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Deep soil stabilization with fly ash

Djupstabilisering av mark med flygaska

Niklas Hansson

Abstract

Deep soil stabilization with fly ash - A laboratory study

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Deep soil stabilization is used to increase the bearing capacity of soils, for example when building railroad embankments. Mixtures of lime and cement are usually used as binding agents. In recent years the interest for recycling industrial materials has increased. Fly ash, a by-product of heat and power plants, is one such material with the potential to work well as a binding agent. The aim of this project, initiated by Vattenfall Research & Development AB, is to evaluate the potential for fly ash to be used as a binding agent in deep soil stabilization.

This was achieved by performing a laboratory study on three different types of fly ash, that is two types of bio ashes and one type of hard coal ash. Different mixtures of binding agents like lime, cement and fly ash was mixed into a clay soil, and test specimens were formed and allowed to harden for 7, 28 and 56 days. Using unconfined compression tests and also some triaxial compression tests the developed compressive strength of the test specimens was tested.

The hard coal ash displayed the most promising results. Substitution of up to 70 % of the lime and cement with fly ash resulted in strength that surpassed that of only lime and cement. The two types of bio ash also showed satisfactory results. Substitution of 30 % of the lime and cement with fly ash, resulted in strengths of 80 % of the strength developed using only lime and cement. All of the three ashes work fine as binding agents in combinations with some lime and cement.

Keywords: Deep soil stabilization, fly ash, binding agent

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Referat

Djupstabilisering av mark med flygaska - En laboratoriestudie

Niklas Hansson

Djupstabilisering används för att stabilisera mark som annars är för svag att bebygga, till exempel inför byggen av banvallar och vägbankar. Blandningar av kalk och cement är de vanligaste bindemedlen. Under senare år har intresset för att använda återvunna industrimaterial ökat. Flygaska, en biprodukt från förbränning i kraftvärmeverk, är ett sådant material som kan ha potential att användas som bindemedel. Projektet har initierats av Vattenfall Research & Development AB, och har som mål att utvärdera flygaskans potential som bindemedel i djupstabilisering.

Målet nåddes genom att utföra en laboratoriestudie på askor från tre olika förbränningsanläggningar, två biobränsleaskor och en stenkolsaska. Olika blandningar med kalk, cement och flygaska blandades med en lerjord och formades till provkroppar. Dessa härdades i 7, 28 eller 56 dygn och trycktestades sedan i framför allt enaxiella trycktest för att utvärdera tryckhållfastheten. Ett begränsat antal triaxiella trycktest utfördes också.

Stenkolsaskan visade mest lovande resultat. Inblandning av upp till 70 % stenkolsaska i den normalt använda kalk- och cementblandningen resulterade i tryckhållfastheter som översteg de som uppnås när enbart en blandning av kalk och cement används. De två biobränsleaskorna visade också goda resultat. Inblandning av 30 % flygaska i kalk- och cementblandningen gav 80 % av hållfastheten som uppnås med enbart kalk och cement som bindemedel. Alla tre flygaskorna fungerar bra som bindemedel i kombination med kalk och cement.

Nyckelord: Djupstabilisering, flygaska, bindemedel

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Preface

This study is my master thesis project that will conclude my studies at the Master of Science program in environmental and aquatic engineering at Uppsala University. The study was carried out for Vattenfall Research & Development AB in Älvkarleby. My supervisor in the project has been Caroline Göthlin - Vattenfall Research & Development AB, and my subject reviewer has been Kennet Axelsson, Department of Earth Sciences, Uppsala University.

My thanks go to my supervisor Caroline Göthlin for all your time and help, and Johan Lagerlund for your invaluable help and assistance with the triaxial testing and for our fruitful discussions on soil mechanics. My thanks also go to Gunnar Tibblin at Sweco Geolabs for all your answers to my many questions on laboratory work, and to Kennet Axelsson for your time and deep knowledge on the subject.

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Populärvetenskaplig sammanfattning

Djupstabilisering av mark med flygaska - En laboratoriestudie

Lera har en förmåga att suga upp och hålla mycket vatten, vilket ökar risken för jordskred vid kraftiga regn. Dessutom komprimeras den långsamt under belastning, vilket leder till att byggnader och vägar kan börja luta, sätta sig eller spricka efter några år. Under hus är det vanligt att man placerar pålar som bär upp husets grundplatta för att komma undan problemen. Detta är svårt när man anlägger t.ex. vägar och banvallar. Ett sätt att lösa problemen är att schakta bort de svagare jordpartierna och fylla ut med massor med bättre bärighet, t.ex. bergkross eller grus. Denna metod är dyr och förutom att man måste skaffa fram fyllandsmassor får man även en stor mängd bortschaktad jord att ta hand om. Ett annat sätt är att stabilisera den svaga jorden så att den blir stabilare och mer bärkraftig. På detta sätt slipper man stora schaktarbeten och bergstäkter, både miljö och pengar kan sparas.

Stabiliseringsmetoden går ut på att man blandar upp jorden med bindemedel som sedan reagerar kemiskt med vatten och jordmaterial och bildar en mer hållfast jordmassa. Mängden bindemedel som blandas in i jorden varierar mycket beroende på jorden och hur hårt det behöver bli. Vanliga inblandningsmängder är i storleksordningen under tio viktprocent. De vanligaste bindemedlen är kalk och cement. Under senare år har intresset för att använda återvunna industrimaterial ökat. Flygaska, en biprodukt från förbränning i kraftvärmeverk, är ett sådant material som kan ha potential att användas som bindemedel. För att testa hur bra aska fungerar som bindemedel genomfördes en laboratoriestudie. För att begränsa omfattningen har arbetet koncentrerats till askans inverkan på hållfastheten. Andra intressanta aspekter, som t.ex. lakning av föroreningar till omgivningen, har inte tagits upp.

Aska från tre olika värmekraftverk valdes ut att ingå i laboratoriestudien, två askor från biobränsleförbränning och en aska från stenkolsförbränning. Eftersom kalk och cement normalt används som bindemedel inkluderades de i studien för att kunna jämföra med vilken hållfasthet som normalt skulle kunna uppnås. Viktiga frågor var:
Hur mycket aska kan blandas in kalkcementblandningen och fortfarande utveckla bra hållfasthet. Hur ser hållfasthetsutvecklingen ut över tid? Är det någon skillnad på askornas hållfasthetsutveckling?

Eftersom det är praktiskt och ekonomiskt svårt att genomföra en stor serie försök ute i verkligheten skedde proven i laboratorium. Det är viktigt att alla prov genomförs på samma sätt samt under kontrollerbara och repeterbara förhållanden så att resultaten verkligen blir jämförbara. Tre olika receptblandningar av kalk, cement och aska valdes ut för att kunna utvärdera hur olika mängder aska påverkar resultaten. Den vanligaste blandningen av bindemedel för stabilisering av mark är 50 % kalk och 50 % cement. För att kunna säga något om hur bra blandningar där olika askor ingår är användes kalk och cement blandningen som referens. Lera blandades med en viss mängd bindemedel (blandningar av kalk, cement och de tre olika askorna). Av denna massa formades ett stort antal cylindriska provkroppar, som var tio centimeter långa 5 centimeter i diameter. Dessa placerades i kylrum för att härda. För att ta reda på hur hållfastheten utvecklas över tid valdes tre olika härdningstider, 7, 28 och 56 dygn. När provkropparna härdat sina respektive härdningstider placerades de i en maskin som pressade ihop provcylindrarna tills de sprack. Samtidigt mättes det tryck som behövdes för detta. Detta tryck är ett mått på provkroppens hållfasthet.

När alla provkroppar testats fanns en stor mängd mätvärden. Genom att behandla mätvärdena i ett datorprogram kunde diagram ritas som visar sambanden mellan härdningstid, hållfasthet och de olika bindemedlens mängder. Det mest intressanta diagrammet skapades genom att rita upp hållfastheten och härdningstiden för de olika bindemedelsblandningarna. Genom att jämföra de olika kurvorna i diagrammet kan många slutsatser dras.

Hållfasthetsutvecklingen är inte jämn under hela härdningstiden. Den största hållfasthetsökningen sker under de första sju dagarna, ökningen i hållfasthet blir sedan mindre och mindre. Olika blandningar ger olika hållfasthet och vissa blandningar fungerar bättre än andra. De blandningar som innehöll stenkolsaska visade mest lovande resultat. Genom att blanda in upp till 70 % stenkolsaska i bindemedelsblandningen fås en hållfasthet som är bättre än om man bara använder kalk och cement. De båda biobränsleaskorna visade också goda resultat. Inblandning av 30 % biobränsleaska i kalk- och cementblandningen gav 80 % av hållfastheten som uppnås med enbart kalk och cement som bindemedel.

Alla tre askorna fungerar bra som bindemedel i kombination med kalk och cement. Denna studie visar att aska kan ha en stor potential som bindemedel inom markstabilisering. Mer forskning behövs dock om aska ska kunna introduceras på marknaden och börja användas i verkliga stabiliseringsprojekt. Det är nödvändigt att utreda hur användning av aska påverkar miljön, t.ex. genom urlakning av giftiga ämnen. Genom att utföra livscykelanalyser för stabiliseringsmedel kan en samlad bild av ämnenas miljöpåverkan ges.

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APPENDICES

1. INTRODUCTION

1.1. BACKGROUND

In recent years the philosophy of recycling has taken a grip of all of society. Household waste is separated and recycled, and there is a growing demand on the industry to do likewise. Some industrial waste is directly harmful, and both people and the environment should be protected from exposure to it. But most materials that have been considered waste can be recycled and used as a raw material in other applications. In addition, the Swedish environmental legislations require that waste producers strive to implement recycling of waste material to an extent as large as possible. This means that instead of producing a product and waste, most industries produce a primary product and a number of secondary products. A power and heating plant is no exception, it produces heat and electricity, and furthermore it produces secondary products in the form of different types of ash. The difference between a secondary product and waste is that the secondary product is generally both useable and not dangerous to people and the environment. Important however, is that technical and environmental properties of the secondary products are evaluated from requirements related to the specific application. In many cases it saves both a lot of energy and a lot of money, to use a recycled material instead of conventional ones. Since fly ash has some stabilizing properties, it is a natural next step to investigate the possibility to use it for deep soil stabilization.

1.2. PROBLEM DESCRIPTION

Since Vattenfall AB produces large quantities of ash, there is a growing interest to do something useful with it. Today ash from hard coal combustion is used in for example concrete and asphalt applications, while ash from bio fuels is a less valuable product. Most of the bio ash is reused, but not always in industrial applications, as the hard coal ash is. The most common application for bio ash is as a structural layer in gravelled roads or surfaces. Only a few percent is landfilled. Fly ash has been used as a building material for 30 years on the European continent, due to the large amount of hard coal combustion products. The energy production in Sweden has not relied on coal to a large extent and has only since the beginning of 1990 used biomass fuel in power plants. In the last ten years the industry in Sweden has realized that waste can be turned into valuable products. The technical utilisation of bio ash has therefore a large potential for more advanced uses, as a product for example in soil stabilization. As a step in that direction, the main objective of the project was:

- Evaluate the potential for ash as a binding agent in deep soil stabilisation at a laboratory scale.

The main objective can be divided into the following sub-objectives:

1. Create understanding for soil mechanics and the chemistry related to deep soil stabilization so as to set up a laboratory testing program.
2. Perform laboratory tests of hard coal ash and bio ash with relevant laboratory equipment.
3. Evaluate laboratory tests in order to tell the potential.

To achieve the objectives relevant reports and papers on the subject were studied, and people with expert knowledge in the area were interviewed. This study deals with the potential to use ash for deep soil stabilization purposes. But to delimit the extent of the study, it focused only on the aspects of strength, and how well different types of ash work as binding agents. The environmental aspects, such as possible leaching of environmentally unfriendly substances, were not discussed at all. Analyses of the marketing possibilities of ash were also excluded.

The hypothesis of the project was:

- Fly ash is a binding agent that works well, and has large potential in deep soil stabilization applications.

