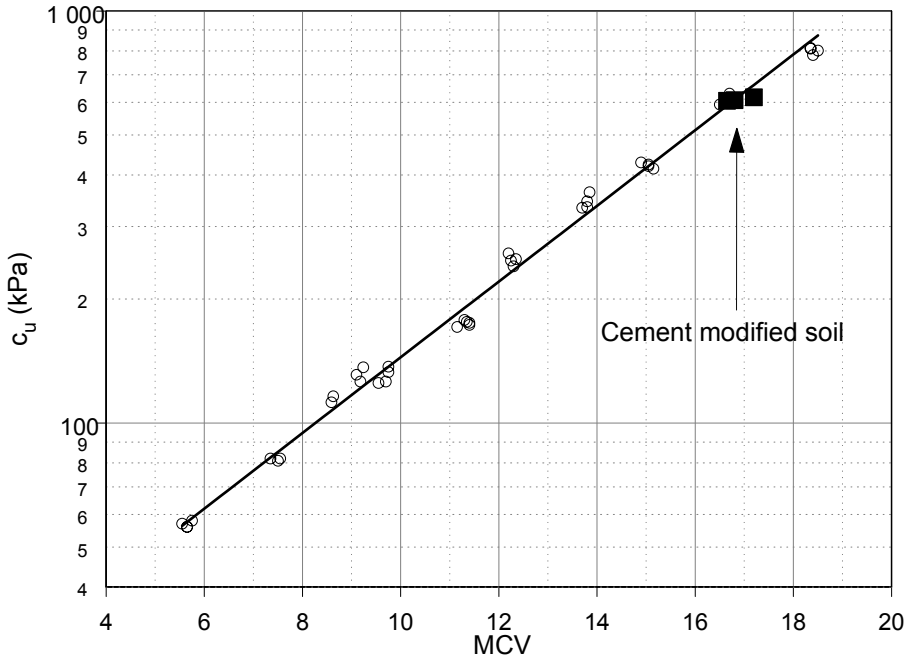


FIGURE 7.57 *Shear strength as a function of MCV for unstabilised and cement modified specimens. The cement content was 2%.*

The results fit very well the MCV - c_u relation for unstabilised soil. This shows that the MCV - c_u relation for an unstabilised soil can be used to evaluate the immediate undrained shear strength of a stabilised soil through MCV tests.

This is the second phase in the test described in Section 7.2.1.4 , page 247. The significant effects on UCS are presented in Figure 7.58.

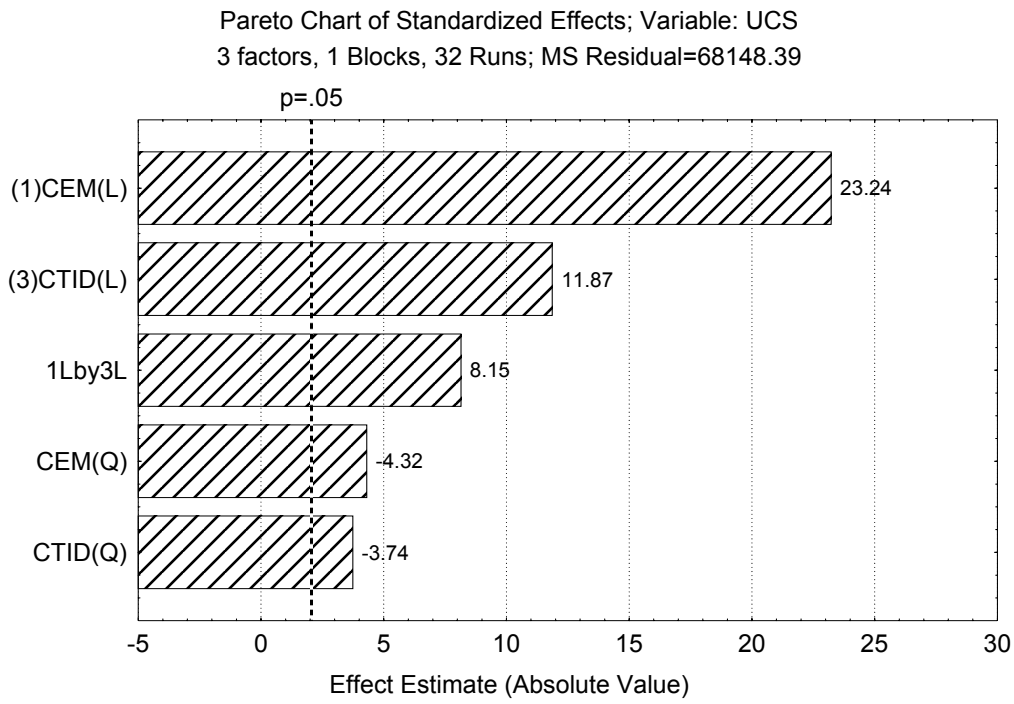


FIGURE 7.58 *Pareto chart showing the significant effects on UCS.*

Figure 7.58 shows that cement, curing time and their interaction has positive effect on UCS. It should be noted that the delay time between mixing and compaction does not affect the UCS, whereas it had a significant negative effect on the dry density, cf. Figure 7.47. This implies that a delay time less than or equal to five hours does not have a negative effect on UCS. Further it implies that the dry density is not directly related to UCS for a stabilised soil within certain limits. However, the decrease in dry density with increasing delay time could have a negative effect on parameters of the stabilised soil such as permeability and durability owing the increased void ratio. This limited test could not give

answers to these questions and a more detailed study has to be performed.

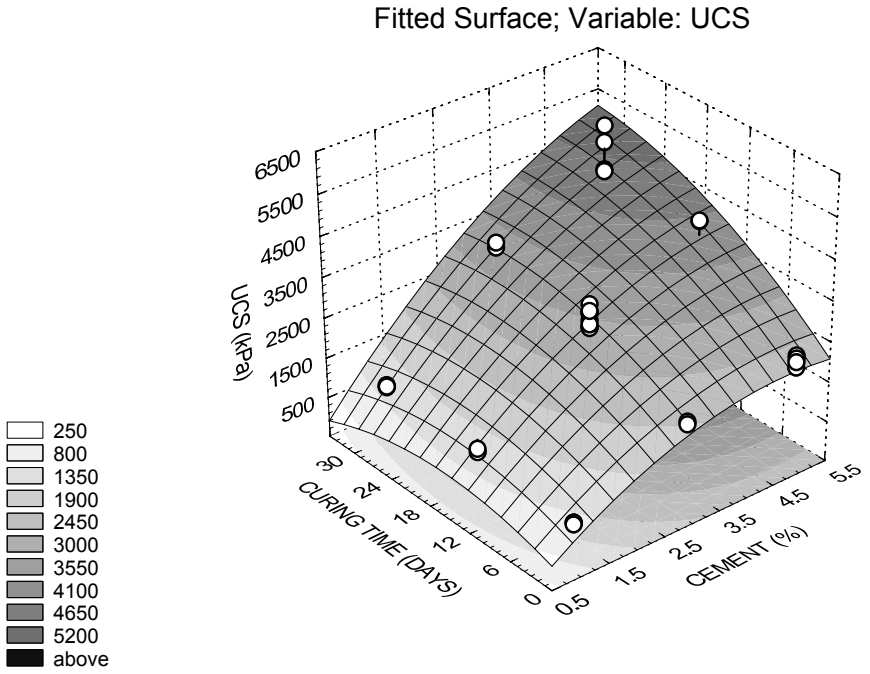


FIGURE 7.59 *Unconfined compression strength as a function of cement content and curing time between compaction and UCS test for the E22FN material.*

Figure 7.59 shows an increasing UCS with cement content for all tested curing times. The effect is more pronounced for a curing period of 29 days than for 1 day curing. Further, the figure shows that for low cement content (1%) the strength increase with curing time is marginal. However, for a cement content of 5% the strength increase with curing time is very pronounced.

7.2.2.6 GrindoSonic impact excitation

The natural frequency of slag-stabilised specimens was tested as a relative measure for the purpose of comparative evaluations. The test arrangement was according to Figure 6.24. The result is presented in Figure 7.60. The natural frequency was determined with 150 observations for each binder type. Compared to the UCS values presented in Figure 7.55, the result in Figure 7.60 is not satisfactory. However, the overall trend for the mean values agrees approximately with the pattern in Figure 7.55.

A possible explanation for the great variation in the frequency values could be found in the shape of the specimens and the direction of the measurement. The samples had a dimension of 50 mm in diameter and 100mm in length and this is close to the minimum relationship between diameter and length. The axial mode was not measured, although the piezo-electric vibration detector registered radial vibration, cf. Figure 6.24.

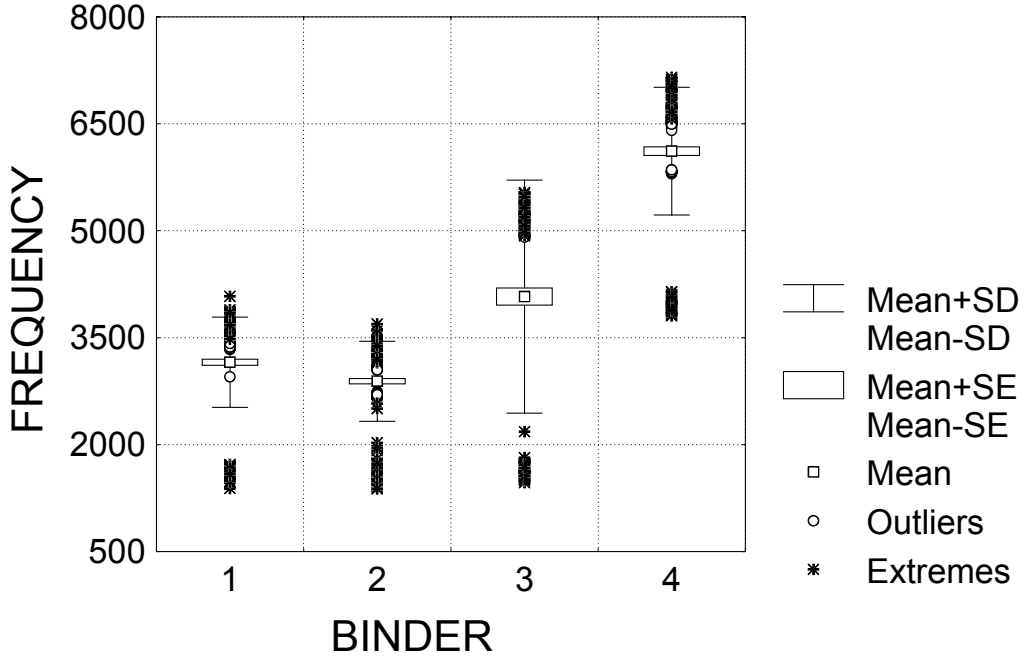


FIGURE 7.60 *Natural frequency for soil type C stabilised with: [1] 2% Cement, [2] 2% Slag [3] 4% Cement, [4] 4% Slag. Curing period for all specimens was 90-days at 20oC.*

7.2.3 The effect of organic agents

In cement chemistry, lignosulphonates are used as water reducers. A negative effect of using lignosulphonates in a concrete is the delay in hardening of the cement. This delay depends mainly on the sucrose content of the lignosulphonates (Taylor, 1997). However, this effect is very useful when it comes to soil stabilisation because of the increased working period for a cement binder. Another beneficial effect is that lignosulphonates reduce the remoulded shear strength of a fine-grained

soil, see Figure 7.61. The reduction of the remoulded shear strength increases the stabilised soil's workability.