2. METHODS FOR DEEP SOIL STABILIZATION

2.1. DEEP SOIL STABILIZATION

Soil stabilization has a long history. As early as 2000 years ago the Romans used lime to stabilize clay soils (Lindh, 2000). In the 1970s, the method to use lime columns in deep stabilization was developed in Sweden (Holm & Åhnberg, 1987). Since the late 1980s, the dominating binding agents have been combinations of lime and cement (Åhnberg, 2006). During the last years, the interest to use recycled industrial material has increased, examples of these are fly ash, blast furnace slag, silica fume and gypsum.

There are different ways of stabilizing soils. The choice of method depends on the objective, though the reactions and theory for stabilization are the same. One is called mass soil stabilization, and means that the total mass of soil is mixed with binding agents. In this method it is difficult to make the stabilized layer deeper then a few meters, but a large area can be easily covered. The method is for example used when the topmost layers of the soil are the weakest and a large area needs to be covered. One other method is called deep soil stabilization and here the stabilization is performed in the form of columns. They do not cover an entire area but they reach many meters into the ground. This is important when for example building embankments for roads and railroads. This study focuses on investigating the effects of deep soil stabilization. However, the results can certainly be applicable to mass soil stabilization as well, but this is not covered in detail.

2.1.1. Methods for installing deep soil stabilization columns

The installation of deep stabilization columns is described in detail in the Swedish standard "SS-EN 14679:2005 Execution of special geotechnical works - Deep mixing". There are two different methods to create lime/cement columns, one dry method and one wet method. When using the wet method the binders are mixed with water and the slurry is then mixed into the soil. This method is mainly used in Japan and not in Europe, therefore it will not be discussed further in this study. The dry method on the other hand is the most commonly used in Sweden and Finland (Kivelö, 1998). The binders are mixed dry in the soil using air pressure, through a mixing tool shaped like a propeller with tilted wings mounted on a hollow shaft. The mixing tool is first rotated down to the desired lowest level of the column at a speed of 100 - 200 revolutions per minute and penetrates down at a rate of 100 mm per revolution (Carlsten, 2000). On the way down the tilted wings disaggregates the soil. When it has reached the lowest level, the rotational direction is reversed and the mixing tool is then withdrawn upwards again, see Fig. 1. The speed, 15 - 25 mm per revolution, is varied depending on the binders, the soil and other parameters (Carlsten, 2000). On the way up the binders are pressed down through the hollow shaft and out of holes near the mixing tool using compressed air approximately, 400 - 600 kPa. By the rotation of the mixing tool, the binders are mixed into the soil in the shape of a cylindrical column, with the same diameter as the mixing tool, typically 0.5 - 0.8 m (Åhnberg 2006). The rate of the binding agents mixed into the soil is set according to design criteria, given as kilogram binding agents per cubic meter of soil or per meter of column. The amount of binders usually varies between 70 and 150 kg/m^3, but the most common is 80 - 100 kg/m^3 (Edstam, 1997). The columns are placed in a pattern, for example in rows, circles, grids or overlapping each other. Which pattern to be used depends on the circumstances and the aim of the stabilization.

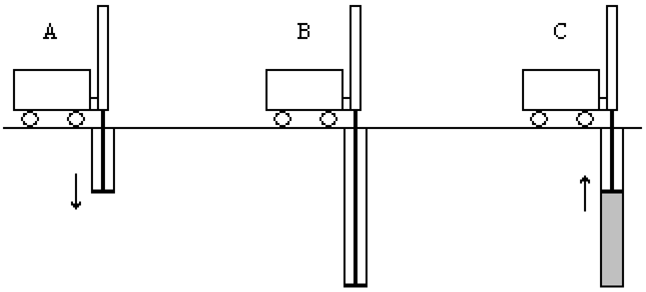

Fig. 1 The installation procedure of a column: A - The mixing tool is rotated down to the desired lowest level, disaggregating the soil simultaneously. B - At the lowest level the rotational direction is reversed before the mixing tool is withdrawn. Binding agent injection starts. C - While the mixing tool withdraws it rotates, causing the binding agents to mix into the soil, forming a cylindrical column of stabilized soil.

2.1.2. Binding agents

Depending on the soil properties, different binding agents are used. Only lime, cement and fly ash are dealt with here, because they are the only binding agents used in this study. Soil stabilization chemistry is covered superficially, due to the fact that it is a very complex science. For deeper reading on the subject, Janz & Johansson 2002 is recommended. Water is a requirement for the chemical reactions of the binders, and the moist in the air is enough to start their chemical reactions. If the binders start to react before they are mixed into the ground their stabilizing effect will be reduced. As long as they are stored in airtight containers they remain all but inert.

Cement

Cement was used for building purposes as early as during the Roman Empire, and is today a fundamental material in the building industry. To manufacture cement, limestone and clay are mixed and ground. Then the mixture is calcinated in a kiln at 1400°C. The result of this process is pellets called Portland clinker. The clinker is then mixed with for example gypsum and ground to micrometer scale. There are a vast number of cement types for different applications. To give the cement other properties different additives are included with the clinker in this last grinding (Janz & Johansson, 2002).

When cement comes in contact with water, chemical reactions start. A cement gel, with a chemical name abbreviated as CSH, forms around the cement particle. This CSH gel is porous and consists mostly of calcium and silicon, with some chemically bound water. It acts like glue as it fills the voids between the particles causing the cement to grow denser and stronger. Due to the chemical reactions, heat is emitted. Many complex chemical reactions take place concurrently. The most important strength enhancing substances consist of calcium, silicon, aluminium and iron. Normally half of the cement particles have reacted after three days, and after three months 90 % has reacted (Janz & Johansson, 2002). Hence the initial strength increase is very fast.

Lime

Burnt lime, CaO, is manufactured by calcinating limestone, $CaCO_3$, at 1100°C. Large quantities of CO_2 are emitted during the calcination reactions, see Equation 1. When calcinating limestone, approximately 40 % of the mass departs as gases, mostly CO_2, (Rogbeck et al., 2006)

$$\mathrm{CaCO_3} + \text{heat} \rightarrow \mathrm{CaO} + \mathrm{CO_2} \qquad (1)$$

CaO exists in several forms, some are easily available for chemical reactions, this is called CaO_{reactive}, and some are bound to chemical compounds. It is therefore a difference between the total amount of CaO and $CaO_{\text{reactive.}}$ Burnt lime typically consists of approximately 90 % CaO_{reactive}. When the burnt lime comes in contact with water it reacts and forms hydrated lime, $Ca(OH)_2$, see Equation 2.

$$\mathrm{CaO} + \mathrm{H_2O} \rightarrow \mathrm{Ca(OH)_2} + \text{heat} \qquad (2)$$

This reaction, Equation 2, gives several important advantages in soil stabilization. It helps to reduce the water content of the soil, which has a fast stabilizing effect. It gives an increase in pH, a condition for secondary pozzolanic reactions to take place (Janz & Johansson, 2002). It produces heat, which increases the speed of chemical reactions. In itself, CaO does not cause any strength increasing products to form, but in secondary pozzolanic reactions with pozzolanic material in the soil, compounds containing mostly calcium, aluminium and silicate is formed. These compounds are similar to those formed by cement. Compared with cement reactions, pozzolanic reactions are very slow. While cement reacts in a matter of days, pozzolanic reactions take months to gain its full strength. The word pozzolana comes from the Romans, who used a volcanic siliceous ash as a hydraulic binder, which they found at the city of Pozzuoli.

Fly ash

When burning hard coal or bio fuels in a boiler, heat, flue gases and ash are produced. The heaviest and largest ash particles fall down in the furnace, and are called bottom ash, and are usually extracted below the furnace. The particles of the bottom ash are generally large and often have a certain amount of unburned organic material in it, often it is wetted in the process. The smaller ash particles are suspended in the flue gases, so before the flue gases are allowed to leave the plant they pass through a filter. Here the lighter and smaller particles are caught in the filter, this ash is called fly ash. This is all illustrated in Fig. 2. All further mentioning of ash in this report refers to fly ash, if nothing else is explicitly expressed.

Fly ash is vitreous to its structure, and contains pozzolanic materials. The burning temperature is important for the quality of the ash. A higher combustion temperature gives a more vitreous ash with finer particles, causing the ash to be more efficient as a binding agent. Hard coal is usually combusted at around 1300°C, and bio fuels at a slightly lower temperature, around 800°C. This is one of the reasons why bio ash usually has a lower stabilizing effect than hard coal ash.

The pozzolanic materials in the ash react with water and CaO, both from the lime and from the cement. It forms similar products as in the cement reactions. Since the pozzolanic reactions are very slow, the strength increase caused by ash is also very slow. The size and the rate of the strength increase are mainly determined by the chemical composition of the fly ash. It can vary significantly from ash to ash since fuel, combustion and combustion technology are the main factors deciding the quality of ash.

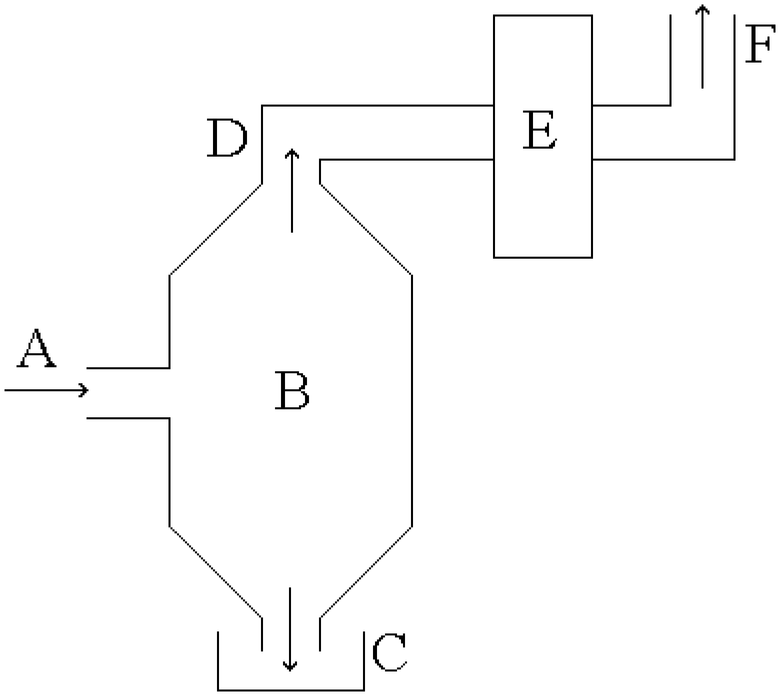

Fig. 2 Schematic view of a boiler. A - Fuel goes in to the furnace. B - Combustion takes place. C - Bottom ash is extracted. D - Flue gases with suspended fly ash. E - Fly ash is separated from the flue gases in a filter. F - Flue gases are emitted.

2.2. EVALUATION OF STABILIZING EFFECT

In the early stages of a planned stabilization project, laboratory studies are an important part to evaluate the outcome of the stabilization. Especially when determining what amount of and what type of binding agents that works best with the soil at the specific site. Additional testing can be needed, for example by installing columns in full scale on the specific site. This enables evaluation of the stabilizing effect in natural conditions. The composition of the soil can vary to a great extent with depth, causing the effect of stabilization to vary likewise. When a stabilization project has been carried out, it is necessary to know if the performed stabilization fulfils the requirements of the standard. It is therefore very important to follow up with extensive testing.

2.2.1. Field evaluation

A few methods exist to evaluate the effect of stabilization. One is to install a probe at the bottom of the column during the manufacture, and at the time of testing pull it up through the column and register the required force. The most common one however is to push a probe down through the hardened column at the time of testing, and register the required force. Either way the result is a graph showing the strength of the column at all depths. Both these methods give a good picture of how successful the stabilization has been. It is important that the path of the probe follows the column straight, which sometimes can be hard to obtain. Otherwise the results might get very misleading, because the strength is not necessary the same in the centre of the column as in its circumference.

Another possibility is to use a piston sampler, to take samples from the column and bring them to a laboratory, where strength testing can be performed. This however changes the environment around the sample significantly, especially the pressure from the surrounding soil.

2.2.2. Laboratory evaluation

Testing the effect of binders through full scale field tests is very expensive and time consuming. Therefore it is important that before a stabilization project starts, a large testing program will be initiated to find the most appropriate binder, and the optimal amount of binder to be added to the soil. These first tests of binders are solely performed in laboratory. The big advantage of laboratory work is that it enables a large number of tests under controlled conditions at a proportionately low price. It is not easy to imitate natural conditions. Therefore it is not always possible to compare the results from a laboratory test with the results from a field evaluation. Different laboratory methods test the same or different parameters under different conditions and hereby laboratory work should be seen as a relative science. Laboratory results cannot be directly compared with results from other laboratory methods or with field results, it has to be compared within the same material. To be able to compare results between studies it is important to include material that has been tested in other studies as a reference.

There are many methods to test soil. To investigate compression properties Oedometer tests, CRS (constant rate strain) tests and isotropic triaxial tests are used. To investigate shear properties, direct shear apparatus tests, triaxial tests and unconfined compression tests are used. Norms and standards have been established for almost all laboratory work, to ensure that all tests are made in the same way. If two laboratories do the same test the results must be equivalent. To make sure that laboratories follow standards and maintain a certain level of quality, an accreditation body inspects and grants accreditation to the laboratories. In Sweden the accreditation body is called Swedac.

Since this study used unconfined compression tests to test stabilizing effect, and some triaxial compression tests were performed in a secondary part of the study, these methods are the ones discussed in detail in the remainder of this chapter.

Test specimen

To test a material, a sample must be prepared, small enough to be easily handled yet large enough so that testing gives reasonably good results. It is important that all test specimens are manufactured in the same way and handled identically, so that nothing else than the parameter intended to test varies. The size of the test specimens is chosen to fit the testing method. For further details on the manufacture of the test specimens see Chapter 4.2.2.

Unconfined compression tests

The most common type of test is the undrained unconfined compression test. The main reasons are that they are quick and easy to perform, and therefore cheap, and they are easy to interpret. The test specimen, in the form of a cylinder, is simply loaded uniaxially (vertically), and the vertical pressure, $\sigma_{1,}$ needed to bring the test specimen to failure is recorded (Axelsson, 2005). It does not imitate natural conditions very well because no regard is taken to the lateral pressure, $\sigma_{3,}$ from surrounding soil. In this case σ_3 is equal to zero, see Fig. 3. The lateral pressure acts stabilizing, increasing the pressure needed to bring the soil to failure. Yet if reference samples are tested in the same way, relative conclusions can still be drawn. Unconfined compression tests are the standard way to test, and because almost all stabilizing tests are performed this way, a large amount of data exists to compare results with.

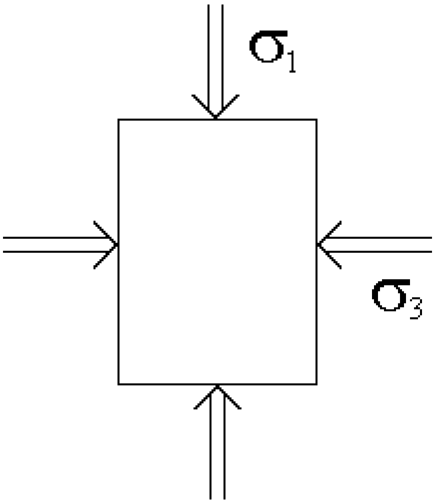

Fig. 3 A test specimen with the vertical pressure σ_1 and the horizontal lateral pressure σ_3.

Drained Triaxial compression tests

Triaxial compression tests differ from unconfined compression by a constant lateral pressure, σ_3. It is applied to the test specimen at the same time as the test specimen is loaded vertically as in the unconfined compression test, see Fig. 3.

This test gives a better understanding of how the soil reacts in situ. If two or preferably three test specimens are tested under different lateral pressures, their σ_1 and σ_3 values are recorded. They can then be plotted and Mohr's circles can be constructed in a Mohr-plane. With the circles a Mohr's envelope can be constructed. This enables the calculation of the cohesion and the friction angle se Fig. 4. A true frictional material like dry sand has no cohesion, only friction keeps the material together. The majority of all soils are, however, mixed materials, where both cohesion and friction acts at the same time, as illustrated in Fig. 4.

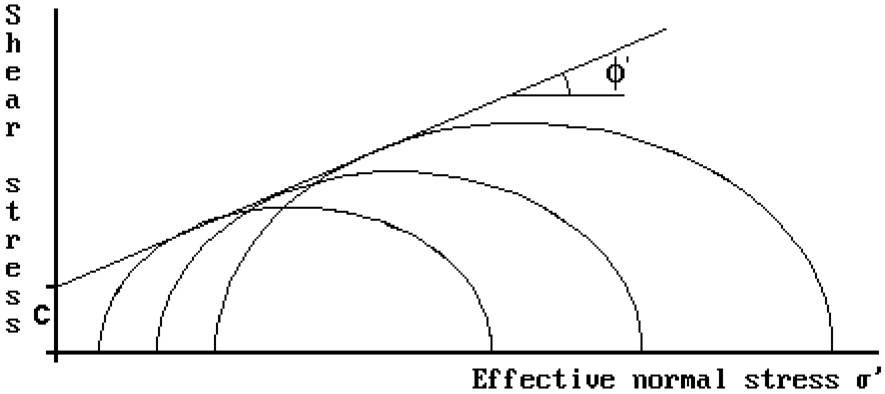

Fig. 4 Mohr's circles for a soil material with both cohesion, c, and friction, ϕ'.

From the angle of friction, ϕ', the failure angle, α, can be calculated with Equation 3 (Axelsson, 2005). This is the most probable angle the failure surface will take in a real triaxial test.

$$\alpha = 45 + \frac{\phi'}{2} \tag{3}$$

The more friction a soil material display, the higher the angle of friction, ϕ', will become, causing the failure angle, α, to grow. Generally, the higher the failure angle is the harder the material is.

Depending on the compression rate either undrained or drained conditions can be imitated in a triaxial compression test. The difference between a drained triaxial compression test and an undrained triaxial compression test is that in the drained case the compression is so slow that the pore water has time to drain away. If the speed is increased to a point beyond the capacity of the pore water to drain, undrained conditions apply. If this is used it is necessary to measure the pore pressure, because it will also be a factor to take into account. If the pore pressure is not measured the results might get misleading.

Drained triaxial compression tests are time consuming and complicated to carry out, in comparison with unconfined compression tests and therefore they are also very expensive to carry out. The cost is approximately ten times higher then for an unconfined compression test. To draw any conclusions two or preferably three test specimen is needed. Besides, only a few laboratories in Sweden are capable of performing triaxial compression tests. When used, it is mainly for research purposes, although these tests imitate natural conditions better then unconfined compression tests very few triaxial compression tests are performed on stabilized soil in Sweden. Consequently the experience in interpretation results from triaxial testing is limited.

3. INVESTIGATED SOIL AND BINDING AGENTS

3.1. INTRODUCTION

In this study one type of soil and three types of different binding agents were used. It is important to know the background of the material, because that might give hints on what to expect from the laboratory experiments.

3.2. SOIL

The soil was excavated in Uppsala, at the construction site of the new railway station. It would have been preferable to extract the soil from the ground with a piston sampler. But due to practical and economic factors, it was excavated by hand in an already existing shaft, at a depth of approximately 4 m. The outmost 20 centimetres of clay was discarded to avoid soil already dried due to contact with air. Three test tubes, with a diameter of 50 mm, were pressed down vertically into the soil. These were then excavated and sealed with rubber lids. The purpose of this was to enable the calculation of the bulk density of the soil and in addition to this enable strength tests of the undisturbed unmixed soil. A large amount of soil was excavated and placed in steel buckets of 25 litres. Every bucket was then closed with a metal lid equipped with rubber packing, which was then sealed with a clamp ring to avoid contact with air. At the laboratory the soil was stored in a climate room at 7 °C and 50 % atmospheric humidity. 7 - 8 °C is the average temperature in the ground in Sweden (Janz & Johansson, 2002), so by storing the soil at this temperature the microbial activity and the speed of chemical reactions are kept at natural levels.

3.3. BINDING AGENTS

Lime, cement and three types of fly ash were used in this project. The fly ashes are all produced in plants owned by Vattenfall AB. These fly ashes will henceforth be referred to as fly ash F, fly ash G and fly ash H.

3.3.1. Fly ash F

Fly ash F is a product of biomass fuel, a combination of peat and wood chips, combusted in a pulverized boiler. The fuel is pulverized before entering the furnace, where it is combusted at 800 - 900 °C. Dolomite, a type of limestone, is added in order to reduce the discharge of NO_x and SO_x in the flue gas. The ash was stored in a plastic sack placed in an open barrel at room temperature (approximately 20°C) where it was stored approximately six month before the laboratory experiments started.

3.3.2. Fly ash G

Fly ash G is a product of biomass fuel, bark and wood chips, combusted in a fluidized bed boiler at approximately 800°C. In the furnace the fuel is mixed with sand, oxygen is added from underneath, causing the sand and fuel to float. In order to reduce the discharge of NO_x and SO_x in the flue gas, dolomite is added. The ash was stored in a sealed plastic bucket at room temperature.

3.3.3. Fly ash H

Fly ash H is a product of hard coal combustion. The combustion takes place in a circulating fluidized bed boiler at approximately 800°C. In the furnace the fuel is mixed with sand, oxygen is added from underneath at high pressure, causing the sand and fuel to float. The ash was stored in a sealed plastic bucket at room temperature.

3.3.4. Lime

The lime is burnt lime (CaO) with a granular size of 0 - 0.1 mm, manufactured by Nordkalk AB. The chemical composition is shown in Appendix A

3.3.5. Cement

The cement is common construction cement, of the type CEM II/42.5 R, manufactured in Skövde, by Cementa AB. The chemical composition is shown in Appendix B

4. PERFORMED PHYSICAL AND CHEMICAL TESTS

4.1. TESTS ON SOIL AND BINDING AGENTS

The author of this report, a master thesis student at Vattenfall Research & Development AB, has performed the majority of the laboratory tests in this study. The only tests carried out externally were analyses of the chemical composition. An important part of the project was the development of laboratory methods for manufacturing of test specimens and compression tests.

The bulk density of the undisturbed soil is the key to calculate how much binder to add per cubic metre of soil. To know if moist has affected the binders before they are used, one good way is to measure the water content of the dry binders. If the water content is high, the binder quality is probably poor since water triggers the chemical reactions.

One way to explain the strength results of the compression tests is to look at the chemical composition of the binders, because chemical composition is one of the key factors in the stabilizing effect, knowing this can help to explain the results better. No tests were performed on the lime and the cement since all necessary information was supplied on their product information sheets, see Appendix A and B.

4.1.1. Bulk density of undisturbed soil

To test the bulk density of the undisturbed soil, the three tubes of undisturbed soil were used. The soil samples were pressed out of the test tubes and both ends were cut straight, the dimensions and weights were noted, and the bulk densities calculated. This was done at the same time as the triaxial compression tests, as these soil samples also were used in the triaxial compression tests.

4.1.2. Water content of unmixed soil

The water content of the soil was calculated in accordance with the standard SIS-CEN ISO/TS 17892-1:2005. Aluminium containers were weighed. After homogenisation of the soil, five test samples were taken and placed in the containers, and weighed again. The samples were placed in an oven at 105 °C for at least 16 - 24 h, and then weighed again. The water content, w, was calculated according to equation (4), where m_w is the mass of evaporated water and m_d is the mass of the dried sample of soil.

$$w = \frac{m_w}{m_d} \cdot 100 \quad [\%] \tag{4}$$

The temperature 105°C is high enough to evaporate all water not strongly bound to chemical compounds, yet it is low enough not to break down for example organic matter.

4.1.3. Water content of fly ash

The water content of the dry fly ashes was tested in the same way as the water content of the unmixed soil, see Subsection 4.1.2.

4.1.4. Chemical tests of fly ash

Samples of all the three types of fly ash were sent to a chemical analysis company (Analytica AB) to obtain the chemical composition of the material. Three tests were performed:

- MG2-analysis, which gives the content of oxides, metals and most elements.
- Content of $CaO_{reactive}$, the CaO free for chemical reactions.
- Content of TOC, total organic carbon. This gives a picture of how well combusted the material is.