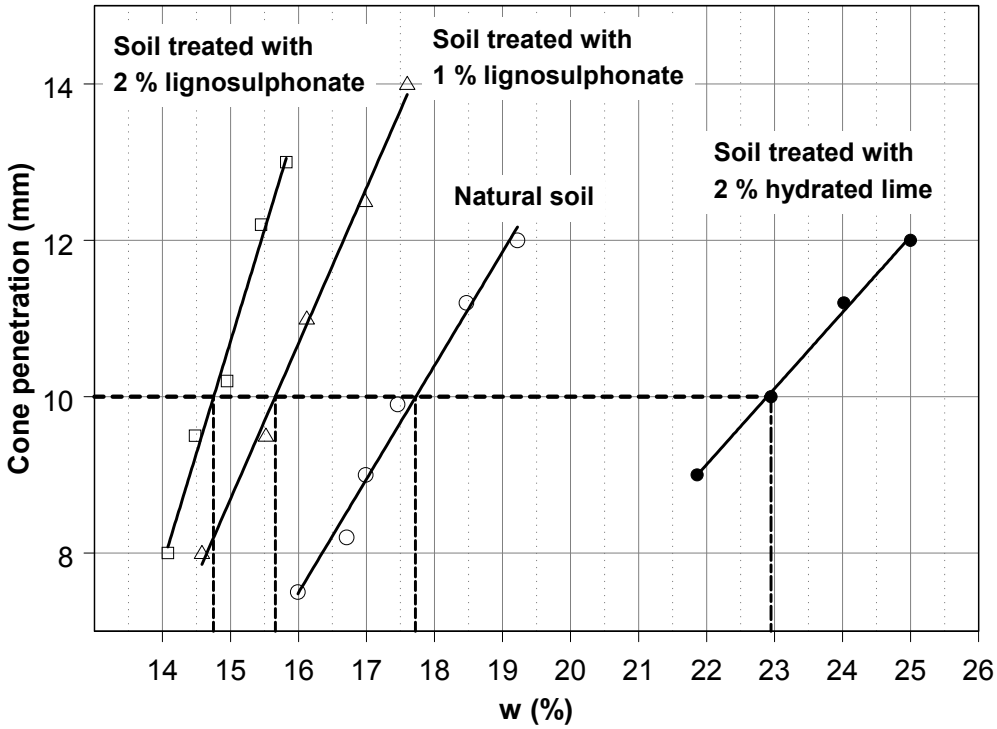


FIGURE 7.61 *Results from fall-cone tests on remoulded E22FN soil, both natural and lignosulphonate treated. The amount of additive was 1 and 2% based on the soils dry mass (Lindh, 2002).*

Soil treatment with 1% lignosulphonate reduces the liquid limit by 2%. It correspodingly reduces the remoulded shear strength at unaltered water content. The effect of 2% lignosulphonate reduced the liquid limit by 3%. The effect of 2% hydrated lime increase the liquid limit by more than 5%.

The effect on the binder's working period was tested with a Central Composite Design (CCD) with three independent factors; delay time, cement and lignosulphonate. The delay time was defined as the time between mixing the cement with the soil and compaction of the stabilised soil. The cement content was based on the dry weight of the soil. The lignosulphonate content is presented as per cent of the cement content since this is the normal way to report cement additives, see Figure 7.62.

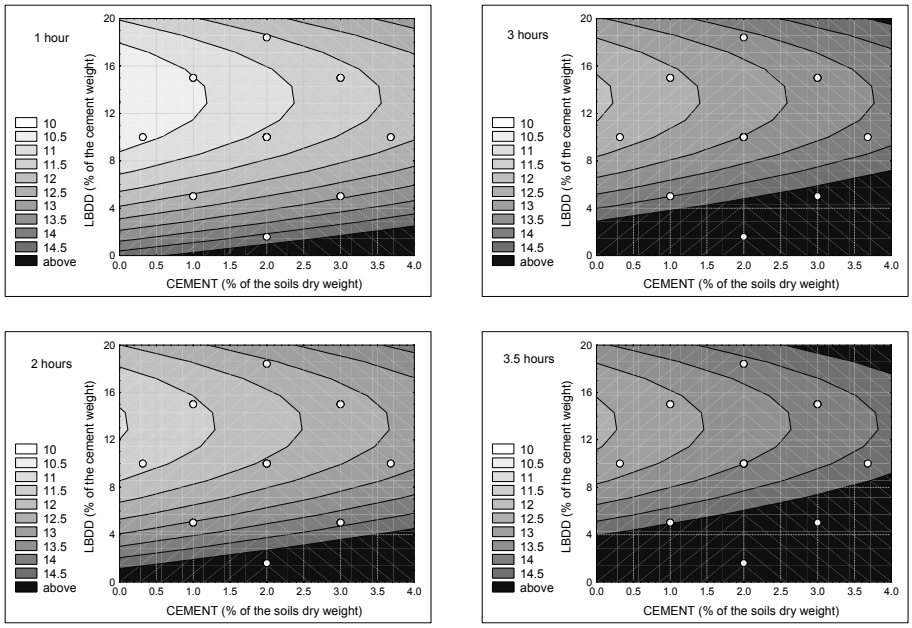


FIGURE 7.62 *MCV response surfaces presented as contour plots for 1, 2, 3 and 3.5 hours delay time. The cement content is based on the soil's dry weight and the lignosulphonate content, LBDD, is based on the dry weight of the cement. (Lindh, 2002).*

The lowest MCVs could be found for lignosulphonate content between 8 and 18% with a minimum around 13% based on the weight of the cement. The effect of using lignosulphonates in combination with a binder is very promising for a stabilised fine-grained soil. However, the

retarding effect from the tested lignosulphonate is not enough to ensure an appropriate working period and has to be increased. The results indicate that the effect of the lignosulphonates has a maximum effect between 8 to 18% based on the cement content. Increasing the lignosulphonate content is not a solution since the effect is reduced with increasing amount; instead another retarding agent could be used together with the lignosulphonate.

To evaluate the effect of lignosulphonate on lime-stabilised specimens, another test was designed. The specimens consisted of Hyllie material and were stabilized with 2% hydrated lime and 0.2% lignosulphonate. They were then compacted with MCA one hour after mixing. After compaction the specimens were removed from the mould and covered with paraffin. The soaked specimens were cured in the same way as the unsoaked until 3 days before testing. Then the paraffin was removed from the lower part of the specimen and this end was immersed in water. The specimens were allowed to soak in water for 3 days before the strength test, see Figure 7.63

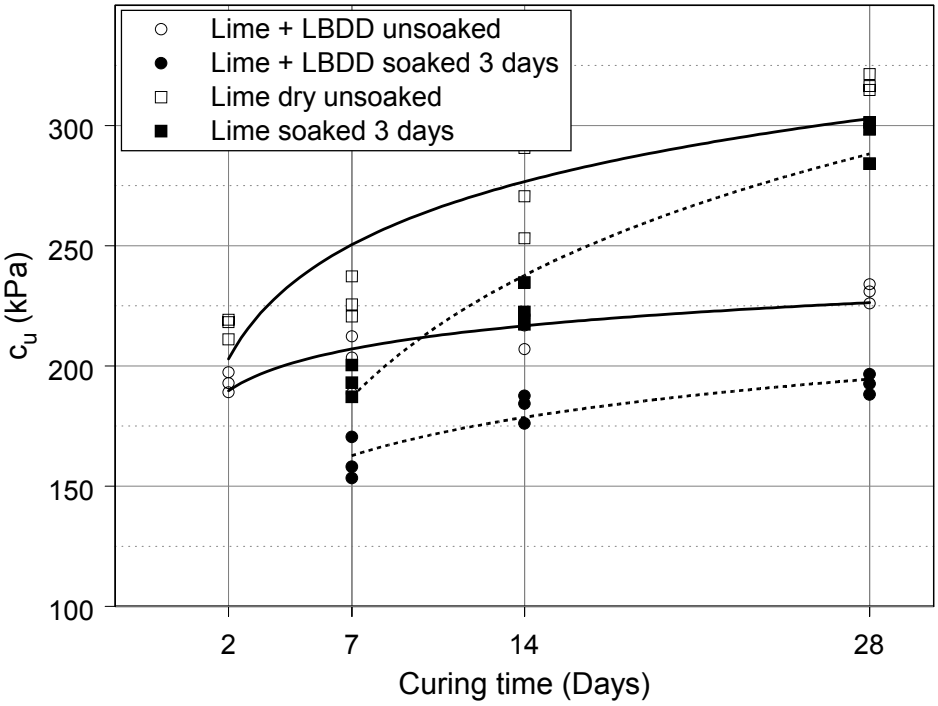


FIGURE 7.63 *Undrained shear strength for lignosulphonate (LBDD) treated and untreated specimens as a function of curing time for unsoaked and soaked specimens.*

Figure 7.63 shows the same pattern for untreated and lignosulphonate treated specimens. The lignosulphonate lowers the undrained shear strength for both soaked and unsoaked samples. However, the undrained shear strength at 7 days of curing is well above normal requirements (c_u = 100 kPa) and thereby acceptable.

7.2.4 Freeze and thaw tests

Two different types of binders were used in these tests: cement and a mixture of ground granulated blast furnace slag (GGBFS) and hydrated lime.

The frost-heave measurements presented in Figure 7.64 show a wide scatter for the specimens stabilised with a cement binder. Such scatter has previously been explained by the use of dynamic compaction equipment (McCabe and Kettle, 1995) or by varying initial water content (Brandl, 1999). In the results presented by McCabe and Kettle (1995), three distinct layers were visible in the specimens compacted by a vibrating hammer. However, the specimens produced in this test did not have any distinct layers or large variations in the initial water contents. The specimens stabilised with a lime-slag binder showed show only a very small frost heave and also a very small scatter between the specimens.

At present it is unclear if the scatter for the OPC binder is due to the testing unit or to the preparation of the specimens. A possible explanation could be a non-homogeneous structure in these specimens owing to the binder type, i.e. cement does not produce the same homogeneity compared to lime. This is intended to be examined in future test series.

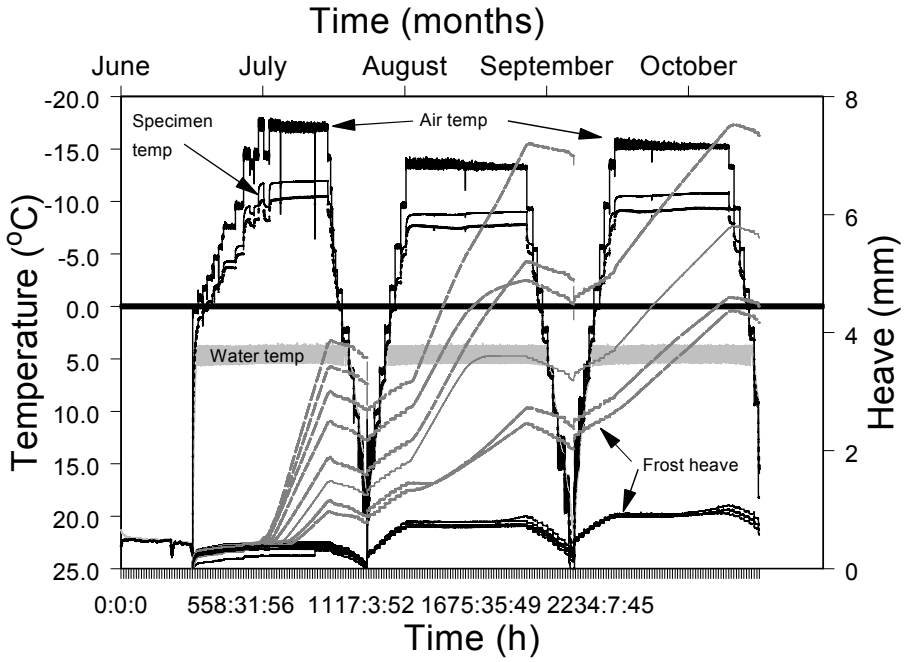


FIGURE 7.64 *Frost heave in test series on soils stabilised with two different types of binders. The specimens with high frost heave are stabilised with a cement binder and the specimens with low frost heave are stabilised with a lime-slag binder. Each binder type is represented with by eight specimens.*

The minimum air temperature was set to different values during the three freeze- and thaw cycles. The intention of this was that if large icelenses should develop, they should occur at different levels in the specimens in the three cycles, see Figure 7.64 and Figure 7.65. The photo of a specimen stabilised with cement in Figure 7.65 shows that only two ice lenses developed, even though frost heaving occrrued during all three cycles, see Figure 7.64. A probable explanation is that the differences in minimum air temperature were too small and that the icelense in the third freeze cycle developed in an existing crack. The different freeze cycles

shown in Figure 7.64 indicate that the frost-heave rate varies for the cement stabilised specimens both within the same cycle and between the cycles. The third cycle with only four cement specimens left, appears to have the most uniform heave rate. This might be a result of the selection of the specimens removed for strength testing. However it is also possible that the first two freeze cycles created more frost-heave, c.f. Viklander and Knutsson (1997).

In the second freeze cycle, the frost heave rate for some of the OPC stabilised specimens dropped before the temperature increase. No explanation for this phenomenon has been found.

FIGURE 7.65 *A soil specimen stabilised with a OPC binder after three freeze-thaw cycles.*

An example of the temperature variation in the test unit is presented in Figure 7.66. The measurements indicate that the PVC-tube conducts heat better than the soil specimen. This could be a result of cold air leaking in between the insulation and the PVC-tube. The thermocouple placed just below the silicone coating shows a higher temperature than that placed outside the PVC-tube and 10mm below the insulation surface. A possible explanation for the temperature difference between the

thermocouples on the top- and middle levels shown in Figure 7.66 is that specimen 4, see Figure 6.21, is placed in the corner of the testing unit.