4.2. TESTS ON STABILIZING EFFECT

In order to test the stabilizing effect of the binding agents, a testing program was established. This describes what parameters to vary, what ingredients to use and what testing methods to use. In this study unconfined compression tests were the most important test method, it was performed on all mixtures of binding agents. This was due to the fact that it is the standard method to test soil stabilization. Therefore the results from this test method are the ones to put most emphasis on when analysing the stabilization results. As a secondary laboratory study, a limited number of triaxial compression tests were performed to evaluate if it is a test method giving information, relevant for soil stabilization, compared to the high prize for its utilization.

4.2.1. Testing program

Examples of parameters that might be of interest to study are:

- Proportions between binding agents; lime, cement and fly ashes.
- Hardening time.
- Total amount of binding agents per cubic metre of soil.
- Type of soil.
- Type of testing method.

For obvious reasons, restrictions must be set on which parameters that should be investigated and which parameters that should not, otherwise the amount of tests would grow out of proportions. The main purpose of this study, as described in Chapter 1, was to evaluate the potential for fly ash to be used in soil stabilization projects, so the proportions between different binding agents was the most important parameter. Since the speed of chemical reactions vary depending on the type of binder, it is interesting to examine the development of the strength over time. Differences in soil composition affect the stabilizing effect, but since the testing program was large, only one soil was chosen. In total 31 batches were manufactured, with four test specimens in each batch. That equals a total of 124 test specimens.

Proportions of binding agents

Most often mixtures of two or more different binders were used. It might be hard to compare results between different studies due to different choices of binder mixtures. It is therefore very important to include a frequently used mixture as a reference, to enable comparisons with other studies. The most commonly used mixing proportion is, by weight, 50 % lime and 50 % cement (Edstam, 1997). This mixture was chosen as the reference mixture. There are several ways to vary the binder proportions, for example to replace a specific amount of lime with fly ash, or replace an amount of cement with fly ash, or replace an amount of both. The last alternative was chosen, with three levels of fly ash admixture, 30 %, 70 % and 100 %, resulting in four different mixtures (Lime/Cement/Ash in %):

- 50/50/0 (Reference)
- 35/35/30
- 15/15/70
- 0/0/100

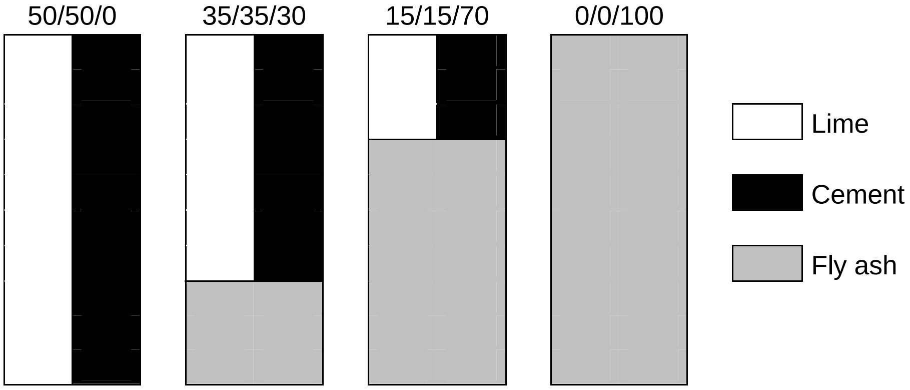

Fig. 5 The relationship between lime, cement and fly ash in the four different mixtures. The mixture 50/50/0 (left) consists of equal parts lime and cement and has no ash admixed. In the mixture 35/35/30 (second from left), 30 % of the lime and cement is replaced by ash. In the mixture 15/15/70 (third from left), 70 % of the lime and cement is replaced with ash. The mixture 0/0/100 (right) consists entirely of ash.

This is illustrated in Fig. 5. One reference mixture and three mixtures with fly ash were chosen. If a larger number of mixtures had been chosen the amount of test specimen would have been too extensive. The mixture covers the whole span of admixture from 0 % to 100 % fly ash. It should be possible to draw conclusions of the intervening mixtures, based on this material. All these four mixtures were tested for all performed hardening times and testing methods.

Hardening times

In order to obtain a good picture of the strength development over time, three different hardening times were chosen, 7, 28 and 56 days. It is expected that most of the strength development contributed by cement occurs before 28 days. Longer hardening times than 56 days would have been interesting due to the slow strength increase of fly ash, but this did not fit within the time frame of the study. Due to the late delivery of ash H there was no time for a test with 56 days hardening time for this ash, see Tab. 1. During hardening the test specimens were stored in a climate room at 7 °C and 50 % atmospheric humidity in order to make conditions as similar as possible to the natural conditions in the ground.

Amount of binding agents

The amount of binder added to the soil was 66 kg/m^3. This was chosen after performing a few initial tests on manufacturing test specimens.

Testing methods

Unconfined compression tests were conducted for all test series since this was the main part of the laboratory work. Triaxial compression tests were conducted only for the test series of ash F, after 28 days of hardening, see Tab. 1.

Tab. 1 Test types and hardening times for testing program, performed on all four mixtures.

Ash type	Test type	Hardening time [days]		
Ash F	Unconfined compression test	7	28	56
Ash G	Unconfined compression test	7	28	56
Ash H	Unconfined compression test	7	28	----
Ash F	Triaxial compression test	----	28	----

4.2.2. Manufacturing of test specimens

The manufacturing of test specimens was performed in accordance with Carlsten (2000). The density of the soil was used to calculate the mass of 1 dm^3 of soil, which was measured using an automatic scale. The soil was homogenized in a Hobart mixer for five minutes. The same apparatus is usually used for mixing cement and concrete, using a flat whisk, see Fig. 6. When mixing clay soils however, the soil sticks to the flat whisk, preventing homogenisation. A whisk shaped like a hook is better, so one was manufactured to optimize the homogenization process.

Fig. 6 Photograph of flat whisk (left) normally used for cement, hook shaped whisk (centre) better for mixing clay soils, Hobart mixer (right) used for mixing the soil and the binding agents.

The proper amount of binders were weighed and added to the homogenized soil, which then was kneaded for five more minutes. After adding the binders, the production of the test specimens must be completed within 30 minutes (Carlsten, 2000) or the material will harden too much. The containers for the test specimens were cylinders made of Plexiglass, 13 - 14 cm in height, with an inner diameter of 50 mm. The test specimens had a length of approximately 11 - 12 cm, to enable trimming of the ends before testing. The test specimens were made in layers, each approximately 3 cm thick. Each layer was compressed at a pressure of 100 kPa during 5 seconds, before the material of the next layer was applied. A fork was used to loosen up the surface of the newly packed layer to avoid distinct boundaries between the layers. Each cylinder was sealed with a broad electric tape, as it is very important that they are airtight. If air is allowed to come in contact with the test specimen, water will evaporate, which is not possible under natural conditions in the ground. The drier the test specimens get, the harder they become, which of course gives misleading results in later testing. All test specimens were stored in 7 °C during the entire hardening time. Each batch contains enough material to manufacture four test specimens. This gives a good chance of getting satisfying results, even if one or two test specimens are damaged. The material of the mixture 0/0/100 was very sticky and difficult to work with. The other mixtures were much easier to form into test specimens.

4.2.3. Unconfined compression tests

All the unconfined compression tests were conducted in accordance with the standard SIS-CEN ISO/TS 17892-7:2005. To prepare a test specimen for testing, it was pushed a few millimetres out of the Plexiglass tube. The end was then cut smooth along the edge of the tube. The test specimen was then pressed back to the other end of the tube, where that end also was trimmed before the tube was removed. According to the standard, the length of the sample should be as close as possible to two times the diameter, which here equals 100 mm. The test specimen was then weighed and the diameter and length was measured, to enable the calculation of the bulk density. The test specimen was placed between two platens, of which the top one was able to tilt. The compression speed was set to 2 mm/min.

The standard prescribes the use of an apparatus with a maximum load capacity of 7 kN. Instead one with the maximum load capacity of 20 kN was used, because this was the available apparatus with the lowest maximum load capacity. This has no significant impact on the results, due to the fact that the standard was developed for testing undisturbed soil samples, which are significantly weaker than the stabilized test specimens. The tests were performed in an apparatus from Hounsfield, which however, does not give a graph of the compression versus the compressive stress over time. It saves the maximum value and the corresponding compression is noted. If the test specimen has not been brought to failure before 15 % compression, the load at 15 % compression is to be recorded instead. Samples for testing the water content were taken from all test specimens, and handled as described in Chapter 4.1.2.

To get a picture of how the compression curve looks, four test specimens were taken aside from the test series of ash F, one from each of the four mixing types, and instead tested in another apparatus, from Instron. This records compression data during the whole process of compression. The drawback of this apparatus was that the pressure cell has a maximum load capacity of 300 kN, making the resolution very low. In spite of that, it gives a picture of what the failure would look like. These tests were performed after 57 days of hardening.

4.2.4. Triaxial compression tests

All drained triaxial tests were conducted at KTH (the Royal Institute of Technology) in Stockholm. Scarcity of working machinery forced the speed of compression to be increased to 0.25 mm/min, enabling four test specimens to be tested in one day. The test specimens were cut, measured and weighed in the same way as in the unconfined compression tests. The test specimens was then placed in a rubber membrane, and placed in a plexiglass cylinder, a triaxial cell. To obtain the desired lateral pressure σ_3 the cell was filled with water at a certain pressure. The rubber membrane then protect the test specimen from the water. The test was then started, and the pressure needed to compress the test specimen was recorded along with the compression.

5. RESULTS AND ANALYSIS OF PERFORMED TESTS

5.1. TESTS ON SOIL AND BINDING AGENTS

Bulk density, water content of the soil, and water content and chemical analyses of fly ash are displayed here. For complete results of chemical analysis of fly ash see Appendix C.

5.1.1. Bulk density of unmixed soil

The bulk density of the unmixed soil was calculated at the same time as the triaxial tests giving three different values: 1785.1 kg/m^3, 1708.7 kg/m^3, and 1716.3 kg/m^3. This results in an average density of 1736.7 kg/m^3.

5.1.2. Water content of unmixed soil

Results of the five water content tests were 53.47 %, 51.56 %, 53.80 %, 49.37 % and 51.53 %. The mean value of these was 51.9 %

5.1.3. Water content of fly ash

Ash F had a water content of 0.37 %, ash G 0.16 % and ash H 0.02 %. These figures are very low, which indicates that all three ashes have been stored in a good way, and that they have not been exposed to moist. As a comparison, it can be mentioned that another ash also was tested in the early stages of the project, but it was discarded from all further research due to the high water content of 20%. Water contents below 0.5 %, as seen in all three types of ash in the project, are in comparison very low.

5.1.4. Chemical analysis of fly ash

Complete chemical test results for all the three types of fly ash are available in Appendix C. Only the most important strength enhancing oxides are dealt with here. For the fly ashes, the values from the chemical tests for TOC (total organic carbon) and the most important strength enhancing oxides are shown in Tab. 2. In the same table the corresponding values for lime and cement are also given, taken from their respective product information sheets, see Appendix A and B. As can be seen in Tab. 2 ash H has the largest sum of important strength enhancing oxides among the ashes, while ash F and G are almost equal. The TOC values are all low, indicating that the combustion process has left very little material not completely combusted in the ash.

Tab. 2 The most important strength enhancing substances and TOC all values in % by weight of dry substance.

	Lime	Cement	Ash F	Ash G	Ash H
SiO_2	2.1	18.6	32.5	39.3	32.6
Al_2O_3	1	5	5.59	6.44	20.2
Fe_2O_3	0.5	2.8	10.8	2.38	5.87
Sum	3.6	26.4	48.89	48.12	58.67
CaO reactive	90	1	6.0	6.5	6.3
TOC			0.2	1.1	5.7

5.2. UNCONFINED COMPRESSION TESTS

In total 28 batches of test specimens were tested in the unconfined compression test. Every batch consists of four test specimens. In most cases, 22 batches, all four test specimens were also successfully tested. But in 6 of the total 28 batches only three specimens were successfully tested. The reason for this was that either they were damaged during trimming or the compression tests were failed. Not in any batch were there less then three test specimens successfully tested.

Complete test results for all test specimens in the unconfined compression tests are available in Appendix D The mean values of the compressive strength of the three to four test specimens in each batch were calculated. A table with all mean values are available in Appendix E. The mean compressive strengths of ash F are shown in Fig. 7, the mean compressive strengths of ash G are shown in Fig. 8 and the mean compressive strengths of ash H are shown in Fig. 9. The mean compressive strengths for all of the three types of ash are assembled in Fig. 10 to simplify comparisons between the different types of ash.

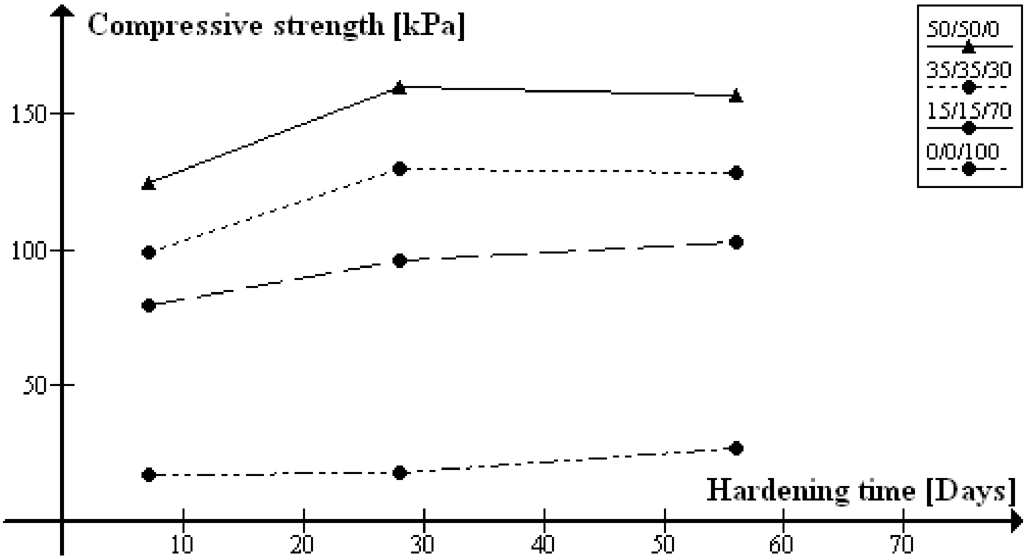

Fig. 7 The compressive strength of the unconfined compression tests for ash F related to the hardening time.

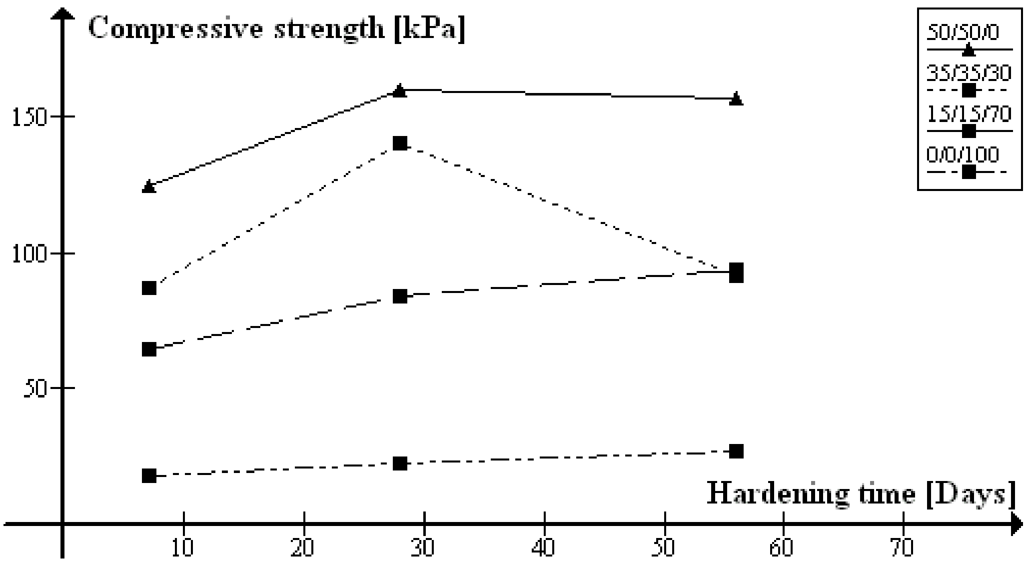

Fig. 8 The compressive strength of the unconfined compression tests for ash G related to the hardening time.

The results of ash G, see Fig. 8, of the mixture 35/35/30 are problematic to interpret. The strength of the four test specimens hardened for 56 days, are all four unexpectedly low. While the strength of the four test specimens hardened for 28 days are all unexpectedly high, none of those are reasonable and are not reliable. This leaves only the 7 days value to be fully reliable. It would be expected that the line of the mixture 35/35/30 of ash G, in Fig. 10, would follow the curve of the mixture 35/35/30 of ash F, but only slightly lower, like the curves of the mixtures 15/15/70 of the same ashes. The one hardened for 56 days was the first batch manufactured, so inexperience in the laboratory procedures might be the reason for this. The explanation for the batch hardened for 28 days is unknown.

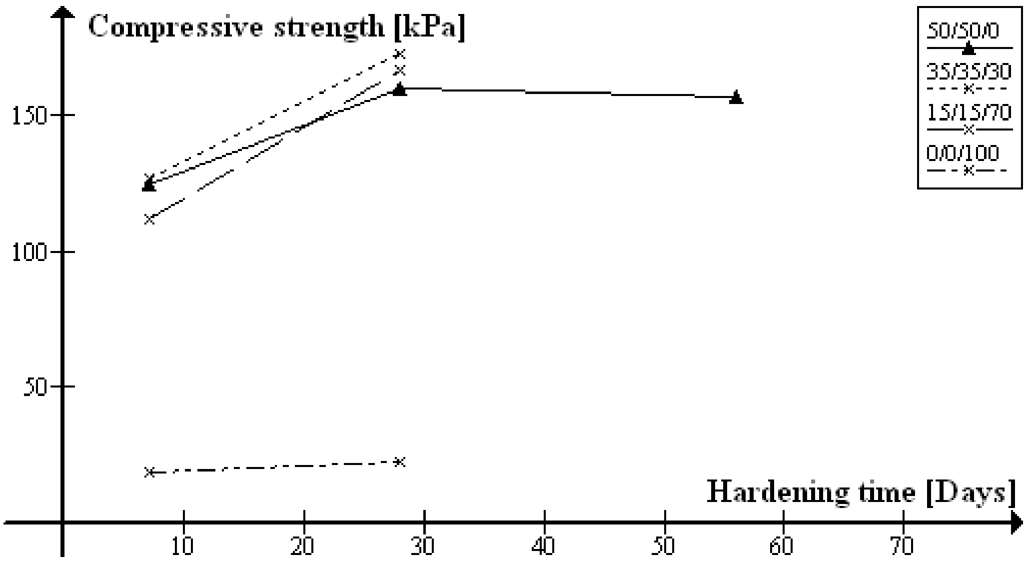

Fig. 9 The compressive strength of the unconfined compression tests for ash H related to the hardening time.

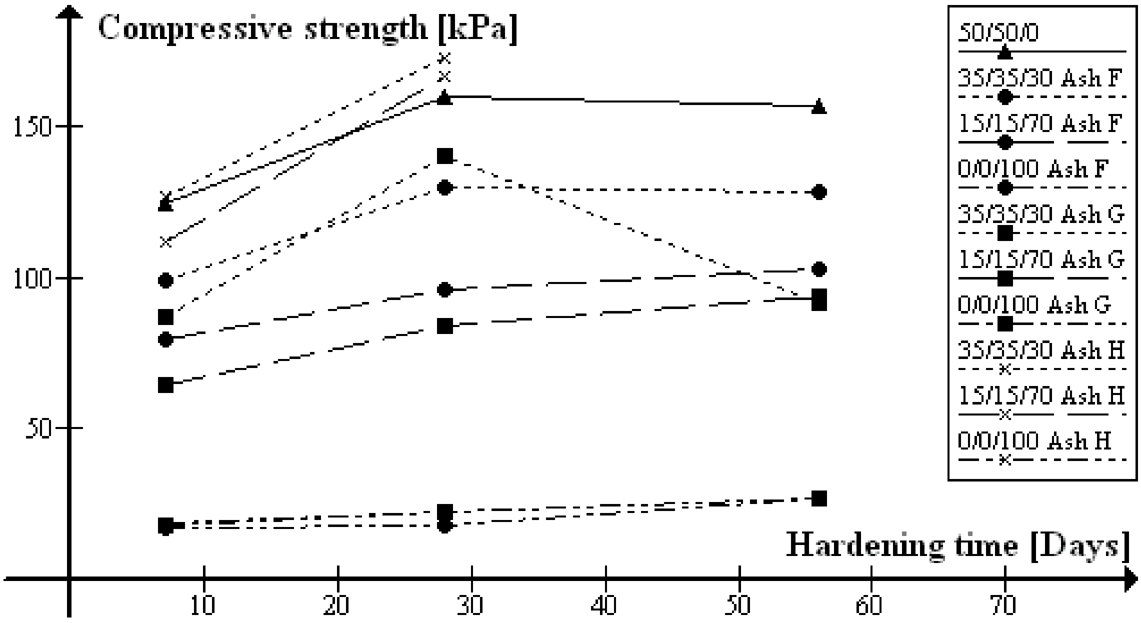

Fig. 10 The compressive strength of the unconfined compression tests for all ashes, plotted against the hardening time.

As can be seen from Fig. 10, the mixes of 0/0/100 of all ashes give very week results at all hardening times with a small, but slightly rising trend with time. No real difference between any of the ashes can bee noted for this mixture. For the 35/35/30 and the 15/15/70 mixtures there are distinct differences between the ashes. Among the bio ashes, ash F display a slightly higher strength than ash G, while the coal ash H give the strongest result of them all. After 28 days both the 35/35/30 and the 15/15/70 mixtures for ash H has passed the 50/50/0 reference mixture in strength.

If the strength values of the batches containing ash are divided by the corresponding 50/50/0 reference mixture value it is possible to see how much better or poorer the stabilization effect is in percentage. This is shown in Fig. 11. For the 0/0/100 mixture for all the three types of ash the strength stays constantly around 15 % of the strength of the 50/50/0 reference mixture. For ash F the value of the 35/35/30 mixture stays constantly at 80 % of the 50/50/0 mixture, while the 15/15/70 mixture stays around 65 %. Ash G stays on average 10 percentage units below ash F. Ash H displays the strongest results after 28 days, the 35/35/30 mixture has 108 % of the strength of the 50/50/0 mixture while the 15/15/70 mixture has achieved 105 %.