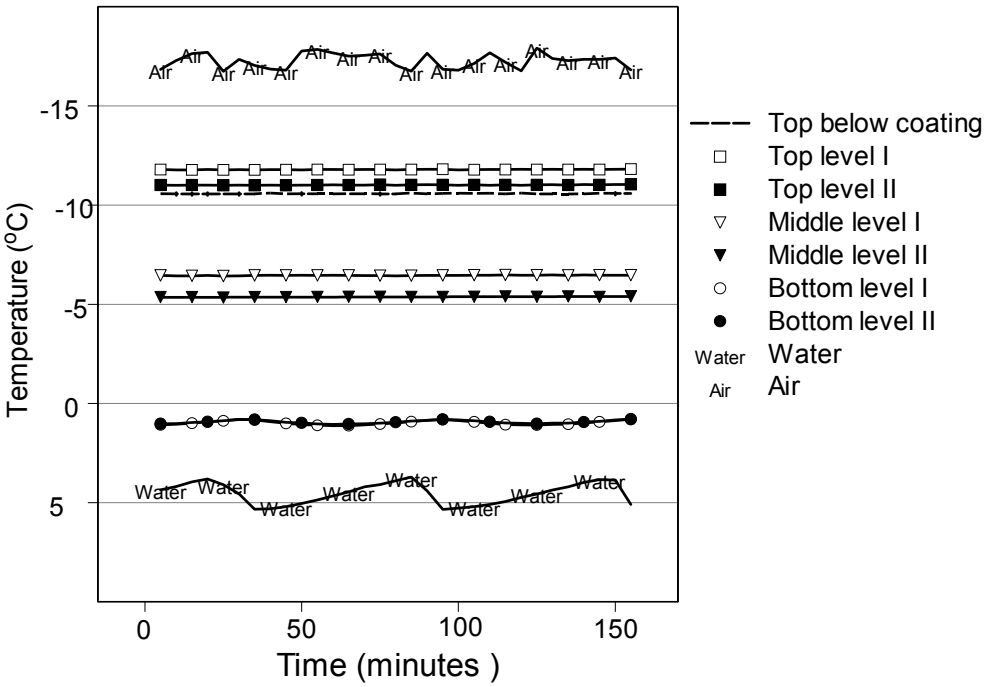


FIGURE 7.66 *Temperature variations in specimen 4, where I and II are measuring points separated by 180 degrees. For elevation placing see Figure 6.22. The diagram shows the results from specimen 4, c.f. Figure 6.21.*

Figure 7.67 shows the unconfined compressive strength (UCS) for specimens stabilised with cement and lime-slag binders after 1, 2 and 3 freeze and thaw cycles. The results indicate a certain reduction of the strength for specimens stabilised with cement, whereas any changes in the lime-slag stabilised specimens are more uncertain. This is in line with the observations that large frost heave occurred only in the cementstabilised-specimens. However, all stabilised specimens performed well, also after three freeze- and thaw cycles.

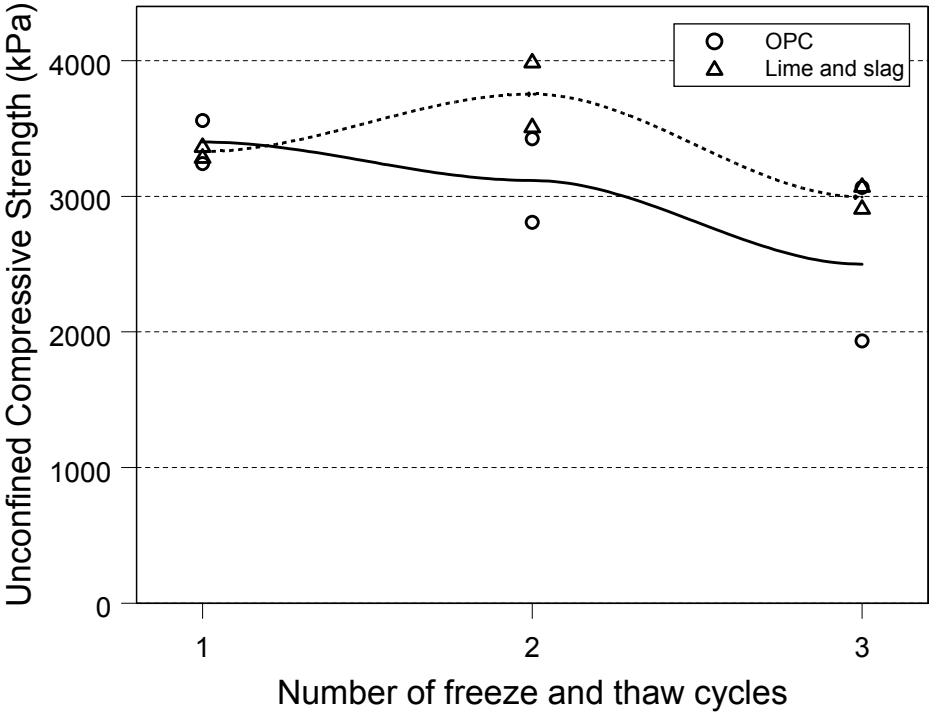


FIGURE 7.67 *Unconfined compressive strength (UCS) for the tested specimens after 1, 2 and 3 freeze- and thaw cycles.*

Figure 7.68 shows ice-lenses developed in a compacted un-stabilised specimen. The photo shows ice-lenses in layers with frozen soil. This should be compared toFigure 7.65 where the ice-lenses have developed in single layers. A possible explanation for this is the increased strength in a stabilised soil compared to an un-stabilised soil. Another explanation could be the difference in soil structure between stabilised and un-stabilised soil.

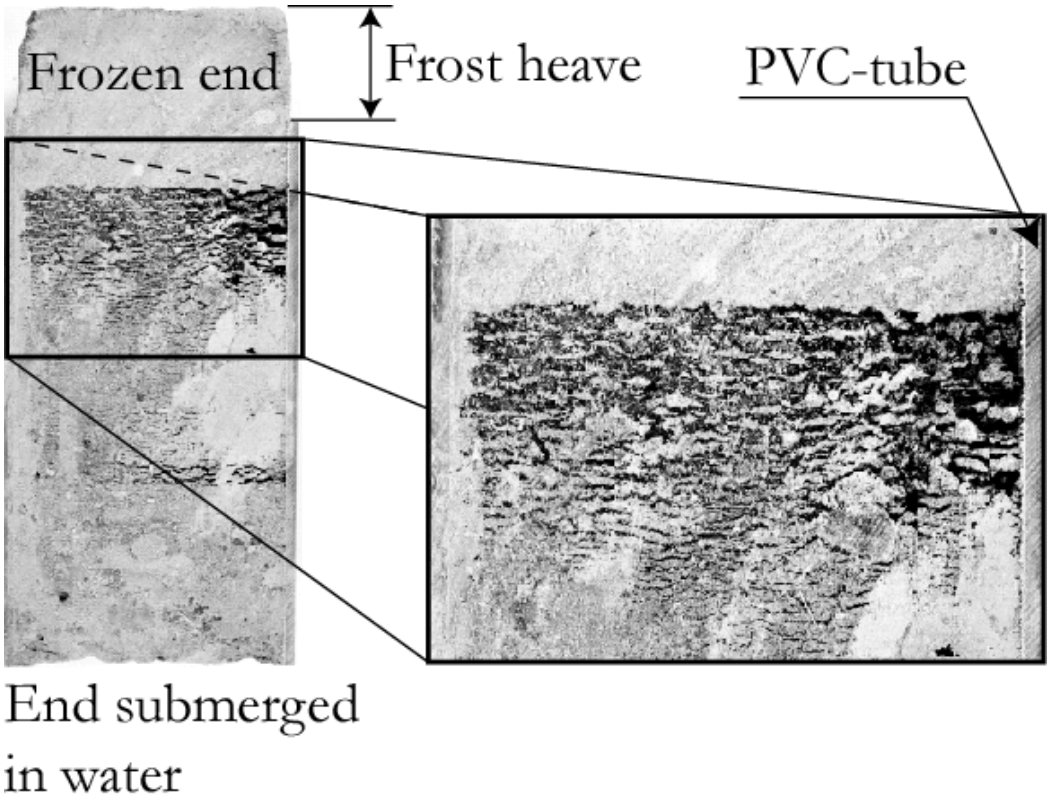


FIGURE 7.68 *Photo showing ice-lences in an unstabilised specimen. The specimen consists of Petersborg material. Photo S-I. Granemark.*

7.3 Field tests

Two different field tests were performed in this study. The first of these tests was performed at Yttre Ringvägen a ring road around Malmö city. The second field test was performed at Malmö-Sturup Airport.

7.3.1 Test site Ollebo

7.3.1.1 Backround

In 1996 the construction of Yttre Ringvägen, a ring road around Malmö City, was started. The soil in the area consists of clay till and silty till. This type of soil is very sensitive to variation in water content. At an early stage it was discovered that the bearing capacity of the embankments was too low to meet the requirement in ROAD 94, which is the general technical construction specification for roads from the Swedish National Road Administration (SNRA). To fulfil the requirements in ROAD 94, either soil stabilisation or soil replacement had to be used.

According to ROAD 94, the bearing capacity can be verified in two different ways, both with an inspection area ≤ 4,500 m^2. In the first alternative, eight random samples of the inspection area are chosen and a static plate load test is performed at each test point. The modulus of elasticity (E_{v2}) has to be at least 25 MPa. The second way to verify the bearing capacity is to use a roller-mounted compaction meter and use the obtained information to choose the two areas with the lowest response, see Figure 7.69. In these low response areas, static plate load tests are

performed. The requirement is a mean modulus of elasticity $E_{v2} \geq 10$ MPa for those two points (Anon., 2003b).

7.3.1.2 Test program

The site at Ollebo was an area of twenty by fifteen metres being a part of the ring road. This area was an embankment with a height of two metres and it was divided in four squares with a test point at the centre of each, see Figure 7.69. The soil is clay till with a clay content of approximately 15% and a natural water content of around 13%.

The final surface compaction control was performed with a roller-mounted compaction meter in combination with static plate load tests. Compaction meter values (CMV) obtained at the test area are presented in Figure 7.69. The surface is relatively homogenous with respect to the CMV. Static plate load tests were performed at the test points and light drop weight tests were also performed.

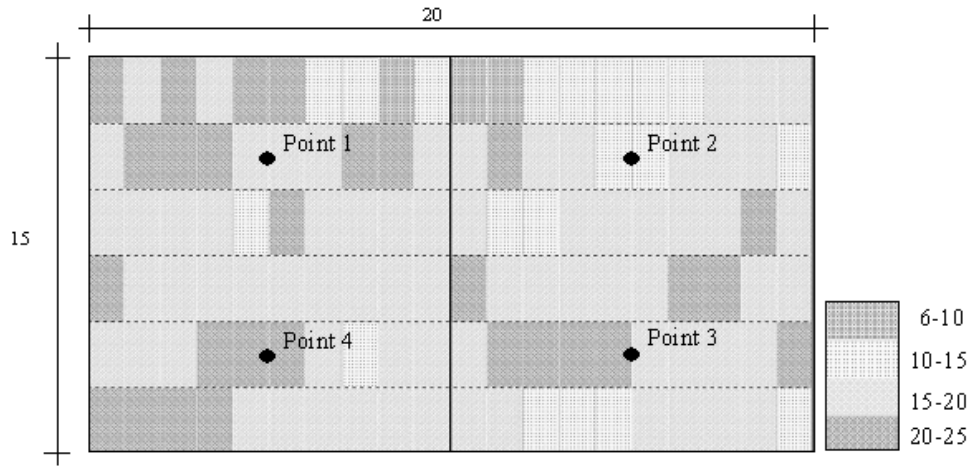


FIGURE 7.69 *A sketch of the test site with the four test points and the distribution of CM-values over the area. (Lindh et al., 2000).*

Results from the static plate load tests and the light drop weight tester are presented in Figure 7.70. The values from the light drop weight tester are converted from a dynamic to a static modulus. This conversion is based on unpublished results from the Swedish Road and Transport Research Institute (VTI). The results in Figure 7.70 show that static plate load tests and light drop weight test gave similar results on the unstabilised soil.