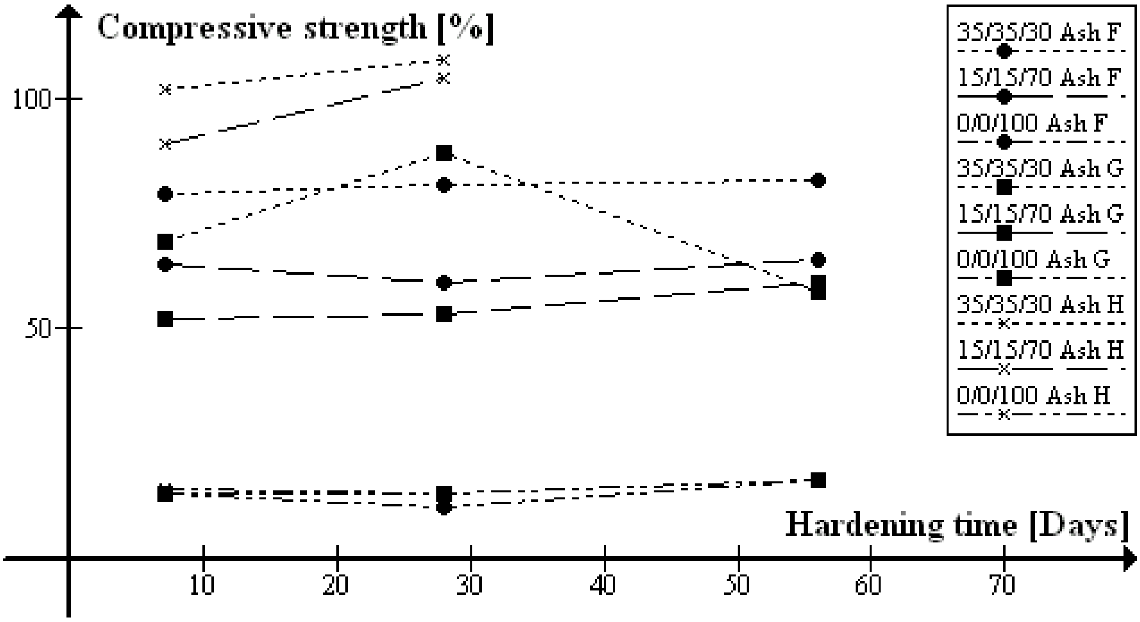

Fig. 11 The compressive strength in percent of the 50/50/0-values related to the hardening time.

The tests performed in the apparatus recording data continuously is shown in Fig. 12. Fig. 13 shows the same information but here the X-axis is cut so that the interesting part of the graph is better shown. The interpretation of these curves must be done with a bit of caution because these results are based on only one test specimen per mix. The resolution of the apparatus is low but it still gives a picture of what the compression curve would look like. The 0/0/100 mixture gives a soft material displaying a curve with no distinct top, see Fig. 12. Besides the fact that the 50/50/0 mixture is stronger than the 35/35/30 mixture, they display very similar failure curves.

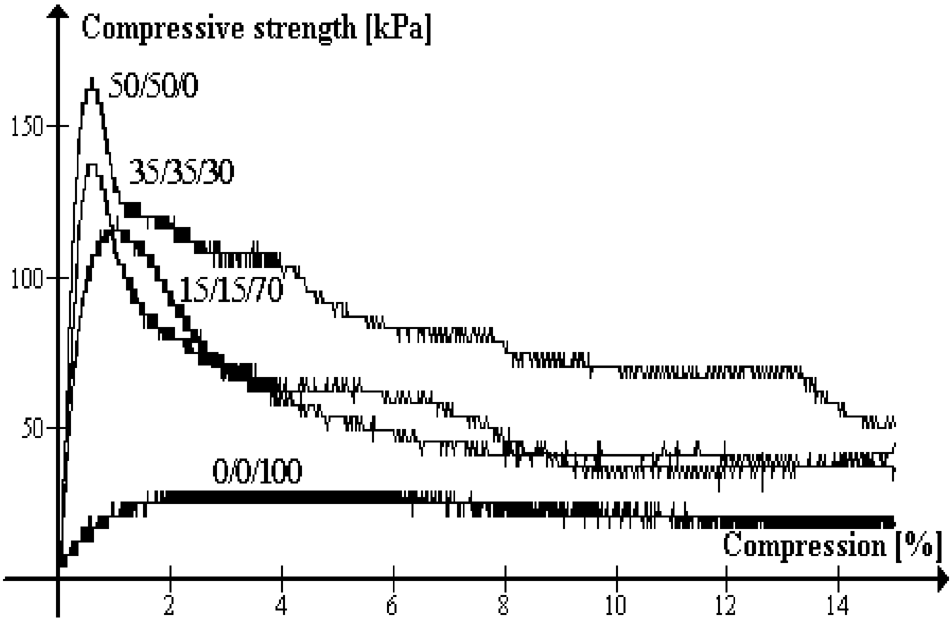

Fig. 12 The compressive strength plotted against the compression(0 - 15 %), for ash F..

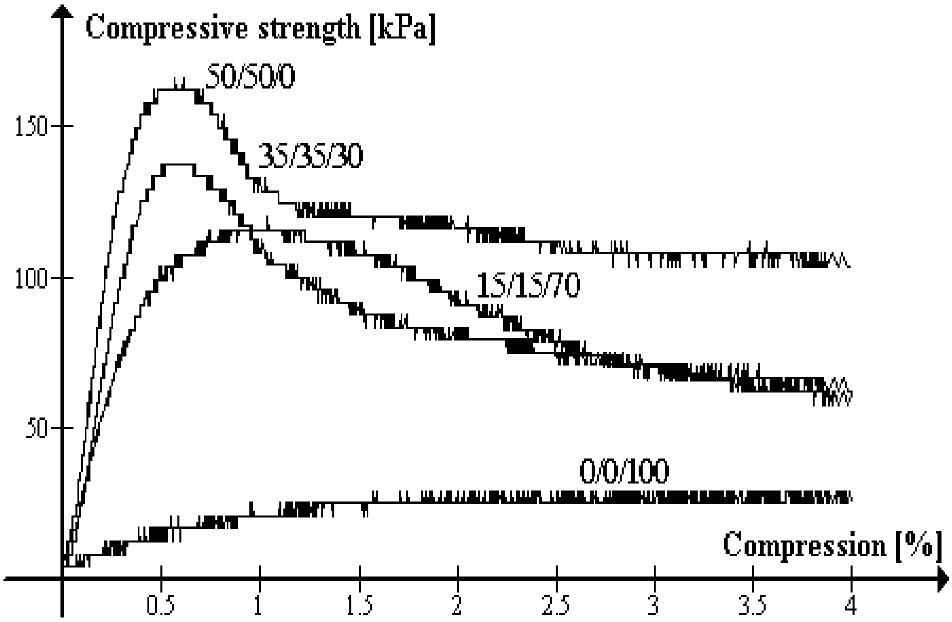

Fig. 13 The compressive strength plotted against the compression(0 - 4 %), for ash F.

5.3. STATISTICAL ANALYSIS OF UNCONFINED COMPRESSION TESTS

This study has worked with four identical test specimens for each test. If analysing the results of the unconfined compression tests of each test specimen, se Appendix D, it can be shown that in only two cases does the strength of the four test specimens vary more then +/- 10 %, maximum variation is +/- 20.2 %. In both cases of variation above +/- 10 %, it concerns the 0/0/100 mixtures, which have a very low strength, so variations amongst it make for greater figures. The mean variation among the test specimens in the same test is +/- 6.2 % and the lowest variation is +/- 1.4 %.

5.4. TRIAXIAL COMPRESSION TESTS

All triaxial compression tests were performed on mixtures of ash F. The tests on three of the four test specimens of 35/35/30 were not successfully performed due to faulty machinery. From the result of only one test specimen no safe conclusions can be drawn. During the rest of the triaxial test series, three out of four tests were successful. This is enough to use the Mohr's circles to construct a Mohr's envelope in the Mohr plane, see Fig. 14 and Fig. 18.

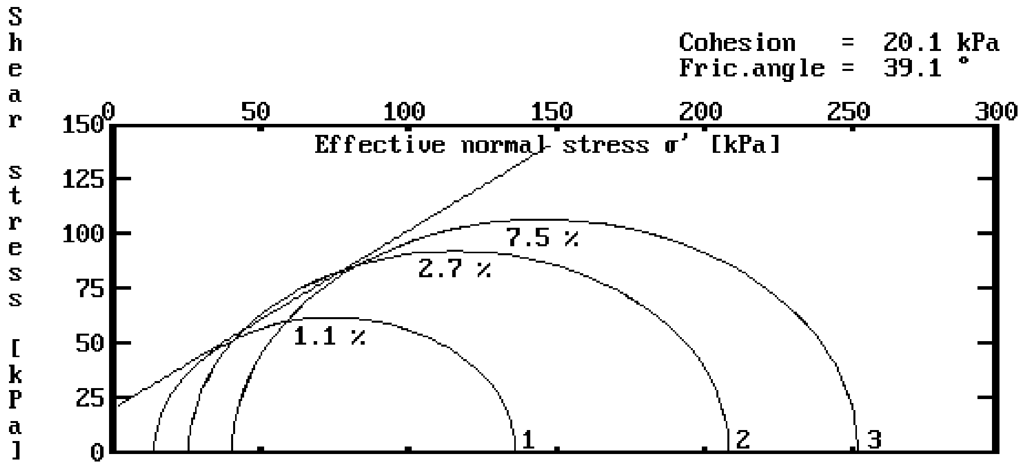

Fig. 14 Mohr's circles for the 50/50/0 mixture.

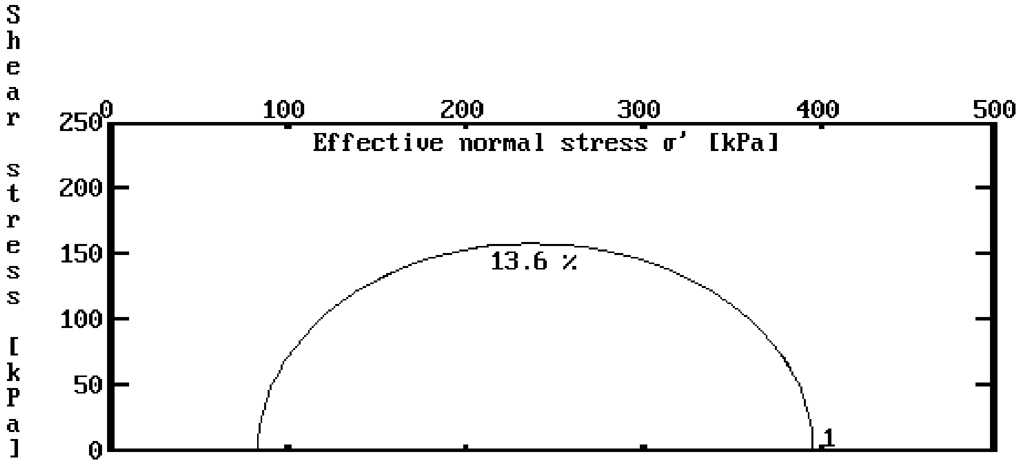

Fig. 15 Mohr's circles for the 35/35/30 mixture.

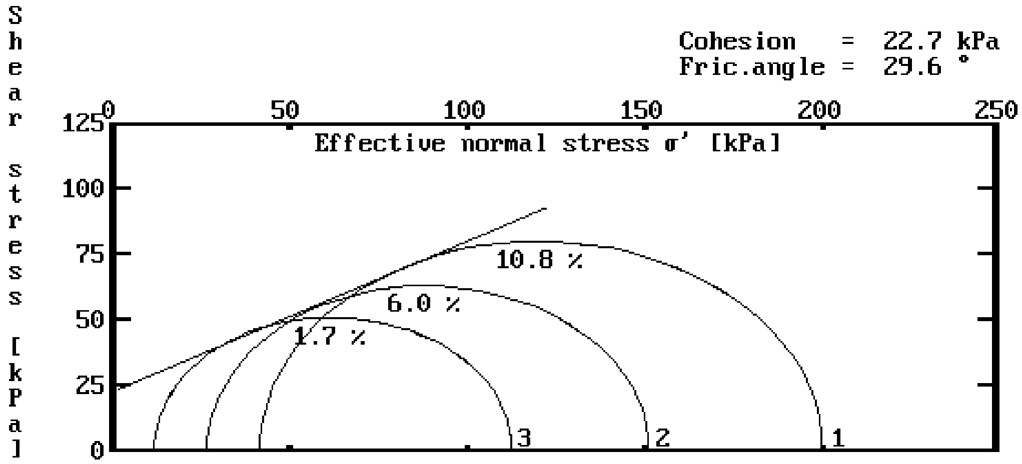

Fig. 16 Mohr's circles for the 15/15/70 mixture.