On the stabilised soil, however, the LDWT gives lower values for three out of four points. The results from CMV shown in Figure 7.69, indicate that the stabilised soil responds rather uniformly over the test area. In Figure 7.69, point 2 shows the lowest stiffness, which is contrary to the results in Figure 7.70 where point 2 gives the highest value. The contradiction could be explained by the difference in influence depth of

the compaction meter and the other two methods. The CM value depends on the weight of the roller and in this case takes a greater depth than the stabilised layer into account.

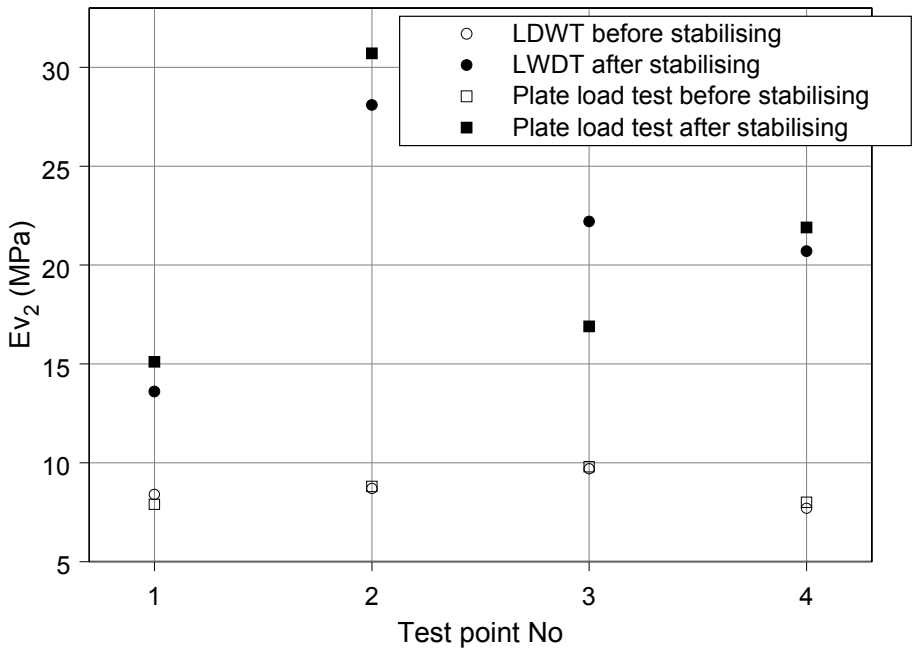


FIGURE 7.70 *Modulus of elasticity from static plate load test (SPLT) and light drop weight tester (LDWT) on stabilised and unstabilised soil.*

A core sampler was used to determine the in-situ density and void ratio. For this purpose a core sampler according to British Standard (BS 1924:1990) was manufactured. In earlier projects, nuclear methods have shown to give unsatisfactory results. The same conclusions were drawn in England by Sherwood (1993). In Figure 7.71 the dry density determined on the core samples is compared to dry density determined on MCV specimens. The core sample results show lower dry density and

higher water content compared to the MCV results. A possible reason for the lower dry density in the field could be that the embankment below the stabilised layer had too high a water content to respond adequately to the compaction.

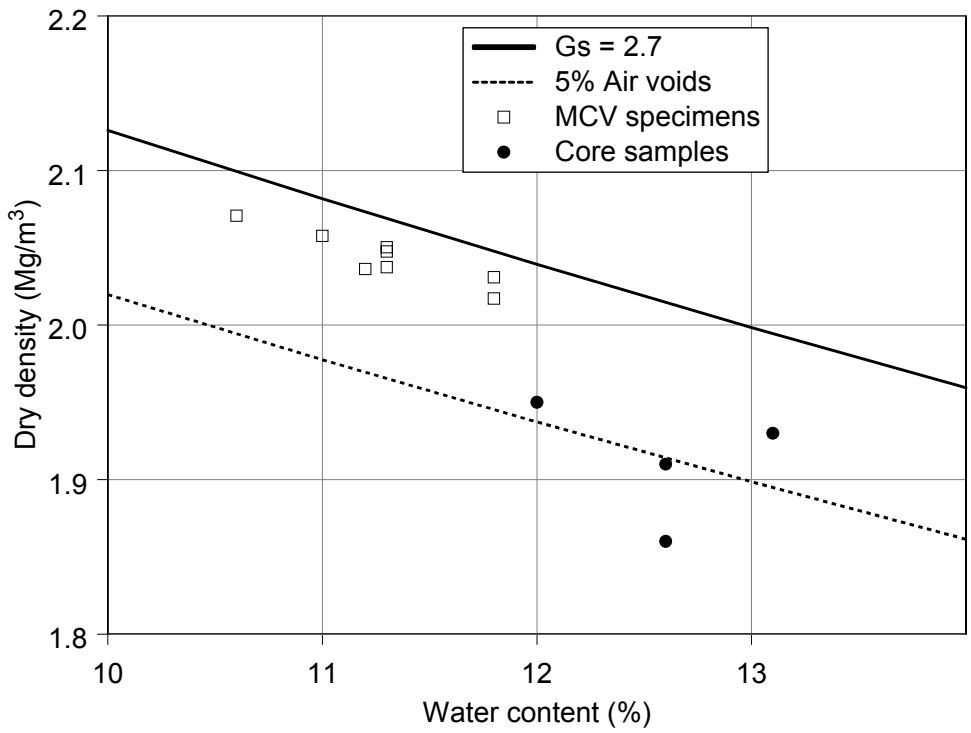


FIGURE 7.71 *Comparison between dry density and water content in the core samples and the MCV specimens.*

Results from the pulverisation tests are presented in Figure 7.72. From this plot it is clear that the P value at the site is > 30% and that the soil fulfils the requirements in this respect.

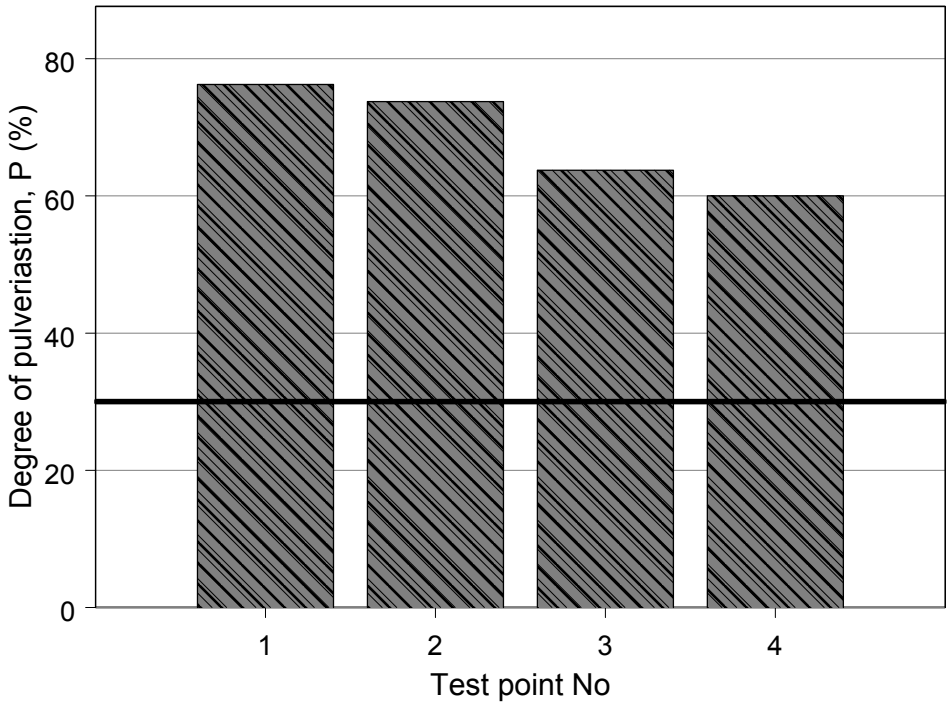


FIGURE 7.72 *Degree of pulverisation at the different test points.The test criterion is P > 30%.*

Initial consumption of lime (ICL) tests were performed on the soil from Ollebo, see Figure 7.73. The result shows that the lime content needs to be 2% of the soil's dry weight to reach a pH of 12.4.

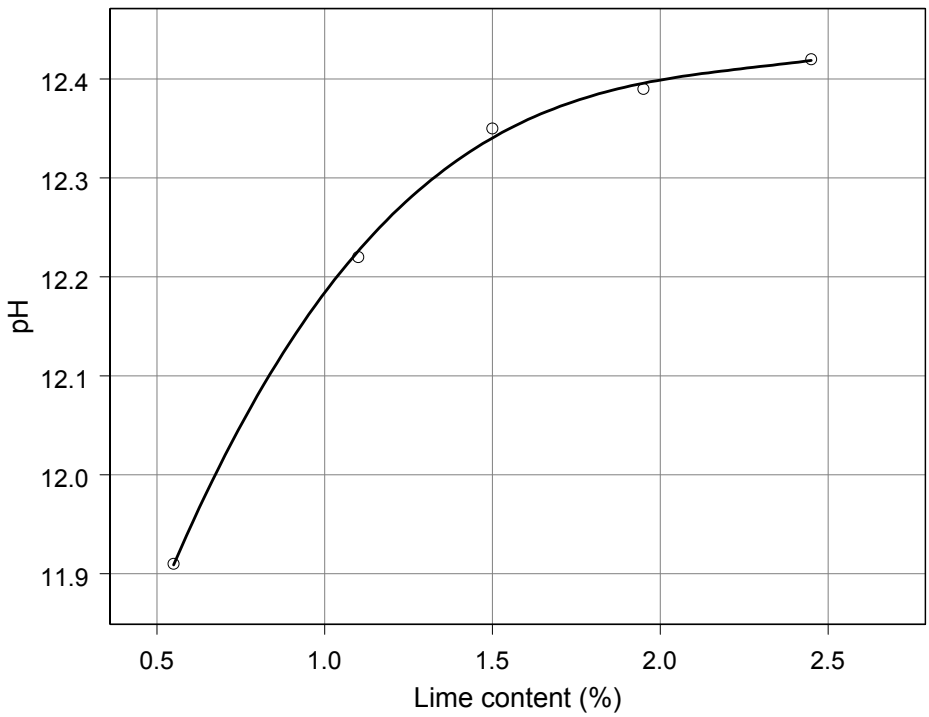


FIGURE 7.73 *Results from an ICL test on the Ollebo soil.*

Four electrode cables with 21 take-outs each were laid out on a line using an electrode separation of 0.25 metres, see Figure 7.74 .

FIGURE 7.74 *Photo showing resistivity equipment and a light drop weight tester (LDWT). Photo Per Löwhagen.*

The resistivity results before and after stabilisation are presented in Figure 7.75. There is a clear difference in resistivity between unstabilised and stabilised soil. Boardman *et al.* (2001) have reported a change in conductivity caused by blending a soil with lime. The differences in resistivity before and after stabilisation depend on several effects. These

effects are change in water content, change in porosity and the change in ion content. In this study, there is no possibility to separate these effects.

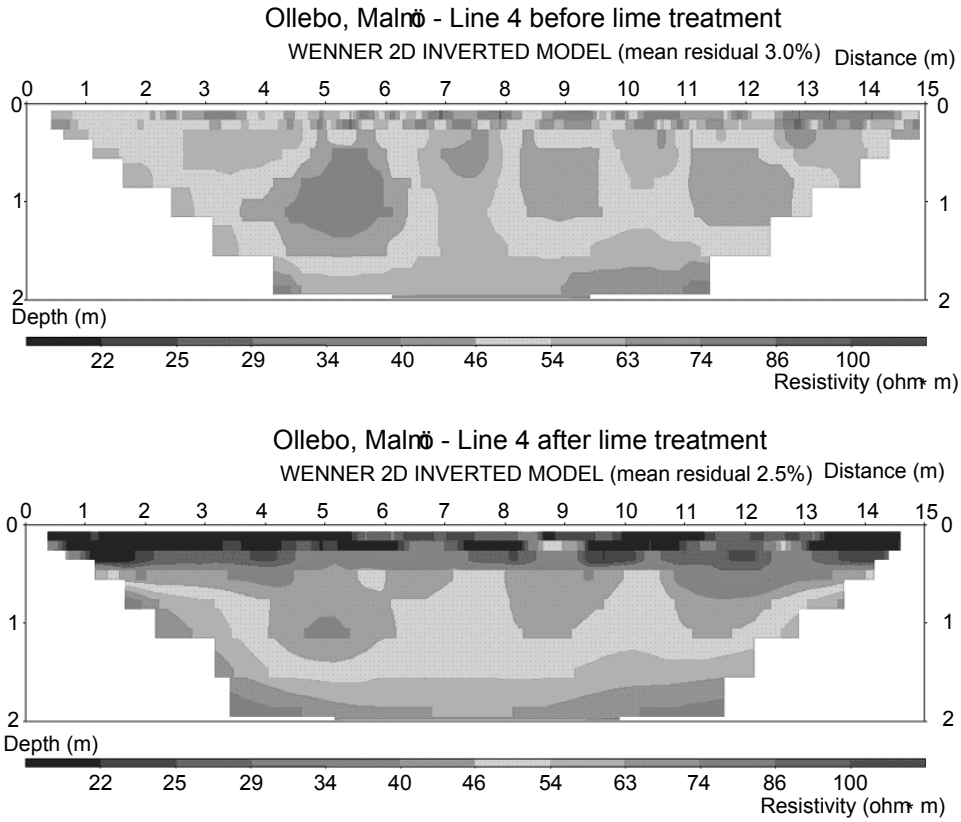


FIGURE 7.75 *Resistivity profiles line 4 at the Ollebo test site before and after lime treatment (Lindh et al., 2000).*

For the homogeneity tests a more flexible method is requested, and the study shows that the resistivity method has a large potential as a complement to traditional testing. The technique could also give a 3-D model of the stabilised area. However, there is a need for faster data acquisition, and this could be solved, for example, with a moving multi-

electrode system that could be pulled after a vehicle. There is also need of a more efficient data-processing used in routine applications.

7.3.2 Test site Sturup

7.3.2.1 Background

In order to extend the capacity at Malmö/Sturup Airport, a new concrete slab for parked aeroplanes had to be constructed. In this area, there was a large volume of peat that had to be replaced. The excavation was at most 12 metres deep and it included of 37,000 m^3 of peat, see Figure 7.76.

FIGURE 7.76 *Photo showing the test site at Malmö/Sturup airport.*

The planned fill material consisted of 80,000 cubic meter of fine-grained till. During test pit excavations the fine-grained till was discovered that the natural water content that was too high to allow the material to be used in its natural state. The moisture condition value (MCV) of the natural soil was between 3 and 5 and it should be at least 7 to meet the requirements.

The solution could be either drying the till to an acceptable water content or totally replacing the till with imported gravel. A third solution, soil modification, came up as a good environmental solution. To evaluate the potential of a soil modification, a laboratory investigation was performed with the main purpose of verifying that the required MCV could be reached.

7.3.2.2 Test program

The testing program was designed to evaluate the effect of soil modification using cement and cement/lime. The modification effect was evaluated by comparing the modified soil with the natural soil partly dried to different water contents. The tests was also to determine the required amount of binder and possible effect of a delay time between mixing and compaction.

The main subject for the pre testing was the soil compaction properties. These were determined according to the Moisture Condition Value (MCV) method and unconfined compressive tests. In the field, vane tests were used. Two different soil/binder mixes were tested apart from the natural soil. The binders used in those mixes were cement and lime/cement (50:50).

The modification effect of 2% cement is shown in Figure 7.77. The untreated soil had water content of approximately 14.5% and after modification approximately 13.2%. According to the MCV calibration line, a specimen with a water content of 13.2% should give a MCV of approximately 5.4. However, owing the modification process the measured MCV was 10. For an untreated soil this implies a water content of approximately 11%. This shows that the soil appear to be dryer than it is.

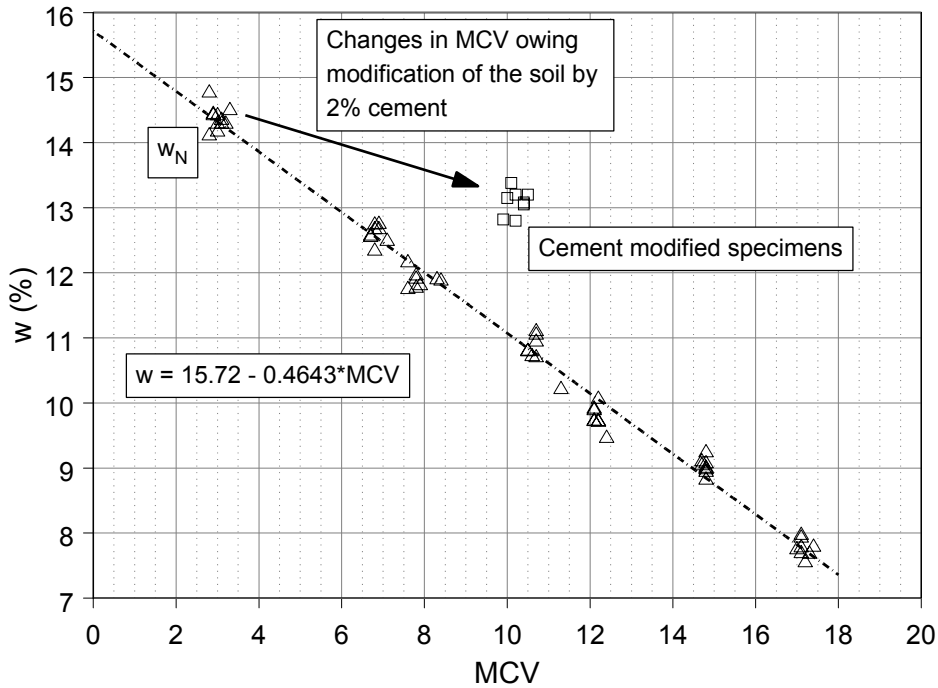


FIGURE 7.77 *MCV calibration line for the Sturup material. The delay time between mixing and compaction was 1 hour for the modified specimens. (Lindh et al., 2001).*

Figure 7.77 showed that the Sturup soil responded well in cement modification. To further study the modification effect more specimens were compacted in the MCA and strength tested, cf. Figure 7.78.

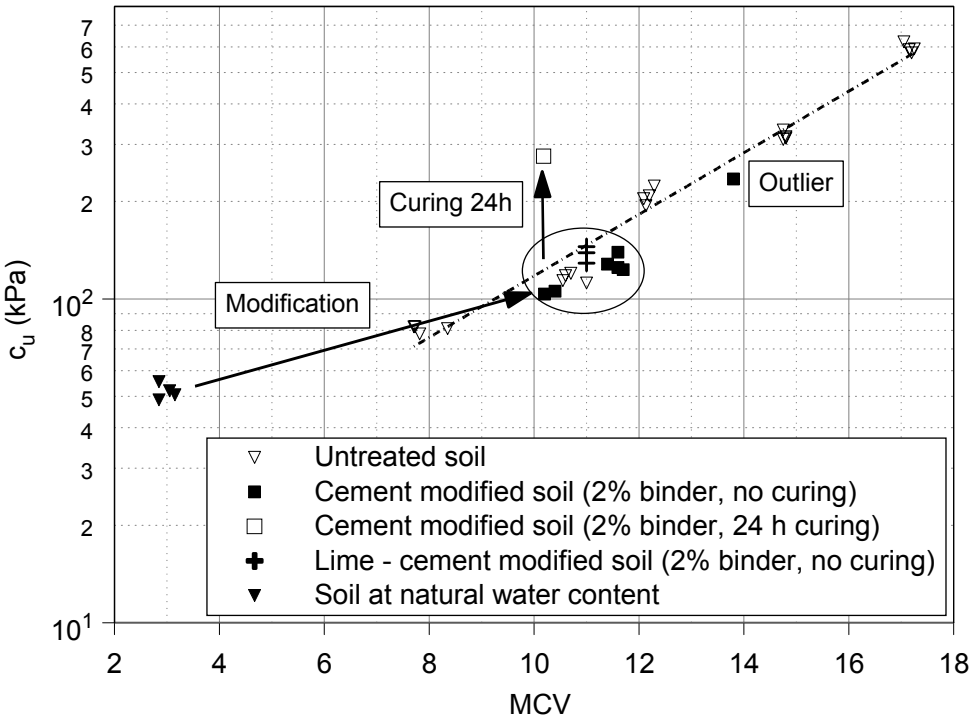


FIGURE 7.78 *Shear strength as a function of MCV for natural and modified soil. All modified specimens were compacted within one hour from mixing soil and binder. The shear strength was determined directly after compaction for the uncured specimens.*

The results show that MCV - c_u relation for an untreated soil could be applied to the modified Sturup material, cf. E22FN material in

Figure 7.57. However, if the soil was allowed to cure for 24 hours there was a strength increase by nearly three times compared to the immediate strength.

The field-testing were performed at two test areas. In the first test area the binder were mixed with the soil before excavation and in the second test area the binder were mixed with the soil in the embankment.

The fill in the first test area was placed with a dozer and compacted with a smooth drum roller. The vane test results for the first test area are presented in Figure 7.79.

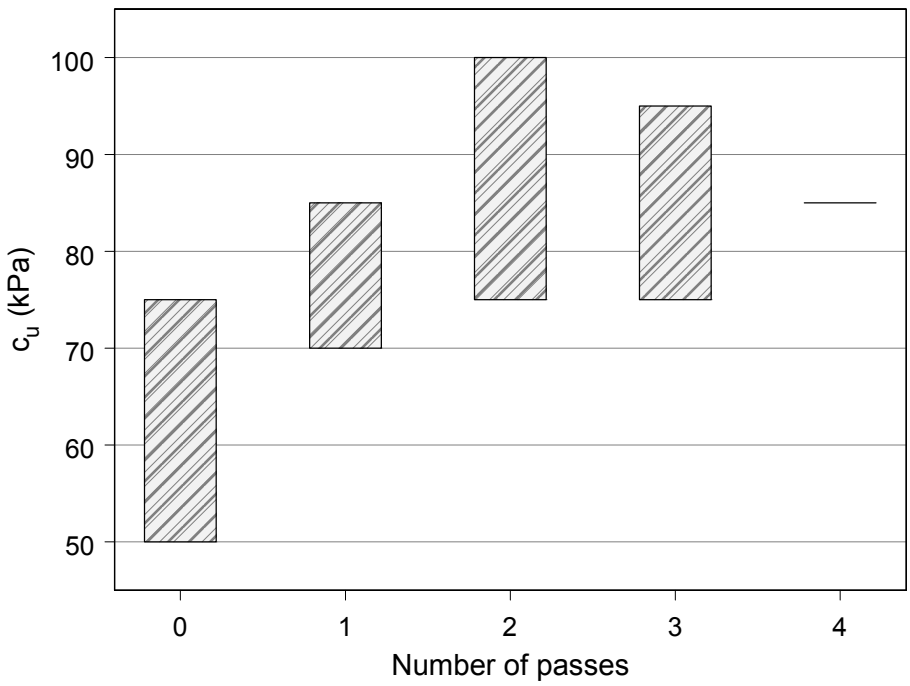


FIGURE 7.79 *Undrained shear strength for test area one as a function of roller passes.*

The graph shows an initial undrained shear strength of 50 to 75 kPa in the fill. After compaction with 4 passes of the smooth drum roller the undrained shear strength was approximately 85 kPa. After a curing period of 16 hours the undrained shear strength varied from 130 to 200 kPa. During the test period the air and soil temperature varied from -1 to 3^0C.

For test area two that was mixed on place shows the same pattern as the material in the first test area. However, the undrained shear strength was significant higher, see Figure 7.80.

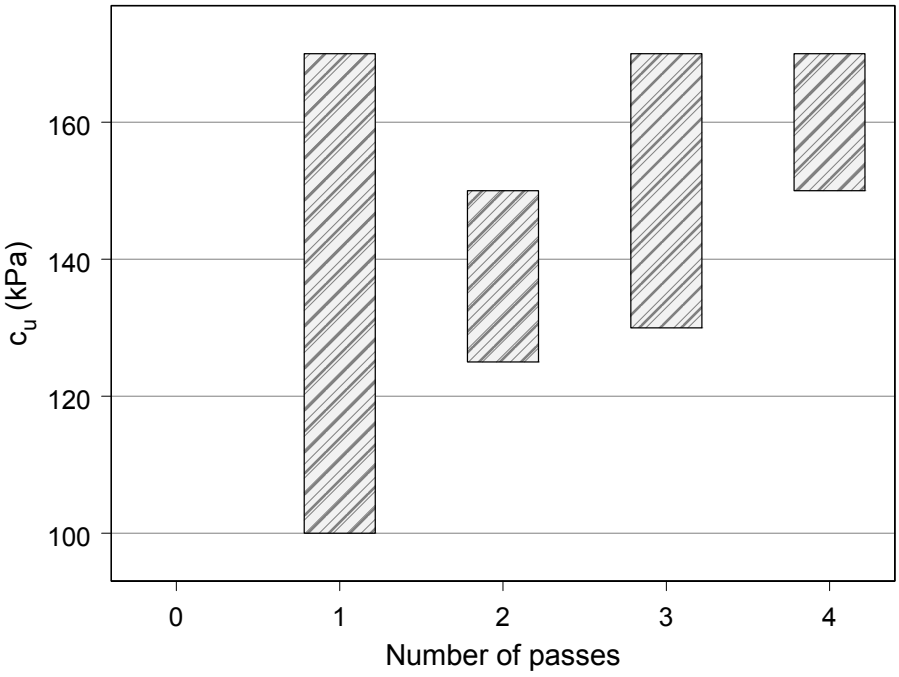


FIGURE 7.80 *Undrained shear strength for test area two as a function of roller passes.*

After a curing period of 16 hours the undrained shear strength varied from 185 to 350 kPa for test area two.