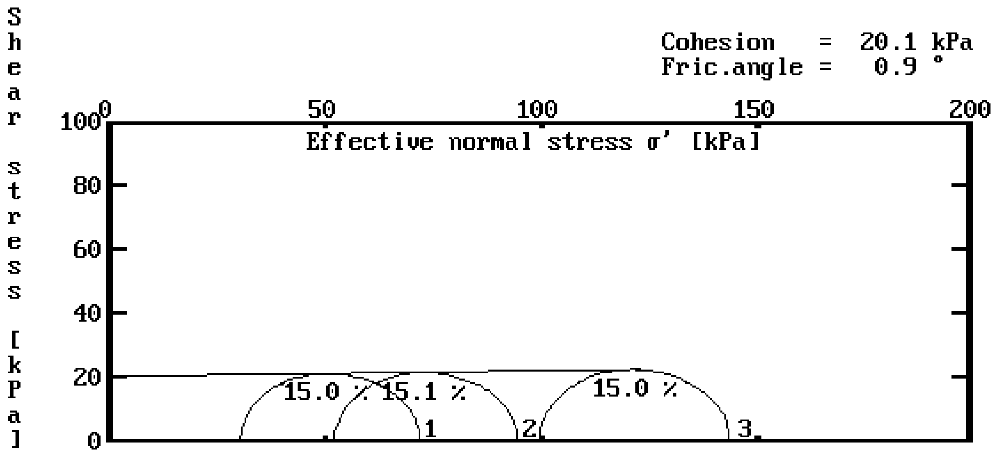

Fig. 17 Mohr's circles for the 0/0/100 mixture.

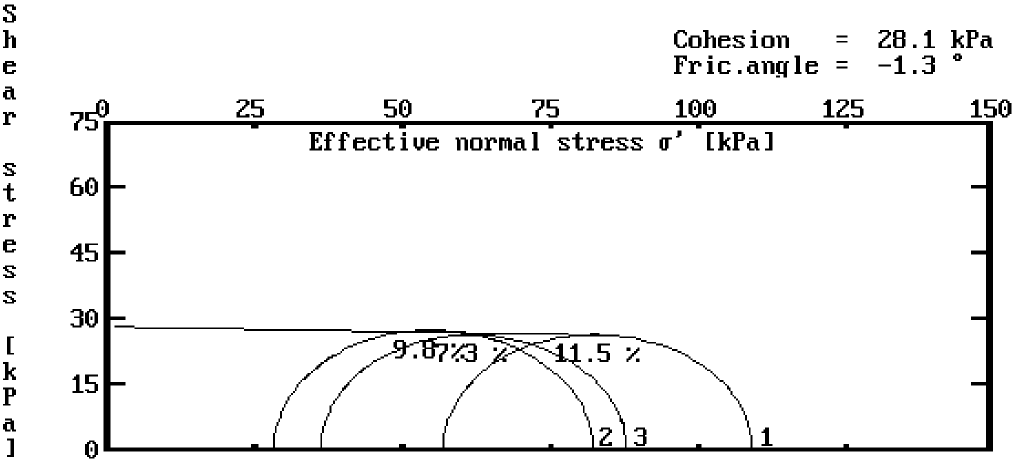

Fig. 18 Mohr's circles for undisturbed soil.

The cohesion and angle of friction are displayed in Tab. 3. If the same cohesion can be assumed for the single test specimen of the 35/35/30 mixture as for the rest of the triaxial tests on stabilized soil (approximately 20 kPa), the angle of friction can be determined to 36°. These results are illustrated in Tab. 3. The angle of friction was used to calculate the failure angle according to equation B, these results are also displayed in Tab. 3.

Tab. 3 The measured cohesion and friction, and the calculated failure angle.

	Cohesion [kPa]	**Angle of friction [°]**	**Failure angle [°]**
50/50/0	20.1	39.1	64.6
35/35/30	*20**	*36**	*63**
15/15/70	22.7	29.6	59.8
0/0/100	20.1	0.9	45.5
Undisturbed soil	28.1	-1.3	44.4

*Calculated values based on only one test specimen and the assumption that the cohesion is 20 kPa.

When studying the results of the triaxial compression tests, it is obvious that the increase in compression speed was too large. A normal clay soil in Sweden has a friction angle of around 30 , and if the speed were adequately slow, so that the pore water had time to drain, this would be the expected approximate result. In this case, however, the water has not had time to drain, making the tests undrained. Since the pore pressure was not measured, the results are probably not entirely reliable. So conclusions from the triaxial compression tests should be made with caution.

6. DISCUSSION

6.1. INTRODUCTION

Discussing the compressive strength of the test specimens in absolute terms, kPa, is difficult, mainly because so many factors affect the resulting compressive strength. When comparing results with other studies, many factors may vary. The most important ones in this case are the amount of binding agent added to the soil, the proportions of different binders and the type of soil used. Since very little work has been done on fly ash, particularly fly ash from bio fuel, it is very difficult to find other relevant studies to compare the results with. Some studies exist but they have usually not tested the same proportions of binders, or have used different soils. This makes it hard to compare the results directly and it is therefore wise to limit direct comparisons to within the same study.

6.2. UNCONFINED COMPRESSION TESTS

The bio ashes F and G display similar results, which, however, differ significantly from the results of the hard coal ash H. Therefore the bio ashes and the hard coal ash are discussed separately.

6.2.1. Reference mixture, 50/50/0

When studying the line of the 50/50/0 mixture in Fig. 10, it can be seen that the largest increase in strength comes before 7 days of hardening. This large increase is mainly due to the cement, which has very fast hardening reactions. This increase continues between 7 and 28 days of hardening, but at a lower rate. Between 28 days and 56 days, there is no rise in strength. This shows two things, firstly that the contribution to strength development from cement seems to be negligible after 28 days, and secondly the strength development from pozzolanic reactions are negligible from this mixture. This means both that the soil in itself does not have enough pozzolanic material to influence the results and that the cement and lime do not seem to contribute with any pozzolanic material. This is a very important conclusion, because it establishes that there is a need for fly ash, because pozzolanic material is something that fly ash can contribute with.

6.2.2. Fly ash from bio fuel combustion

For the 0/0/100 mixtures of both of the bio ashes, the strength increase is very low. Fig. 10 shows that the strength is slightly higher after 56 days than after 7 days implying that some pozzolanic reactions do take place. Yet, without the contribution of the initially fast cement reactions the strength is still extremely low. The reason why the 0/0/100 mixture is not a successful mixture could be explained by chemistry. The pozzolanic reactions are dependent on both access to water, pozzolanic material and the content of CaO, and probably there are simply not be enough CaO to react with. This shows that a certain amount of CaO from cement or lime is necessary. It would be interesting to test admixtures between 70 and 100 % ash to see where the turning point is. One other possibility for the unsuccessful stabilization could lie in the very low amount of binding agents added. If the amount of binder added to the soil, currently 66 kg/m^3, is doubled or tripled a notable stabilizing effect might be manifested.

When looking at the curves in Fig. 10 for the 35/35/30 and 15/15/70 mixtures, it is easy to think that because the overall strength is lower than in the case of the 50/50/0 mixture, those mixtures are not good. But when examining this a bit more closely, some important things do show the contrary. The first thing to note is that the more bio ash that is admixed the larger the increase between 28 and 56 days is, showing that here the slow pozzolanic reactions contribute to the strength increase. After 28 days, the strength of the 35/35/30 mixtures has achieved approximately 80 % of the strength achieved by 50/50/0. The 15/15/70 mixture has after the same time achieved 50 - 60 % of the strength achieved by the 50/50/0 mixture, which is shown in Fig. 11. Yet as can be seen in Fig. 10, the strength is still slowly rising, while the strength of the 50/50/0 mixture has ceased to grow.

The compressive strength results of ash F display a slightly stronger response then those of ash G. The reason for this is hard to know; it could be related to the different combustion processes, the different fuels or the combustion temperature. Those factors affect both the chemical composition and the surface area of the ash particles. Whatever the reason, it can be concluded that ash F displays slightly better results then ash G, though this does not in any way imply that ash G is not usable for stabilization purposes. Both ash F and G seem very suitable to be used as binding agents. To substitute 30 % of the lime and the cement with fly ash and still maintain 80 % of the strength should in many stabilisation projects be considered fully acceptable.

6.2.3. Fly ash from hard coal combustion

It is expected that the hard coal ash H should display stronger response than that from the bio ashes, as discussed in Subsection 2.1.2, yet it is stronger than could be expected. Because of the limited time available for the master thesis project and due to the late delivery of this ash, hardening times longer than 28 days could not be achieved. Yet, as pozzolanic reactions are slow, longer hardening times would have been desirable. On the strength development over a longer period of time one can only speculate, yet it should continue to grow, but possibly with a more moderate rate since the pozzolanic reactions are slow.

After 7 days, the 35/35/30 mixture of ash H is already stronger than the 50/50/0 mixture, and after 28 days the 15/15/70 mixture has also passed the 50/50/0 mixture in strength. This must be considered outstanding, that when substituting up to 70 % of lime and cement with a secondary product material the results are stronger then when using only lime and cement. When studying the 0/0/100 mixture it is obvious that the result for this mixture is equivalent with the results of the same mixture for the other ashes. The question that arises is how much of the lime and cement that could be substituted with ash and still get a result that is adequately strong. Somewhere between 70 and 100 %, lays a threshold value when the resulting strength will plummet, but above this, there is a span where the results are still good. Finding this is a thing for future investigations.

6.3. TRIAXIAL COMPRESSION TESTS

As mentioned in Subsection 4.2.4 the triaxial tests were not performed under drained conditions, and without measuring the pore pressure. Therefore interpreting what the results imply is somewhat difficult. A friction angle should have been visible in Fig. 17 and Fig. 18, if the tests had been performed correctly. Since triaxial compression tests were conducted as a side study, and the main testing method to evaluate the strength was the unconfined compression tests, this has no significant impact on the outcome of the study as a whole.

When a test is performed under drained conditions the testing time should be several hours, preferably a day. When testing unmixed soil samples it works fine to test the test specimens of a triaxial test at different points in time, but when testing materials that harden over time it is important that all test specimens in the same triaxial test are tested at the same time. If for example there is only one triaxial testing apparatus and, four test specimens are tested, one each day for four days, the difference in hardening time between the first and the last test specimen to be tested is so big that it can affect the results of the test. So if triaxial compression tests are to be performed for soil stabilization it is essential that there is enough testing machinery to test all the test specimens in the test at the same time. In this study there was only one testing apparatus working, making this impossible.

The triaxial compression tests have been performed for ash F only, so the results cannot be copied directly for the other ashes. However, the general trend can probably be applied for both the bio ashes, but probably not for the coal ash, because of the very different results obtained from the unconfined compression tests.

What can be said concerning the test results of Tab. 3, is that the cohesion of the undisturbed soil samples seems significantly higher than the cohesion of the stabilized test specimens. This is probably due to the fact that the mixing breaks up the structure of the clay soil, resulting in a lower contribution to strength from the cohesion. The 50/50/0 and 15/15/70 mixtures display a friction angle as seen in Tab. 3. The question is how much higher it should be if the pore pressure had been measured. Both the undisturbed soil samples and the test specimens of the 0/0/100 mixture are very wet and sticky. The test specimens of the 50/50/0, 35/35/30 and 15/15/70 mixtures, however, felt significantly drier. It is possible that there is much less pore water to drain, which in other words could mean that the results of those tests are not entirely incorrect after all. There is one more thing supporting this. The successful test of the mixture 35/35/30 was tested over an entire day. So if the same cohesion is assumed for this test as for the rest of the mixed materials, 20 kPa, as shown in Tab. 3 a friction angle can be calculated. This friction angle matches those from the tests of the 50/50/0 and 15/15/70 mixtures.

A problem in deep soil stabilization, in practice, is that the columns tend to become too strong and hard, due to the desire to be on the safe side with the behaviour of conventional stabilizing mixes. This makes it more similar to a pile, and prevents the interaction with the surrounding soil. A softer and slightly weaker material could in many cases be advantageous. As shown in Tab. 3, if more lime and cement are substituted with fly ash, the friction angle decreases and the material becomes slightly weaker and softer, which is also illustrated in Fig. 12 and Fig. 13. If the strength decrease is not too extensive, the slightly softer material is an advantage worth examining further.