A plausible explanation for the lower undrained shear strength in test area one is the stabiliser had a severe bogging and thereby milled to a larger depth than calculated, see Figure 7.81. The increased milling depth leads to a dilution of the binder content in the modified soil and a lower undrained shear strength. The bogging on test area two was not so sever.

FIGURE 7.81 *Photo showing the bogging of the stabiliser.*

The results from this field test show a great potential in modifying fill material. Both the technique to modify at the borrow site and modification at the fill site have been successful.

8 Conclusions

8.1 Conclusions

Fine-grained tills can be used as qualified fill material in earthworks, where in relevant applications, then replace gravel and crushed rock material. Gravel and crushed materials are generally used today owing to bad experience of fine-grained tills. Such bad experience is often caused by lack of knowledge of how to treat fine-grained soils correctly. One other factor that restricts the use of fine-grained tills is lack of time for pre-testing and the need for proper weather conditions. Fine-grained tills can not generally be used during rainy or cold seasons. Infrastructure projects are often large investments and the payback period needs to start as early as possible.

With better knowledge and more developed tools to predict and verify the properties of fine-grained tills, these can be used with a good result. One key tool is the soils' acceptability criteria. The acceptability criteria could be defined in many different ways, e.g. by water content, MCV, shear strength, degree of saturation etc.

The criteria shall also be able to predict the effect of stabilising the fine-grained till by different additives. The MCA has been proven to be very efficient and accurate as a tool for evaluating the soil acceptability. This thesis shows that it is possible to predict the compaction and shear strength characteristics at different water contents from the moisture condition value (MCV). The water content is known to be a determinant factor of the performance of a fine-grained till. It can also indicate the soil's susceptibility to wetting and/or drying. The MCA is a much faster test method compared to the Proctor method. Since the MCA is developed to be suitable for field conditions, it is easy to perform the test on site and the test result is available within 10 minutes. MCA in combination with vane tests includes both the acceptability tool and the control tool. However, the vane test results are restricted to fine-grained tills.

The compaction- and strength properties of the soil have to be determined during an early stage of the design. Then it is possible to plan for activities such as creating mass balance along the alignment and to consider different treatment methods. The soil treatment could include stock piling and dewatering of the soil by aereation. However, a faster and more precise soil treatment method is soil stabilisation. Soil stabilisation creates a fill material whith is not as weather dependent.

The soil modification process is shown to be a fast and efficient way to treat a wet soil which can be used immediately after the treatment. The tests presented show that Swedish fine-grained tills can be treated by a single additive or by combined additives.

Lime is the main soil-stabilising agent used wordwide. However, prefereably cement can be used in cold weather conditions, since that

chemical reactions also take place at low temperatures in contrast to lime with its pozzolanic reactions. Soil modification lowers the soil's water content and thereby improves the soil's compaction properties. However, the change in compaction properties is mainly an effect of the flocculation of the fine particles of the soil.

The soil stabilisation process includes a first phase to achieve a compactable soil. The effect will then proceed after compaction to reach a good bearing capacity. Different stabilising agents should be chosen depending on soil type and purposeof the stabilisation. Blended binders have been proven to be more efficient then single binders. Blended binders have a longer working period than OPC, which is advantageous.

The evaluations of blended binders should preferably be performed with response surface methodology (RSM), a statistical evaluation technique that evaluates the interactions between the different agents. Different RSM techniques should be used, depending on which type of parameter is to be evaluated.

As a conclusion, the research work shows that:

- MCA can be used to predict the density and undrained shear strength for a fine-grained till
- MCA can be used to predict the immediate shear strength for a modified or stabilised fine -grained till.
- Unaxial compression tests are preferable to evaluate the short-term shear strength
- Triaxial tests show that increased consolidation stress has a good effect on wet fine-grained fills (low MCV)
- Different additives and amounts should be tested in each case before the final design

8.2 Future research

During this research project some areas of special interest have been identified. These are: Incorporating more types of fine- and medium-grained soil as well as fine grained industrial by-products to the data presented here. Incorporating different materials (e.g. waste rubber) into a stabilised soil to decrease its stiffness is a way to improve the soil's freeze- and thaw characteristics and to reduce the amount of frost damages on roads. Further research needs to be done regarding agents to make the stabilised soil hydrophobic.

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Appendix A

A.1 Evaluation of binder quantity by statistical methods

The purpose of this evaluation is to optimise the amount of binder required to achieve the desired quality. Maximum unconfined compressive strength is achieved with a certain optimum quantity of stabilisation agent. However, this is the contrary to economic considerations and could also result in a stabilised soil with too high stiffness that will result in cracking. Therefore, a compromise that fulfils all aspects must be found. Regarding quantity evaluation, two different techniques have been used: Central Composite Design (CCD) and the Box-Behnken Design (BBD).

A.1.1 Central Composite Design (CCD)

The CCD is one of the most common second-order designs used in engineering (Myers and Montgomery, 1995). A CCD consists of a 2^k factorial with n_f runs, 2^k axial or star runs, and n_c centre runs. The k=2 CCD is presented in Figure A.1.

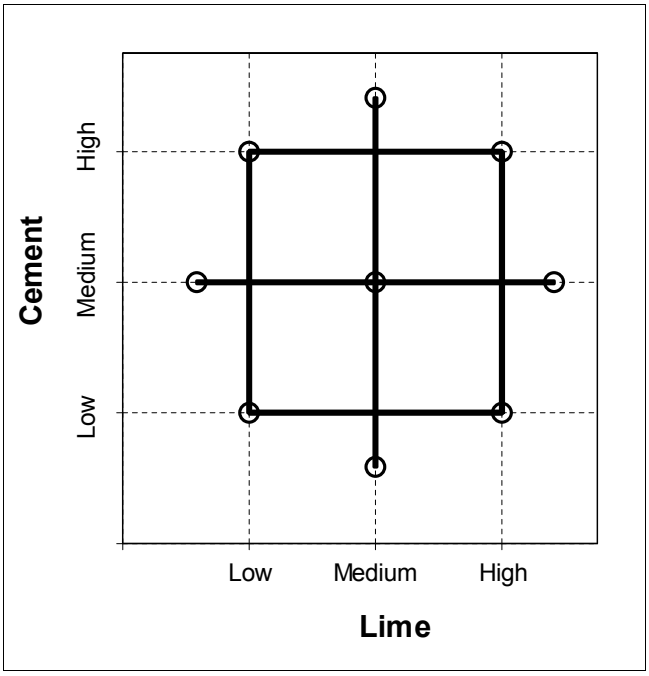

FIGURE A.1 *A CCD for k = 2 with star points and rotatable second-order designs.*

In this design each factor will be tested at least at three levels, low, high and medium. The response follows the general model equation, Equation A.1.

$$y = \beta_0 + \beta_1 \cdot x_1 + \ldots + \beta_k \cdot x_k + \beta_{12} \cdot x_1 \cdot x_2 + \beta_{13} \cdot x_1 \cdot x_3 + \ldots + \beta_{(k-1,k)} \cdot x_{k-1} \cdot x_k + \beta_{11} \cdot x_1^2 + \ldots + \beta_{kk} \cdot x_k^2 + \varepsilon \quad \textbf{(EQ : A.1)}$$

where:

β_i = regression coefficient

ε = statistical error, $\varepsilon \in (0,\sigma^2)$.

In other words, fitting a model to the observed values of the dependent variable y, that includes (i) main effects for factors $x_1, ..., x_k$, (ii) their interactions ($x_1 \cdot x_2$, $x_1 \cdot x_3$, ... ,$x_{k-1} \cdot x_k$), and (iii) their quadratic components (x_1^2, ..., x_k^2). No assumptions are made concerning the "levels" of the factors, and one can analyse any set of continuous values for the factors that can be analysed (Anon., 1995a). This general model has several advantages:

1. It is flexible especially in the search for any optimum in the response. It is possible to find maxima, minima and saddle points, (or optimal regions).
2. It is easy to estimate the parameters.
3. Experience of this model is generally good in other applications of experiments in industrial design.
4. Although the model will not function for the whole factorial range, a good approximate representation of the true response function will be obtained for parts of the operation region, for example, in the narrow range around the optimum.
5. Depending on the choice of levels of the independent factors the model can be used as an aid in determining in which direction the optimum lies.
6. There is a close connection between the second-order models and factorial experiments such as CCDs.

A CCD with k=2 can be described as a 2^2 factorial design with four axial runs, as shown in Figure A.1. This design can be used to fit the second-order model (Montgomery, 1996).

If a face-centred CCD for k=2 is used the treatment combination is exactly as in a 3^2 design, as shown in Figure A.2.

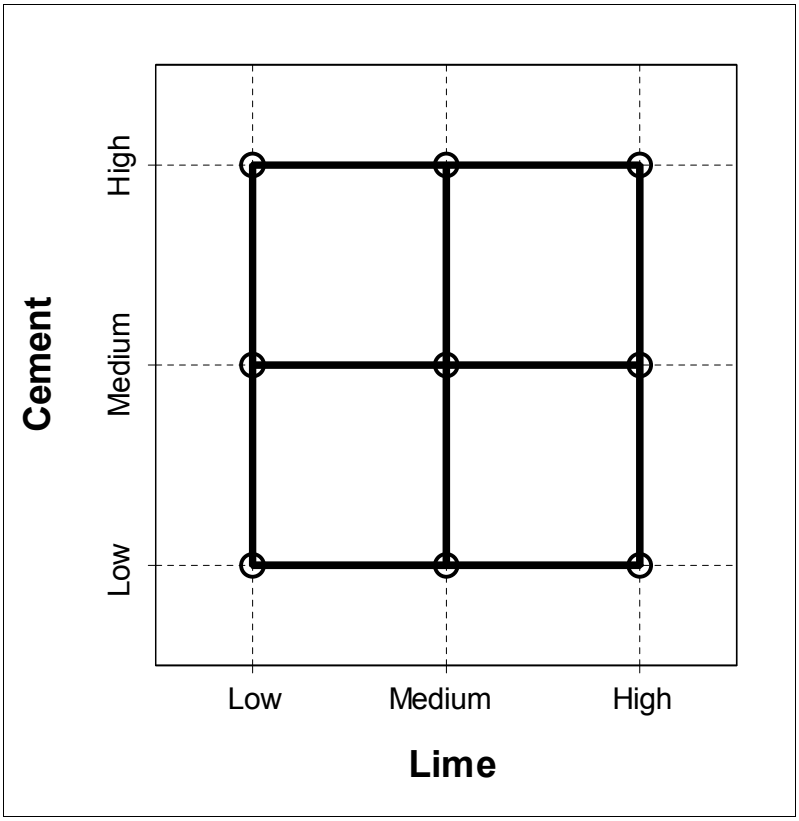

FIGURE A.2 *The figure shows a 3^2 factorial design.*

Compared with a factorial design, there are some constraints on some factor combinations for a standard CCD. For example, high levels of both cement and lime bind a lot of water which will result in a very dry soil mixture that will be very difficult to compact and is therefore undesired. This difference in operative region is illustrated by Figure A.3.

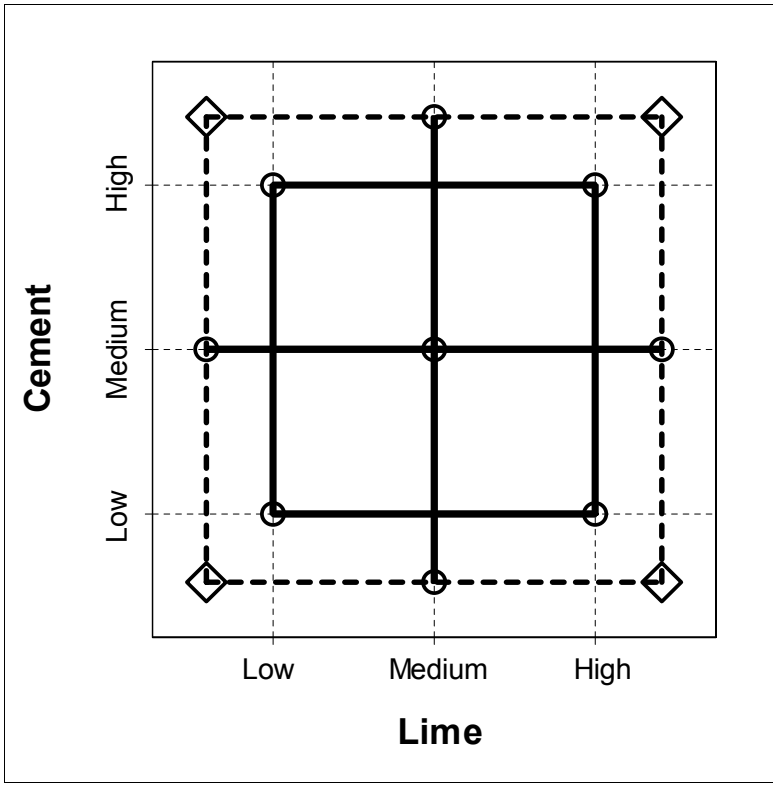

FIGURE A.3 *The figure shows a comparison between a 2-factor CCD and a 2^2 factorial design. The continuous line represents the CCD and the dashed lines represent the 2-factor design.*

The first experimental design tested in this study was a CCD for k=3 with cement, lime and water as independent variables. The soil used in the experiment was clay till from Yttre Ringvägen (the outher ring road) in Malmö. This clay till is being used as embankment fill material for the connecting road for the Öresund fixed link. The material used in the test presented here was excavated from a trial pit at a location named Petersborg.

Due to insufficient knowledge regarding the variation of the water content of the soil specimen, the test range with respect to water content was too small. The theoretical value of the natural water content was 14.8% and the water content in the design was from 15.5 to 16.5%. The actual water content in the clay till was in a range of 15.1 to 17.4%. There were considerable problems to achieving the correct value specified in the design. Considerable effort was done to minimise the variation in water content and grading. The result was biased most likely from the variation in water content.

The second experimental design tested was a face centre CCD for k=2 with cement and quicklime as independent variables. The intention using the face centre design was to evaluate the effect of both mixed and unmixed stabilisation agents. The design shown in Figure A.1 was changed to that shown in Figure A.2. The low values were set to zero. This means that eight of the design points were stabilised material and one unstabilised, i.e. the unstabilised soil is part of the experimental region. With the low values set to zero two design points were lime and two were cement. If the low values in a standard CCD had been set to zero two of the star points would have been negative and the design impossible.

Results from one of these experiments are presented in Figure A.4 and Figure A.5. In Figure A.4, only the significant effects are used to draw the surface while in Figure A.5 all effects are used to draw the surface. This specific run consists of 40 samples, which means that every point in the design represents four samples, except for the centre point which represent eight samples. The linear main effects and the 2-way interactions significantly influence the change in water content.

The experiment (design) was evaluated by the analysis of variance technique (ANOVA) and the significance level chosen was 0.05. Analysis of variance (ANOVA) was used to test the null hypothesis of no linear or quadratic main effects of the factors and no interaction between them; the chosen significance level was 0.05. The results of the analysis of variance are presented in Table A.1.

Table A.1: *Analysis of vatiance for the change in water content.*

Factor	**Anova; R^2 = 0.79873; Adj[a]: 0.76809, 2-factors, 1 block, 40 Runs; MS-res = 0.10063, Dependent Variable: W_AFTER**				
	SS[b]	**df[c]**	**MS[d]**	**F[e]**	**p[f]**
(1) Cem. (Linear)	8.3162	1	8.3162	82.640	0.0000
Cem. (Quadratic)	0.0353	1	0.0353	0.351	0.5572
(2) Q.lime. (Linear)	4.1409	1	4.1409	41.148	0.0000
Q.lime. (Quadratic)	0.2492	1	0.2492	2.476	0.1247
1(L) by 2(L) (interaction)	0.7843	1	0.7843	7.793	0.0085
Error	3.4215	34	0.1006		
Total SS	16.923	39			

a. Adjusted R^2 (Montgomery, 1996).
b. Sum of Squares (for each treatment, error and total).
c. Degrees of freedom.
d. Means of squares.
e. F-test. $F = MS_{treatment}/MSe$
f. p-level. Signifficance level.

The p levels in Table A.1 can then be described as the probability of rejecting the null hypothesis when it is actually true. In case with p = 0.05,

this indicates that there is 5% probability that any relation between the variables found in the sample is a "fluke" (Anon., 1995a).

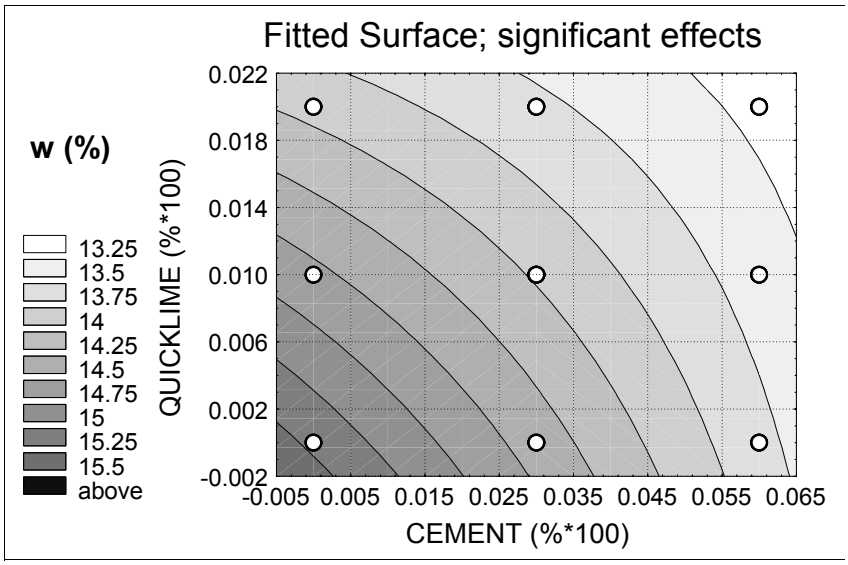

FIGURE A.4 *The significant differences in water content depending on type of and amount of stabilisating agent.*

The response surface in Figure A.4 is represented by the fitted function in Equation A.2 (cf. Equation A.1).

$$w = 15.2 - 27 \cdot cement - 63.7 \cdot lime + 738 \cdot cement \cdot lime \qquad \textbf{(EQ : A.2)}$$

Where the amounts of lime and cement are defined as percentages of the soils dry weight. Equation A.2 only includes the significant variables cement, lime and the interaction term cement*lime.

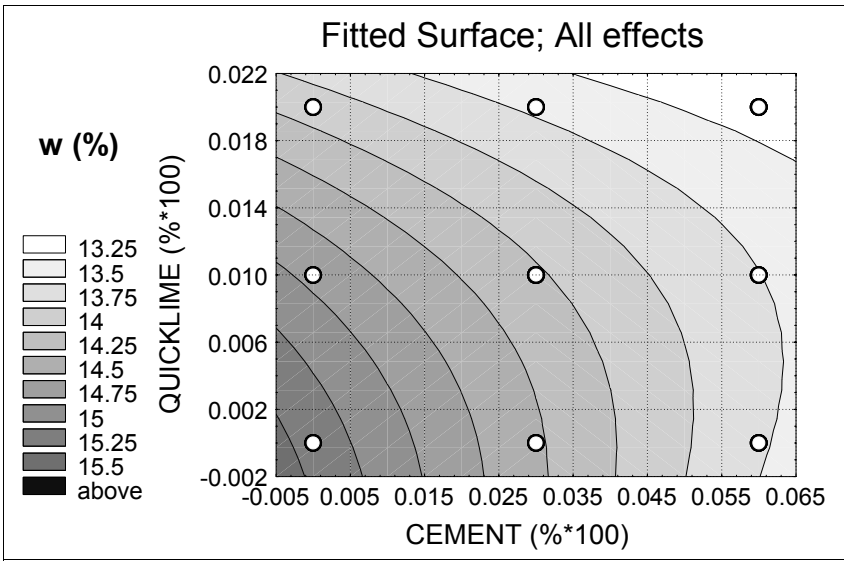

FIGURE A.5 *The differences in water content depending on type of stabilisation agent and amount of stabilisation agent without adjusting for significance.*

The response surface in Figure A.5 is represented by the fitted function in Equation A.3:

$$w = 15.2 - 31.1 \cdot cement + 68.4 \cdot cement^2 - 31 \cdot lime - 1634.2 \cdot lime^2 + 738 \cdot cemen(t \cdot lime) \quad \textbf{(EQ : A.3)}$$

In Equation A.3 all terms are included in computing the fitted function. From Table A.1 it is clear that there is an interaction between quicklime and cement, as indicated in Figure A.4 and Figure A.5. The interaction between lime and cement is significant even at p = 0.01. The result can also be presented in a Pareto chart, see Figure A.6.

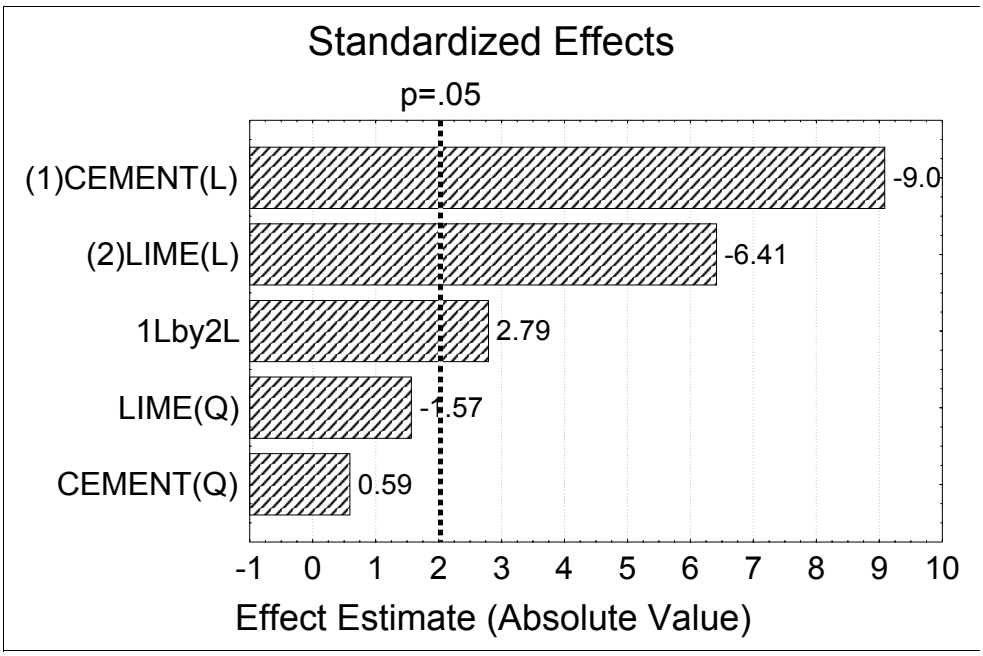

FIGURE A.6 *Pareto chart showing which effects that significantly change the variable w after stabilisation at p = 0.05.*

In this Pareto chart a column presents the effects of the ANOVA in Table A.1. The effects are sorted by magnitude. From the chart, it is possible to estimate how large an effect must be to be statistically significant at level p.

In Figure A.6 it is possible to see that cement, lime and the interaction between cement and lime are significant at p = 0.05 and that the quadratic effects of cement and lime are non significant.

There are no second order terms in the Equation A.2 as the equation is reduced with respect to significant effects, compare with Table A.1.

The interaction part of the equation is positive, which means that a mixture of quicklime and cement does not reduce the water content as much as a single stabilisation agent. This can be very effective in soils sensitive to variation in water content, such as the clay till in southwest Scania.

If it is of importance that a single binder is a part of the test design, a Box-Behnken Design should be considered instead of using a face center CCD. This recommendation is based on the results from four other runs, in which standard CCD and face-centred CCD were compared. Pareto charts of standardised effects from four different runs are presented in Figure A.7.

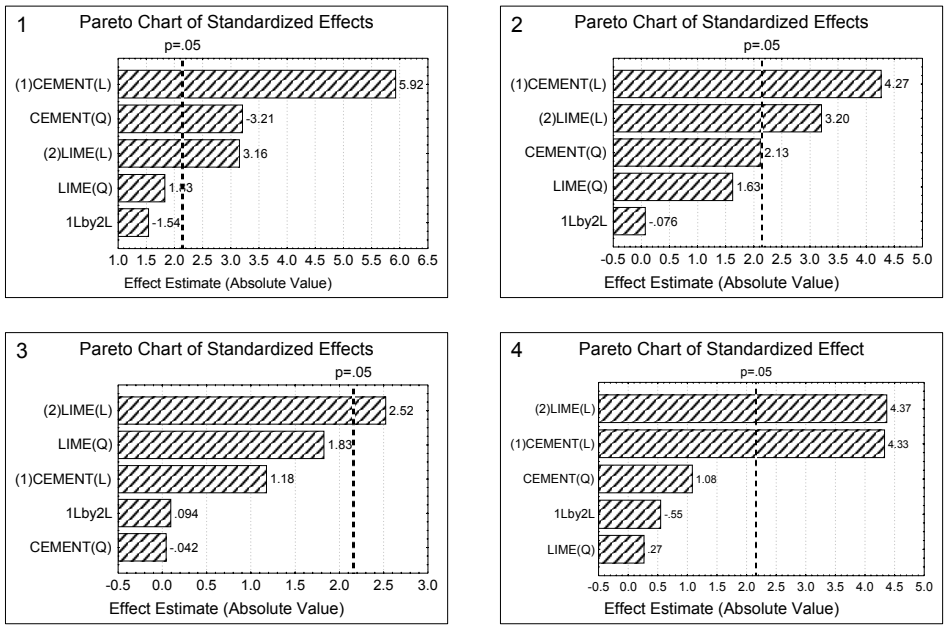

FIGURE A.7 . *Pareto charts of standardised effects from four different runs. Subplot 1 and 3 represent face center CCD and subplot 2 and 4 represents standard CCD. The response variable is the change in water content before versus after stabilisation*

In these tests, two different clay tills were tested and the response variable was the change in water content before and after stabilisation (Δw). The subplots 1 and 3 represent the two soils with face-centred CCDs. In subplot 1, three significant effects can be observed, namely the linear effect of cement (L), the quadratic effect of cement (Q) and the linear effect of lime (L). Both the linear effect of cement and the linear effect of lime increase Δw, whereas the quadratic effect of cement decreases Δw. The most likely explanation of this is disturbance from the unstabilised part of the design. The results in subplot 1 can be compared with those in subplot 2 where two effects are significant; cement linear and lime linear. The quadratic effect of cement is not significant at $p = 0.05$ in a standard CCD. The difference in results is probably caused by the fact that in a face-centred CCD, the results from the unstabilised soil disturb the final result. In subplot 3 only the linear effect of lime was found to be significant and this result must be questioned. This result disagrees with subplot 4, in with it is shown that the linear effects of both lime and cement are significant. It is beyond doubt that adding 2.5% lime to a soil reduces the water content. This means that the results in subplot 3 are disturbed. In order to ascertain whenever some exceptional data could be found, predicted versus residual values were plotted, see Figure A.8.

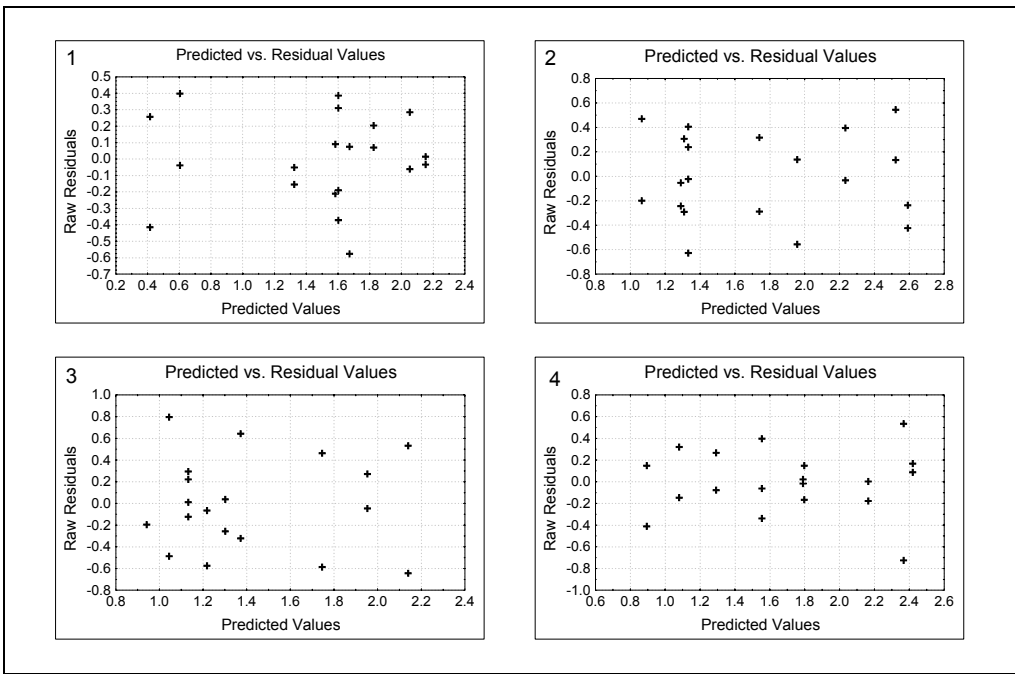

FIGURE A.8 *Predicted versus residual values from the runs in Figure A.7.*

No unusual pattern could be observed in any of the four subplots in Figure A.8, which could explain this phenomenon. Even if the scientist aims to obtain the best possible results and uses statistics in the experimental planning, the results must be scrutinised to avoid erroneous conclusions.

A.1.2 Box-Behnken Design (BBD)

Box-Behnken is another experimental design that is useful to evaluate the amount of binders. As in the CCD each factor was tested at three levels, low medium and high. In the Box-Behnken Design there are no points in the corners of the cube and no face points. This design better

suits the situation of interest in predicting the response for a single binder not including origin (i.e. no stabilisation agents). The BBD is constrained with respect to low and high levels. If a design has three or more different binders, the high level of all binders is not used in the design. This is a positive effect if the region of interest is between 1% and 5% for three binders, the amount of binders will otherwise be 15% if all high levels are used at the same time and this is inappropriate in most cases. This design is economical and therefore particularly useful when it is too expensive to perform the necessary experimental runs in the other designs. The BBD is presented in Figure A.9.

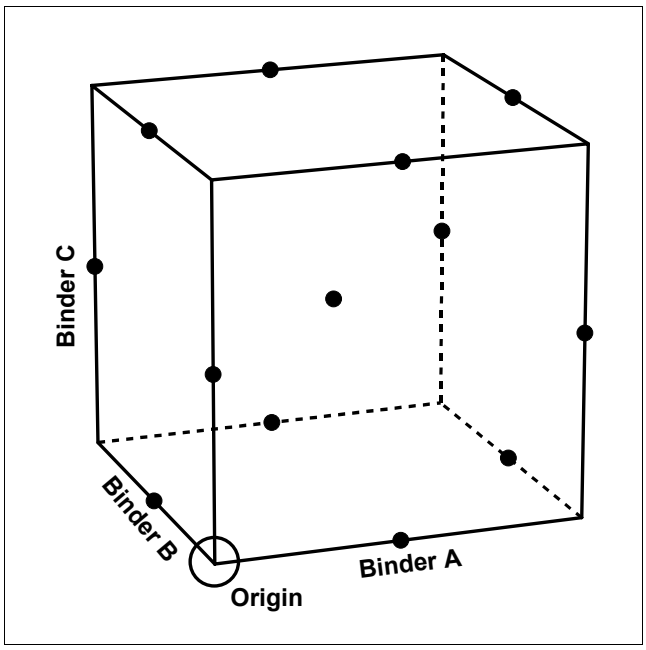

FIGURE A.9 *Box-Behnken design for three factors with a centre point.*

The origin is located at a cube corner. In this design the minimum number of factors is three. This design has been used to evaluate the working time of three different binders at three different curing times before compaction.

A.2 Evaluation of binder type

In some cases the amount of binder is set to a fixed value, or the interaction between different binders for a fixed level are examined, a mixture design need to be used. This evaluation is a form of product formulation to make the most of the stabilisation agents. In the mixture design the sum of the components is constant, i.e. the amount of binders is not an independent variable. This means that the sum of the binders must always be 100%, independent of how they are mixed. In the quantity evaluation the low, medium and high levels are independent and it is up to the scientist to decide upon the levels. In mixture design, levels are not independent. From the quantity evaluation a single level is chosen, e.g. 2.5% stabilisation agent per dry weight of soil. If the mixture contains three different binders, cement, lime and slag the blends can be as follows:

- Blend 1: cement = 30%, lime = 50%, slag = 20%
- Blend 2: cement = 50%, lime = 50%, slag = 0%
- Blend 3: cement = 100%, lime = 0%, slag = 0%

Blend 1 is an example of complete mixture, that is, that it is made up of all three of the binders. Blend 2 is a binary blend and blend 3 is single blend containing only cement.

Common models for mixture data are:

Quadratic model, see Equation A.4:

$$y = \beta_1 \cdot x_1 + \beta_2 \cdot x_2 + \beta_3 \cdot x_3 + \beta_{12} \cdot x_1 \cdot x_2 + \beta_{13} \cdot x_1 \cdot x_3 + \beta_{23} \cdot x_2 \cdot x_3 + \varepsilon \quad \textbf{(EQ : A.4)}$$

Special cubic model, see Equation A.5:

$$y = \beta_1 \cdot x_1 + \beta_2 \cdot x_2 + \beta_3 \cdot x_3 + \beta_{12} \cdot x_1 \cdot x_2 + \beta_{13} \cdot x_1 \cdot x_2 + \beta_{23} \cdot x_2 \cdot x_3 + \beta_{123} \cdot x_1 \cdot x_2 \cdot x_3 + \varepsilon \quad \textbf{(EQ : A.5)}$$

The difference between the models is that in the special cubic model there is a test for interactions between all the factors $x_1 \cdot x_2 \cdot x_3$.

A.2.1 Simplex design

The simplex designs consists of simplex-lattice and simplex-centroid designs. These two designs can be complemented with constraints if necessary. If a stabilisation agent requires an activator, e.g. some kind of slag, the single blend of that slag will affect the test and an upper constraint will be needed. Here the use of simplex-lattice design will be discussed. Figure A.10 shows a design for three components and a polynomial order 2 simplex lattice design.

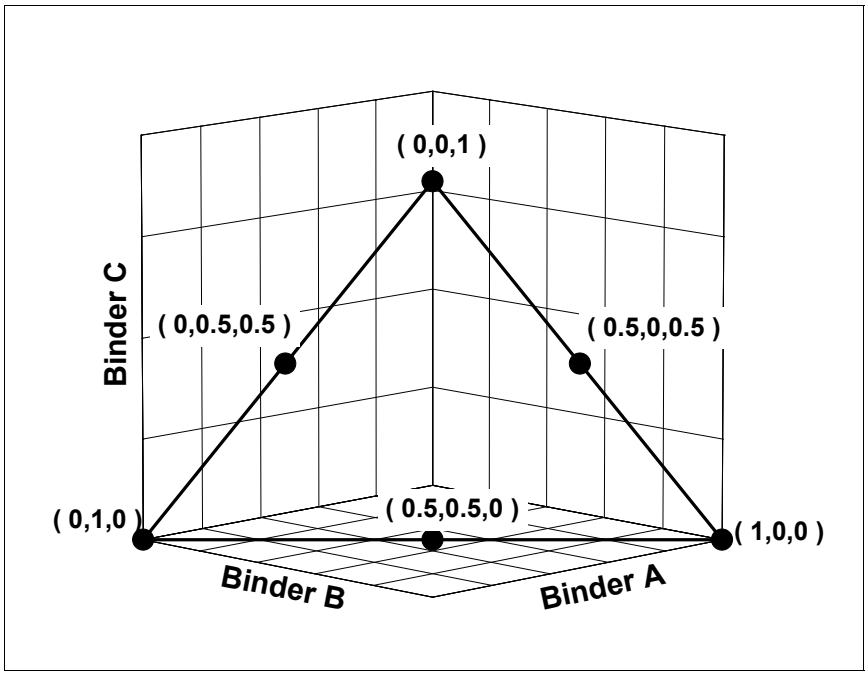

FIGURE A.10 *Simplex-Lattice design {3,2}.*

The simplex designs have been criticised as the experimental runs occur on the boundary and predictions in the interior will thus be uncertain. The design does not contain any complete mixtures, only binary mixtures or single mixtures. The interaction of all components can be negative. If the complete design region is of interest, this can be fixed by adding interior points. To gain increase in resolution, the experimental runs increase from 6 to 10.

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