6.4. FURTHER DISCUSSION

It would be valuable to carry out stabilization studies on several types of soils. Examples of interesting soils are clay soils deposited under marine conditions, from the west cost of Sweden, organic soils, like gyttja or peat, clay soils deposited in inland condition, or clay soils from the east coast of Sweden, other then the one from Uppsala in order to see how it affects the stabilizing result. In order to compare results with other studies, and gain acceptance for fly ashes as binding agents on the market and by the big companies in the business, it is necessary to perform testing in very typical soils, preferably soils commonly used by other companies in development of binding agents.

Since cement reactions are fast and pozzolanic reactions are very slow, longer hardening times than 56 days are desirable to better show the advantages of fly ash. 90 days is a common hardening time, but as long as half a year or even up to a year might prove interesting. Here, economic factors might put a limit on the time frame, because in industrial development, results may be needed in shorter time spans. To make a project more effective, tests on fewer hardening times or better chosen ones might be a way to free time and resources for a more extensive testing program. 7 days give a good insight on the initial development of strength but it gives no valuable information on the final long-term result, while 56 days is a bit to short too see the long-term effects. For example, 28 and 91 days or 28 and 140 days might be good choices.

As mentioned earlier, this study has worked with four test specimens for each test. In most studies, however, only two test specimens are used. This is also the recommended number in the guidelines for laboratory testing (Carlsten, 2000). If every mixed batch still produces four test specimens as recommended in the standard, but only two are tested at each given time, each batch will give results for two different hardening times at the same cost as one when using four test specimens. As most of the test specimens vary less then +/- 10 % in strength they seem to hold a homogeneous level of quality, which shows that four test specimens might be more than what is strictly needed. Yet, a problem arises if one of only two test specimens is damaged, then there are only one test specimen left, and that is not a desired situation. But as seen in this study, quite few test specimens were damaged. Though two test specimens might be a bit insecure, the recommendation for further laboratory work is to use only two test specimens for each test.

The amount of binder added to the soil, 66 kg/m^3, used in this project, can be considered very low compared to most other studies. This of course affects the stabilization achieved, causing it to be lower. The more binding agent added the more stabilizing reactions will take place, and the stronger the material will get. If a larger amount of binder was used, the curves in Fig. 10 would have been steeper and it would be easier to see the effects of stabilization. If not enough binding agents are used, the result is that a load-bearing skeleton cannot form, as mentioned by Janz & Johansson (2002), causing the stabilization to be very low. This is probably what happened when using the 0/0/100 mixtures. This does not, however, exclude the possibility to use 100 % ash as a binding agent, but it might require a very large amount of binder to be used.

7. CONCLUSIONS

7.1. CONCLUSIONS OF THE PROJECT

According to the test results obtained in this study, deep soil stabilization, using fly ash as a binding agent, seems to work fine. 100 % ash, however, is not a successful mixture, since some lime and cement are necessary. Among the three ashes, the coal ash H is the most promising, because substitution of more then 70 % of lime and cement with ash can be done and still a stronger material is obtained than if only lime and cement is used. Both ash F and G also work well for soil stabilization. Admixture of 30 % seems most promising, since a substitution of 30 % of the lime and cement with ash still maintains 80% of the strength developed using only lime and cement. More research is necessary before these three ashes can be used in a real stabilization project. Some improvements to the laboratory experiments can be made, these are described in the following section.

7.2. SUGGESTIONS FOR FURTHER RESEARCH

Before using ash in soil stabilization applications it is necessary to know the effect it will have on the environment. An important aspect to look at is for example leaching. Other interesting aspects, apart from the possibly negative effects of leaching are the benefits to the environment from using recycled materials instead of newly produced ones. Here LCA (Life cycle assessment) is a great tool. A market analysis of the possibilities to introduce ash on the market might also be interesting.

This study has given a good overview of the properties of the studied ashes, but if these ashes are to be used in real stabilization projects, more extensive and detailed laboratory studies must be conducted. Some things in the laboratory methods exist to develop, in order to improve the results, some examples of this are:

- Use larger amounts of binder then 66 kg/m^3, somewhere above 100 kg/m^3 should be more appropriate.
- Perform tests after fewer but longer hardening times. 28 and 91 days or 28 and 140 days are better to give justice to the slow pozzolanic reactions of the fly ash. Even longer hardening times could be of interest.
- Decrease the amount of test specimens in each test from four to two test specimens. This would double the amount of tests performed at the same cost, without much risk of loosing accuracy in the results.
- Use other types of soils, especially soil types similar to those commonly used in development of binding agents by other companies or institutions. This would greatly improve the possibilities to compare results with other studies.

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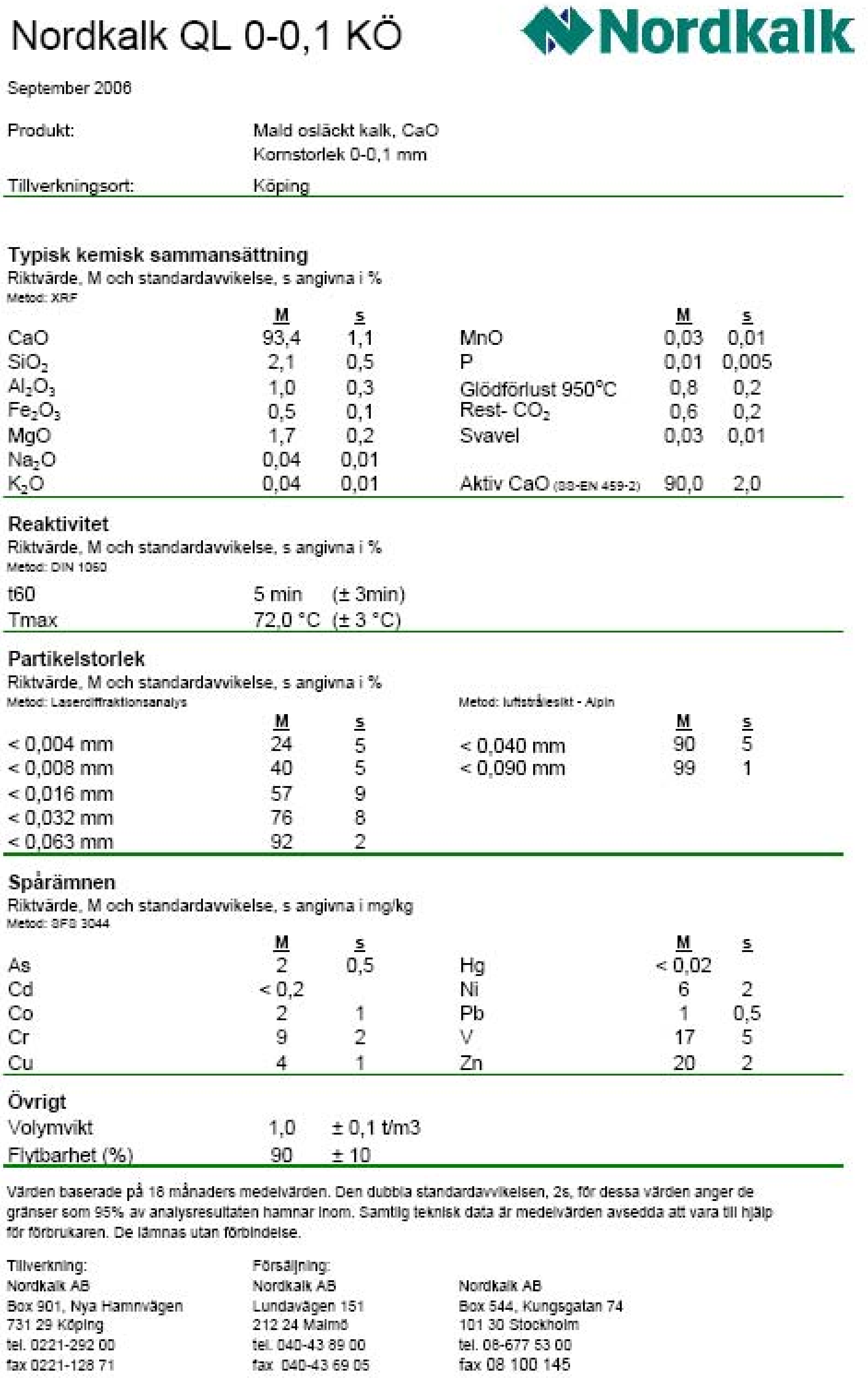
Appendix A - Product information sheet for lime



Nordkalk QL 0-0,1 KÖ

Nordkalk

September 2006

Produkt:	Mald osläckt kalk, CaO Kornstorlek 0-0,1 mm
Tillverkningsort:	Köping

Typisk kemisk sammansättning

Riktvärde, M och standardavvikelse, s angivna i %

Metod: XRF

	M	s		M	s
CaO	93,4	1,1	MnO	0,03	0,01
SiO_2	2,1	0,5	P	0,01	0,005
Al_2O_3	1,0	0,3	Glödförlust 950°C	0,8	0,2
Fe_2O_3	0,5	0,1	Rest- CO_2	0,6	0,2
MgO	1,7	0,2	Svavel	0,03	0,01
Na_2O	0,04	0,01			
K_2O	0,04	0,01	Aktiv CaO (SS-EN 459-2)	90,0	2,0

Reaktivitet

Riktvärde, M och standardavvikelse, s angivna i %

Metod: DIN 1060

t60	5 min	(± 3min)
Tmax	72,0 °C	(± 3 °C)

Partikelstorlek

Riktvärde, M och standardavvikelse, s angivna i %

Metod: Laserdiffraktionsanalys	M	s	Metod: luftstrålesikt - Alpin	M	s
< 0,004 mm	24	5	< 0,040 mm	90	5
< 0,008 mm	40	5	< 0,090 mm	99	1
< 0,016 mm	57	9			
< 0,032 mm	76	8			
< 0,063 mm	92	2			

Spårämnen

Riktvärde, M och standardavvikelse, s angivna i mg/kg

Metod: SFS 3044

	M	s		M	s
As	2	0,5	Hg	< 0,02	
Cd	< 0,2		Ni	6	2
Co	2	1	Pb	1	0,5
Cr	9	2	V	17	5
Cu	4	1	Zn	20	2

Övrigt

Volymvikt	1,0	± 0,1 t/m3
Flytbarhet (%)	90	± 10

Värden baserade på 18 månaders medelvärden. Den dubbla standardavvikelsen, 2s, för dessa värden anger de gränser som 95% av analysresultaten hamnar inom. Samtlig teknisk data är medelvärden avsedda att vara till hjälp för förbrukaren. De lämnas utan förbindelse.

Tillverkning:
Nordkalk AB
Box 901, Nya Hamnvägen
731 29 Köping
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Försäljning:
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Nordkalk AB
Box 544, Kungsgatan 74
101 30 Stockholm
tel. 08-677 53 00
fax 08 100 145

Typanalys 2005

Byggcement Std PK Skövde

CEM II/A-LL 42,5 R

KEMISK ANALYS

	Medelvärde	
CaO	61,7	%
SiO_2	18,6	%
Al_2O_3	5,0	%
Fe_2O_3	2,8	%
MgO	1,2	%
Na_2O	0,12	%
K_2O	1,3	%
SO_3	3,7	%
Cl	0,01	%
Vattenlöslig kromat	< 2	mg/kg
Vithet R46	26,4	%

TRYCKHÅLLFASTHET

	Medelvärde	
1 dygn	24,2	MPa
2 dygn	35,4	MPa
28 dygn	55,5	MPa

ÖVRIGA FYSIKALISKA DATA

Vattenbehov	28,0	%
Bindetid	135	min
Volymbeständighet	1,1	mm
Specifik yta	435	m^2/kg
Densitet	3088	kg/m^3

ÖVRIGA UPPLYSNINGAR

Kalksten	9,7	%
C_3A	7,9	%

KORNSTORLEKSFÖRDELNING

125	µm	100,0	%
63	µm	95,1	%
32	µm	75,1	%
15	µm	46,9	%
8	µm	29,2	%
5	µm	19,1	%
3	µm	10,6	%
2	µm	5,4	%
1	µm	0,64	%

Appendix C - Results of chemical analysis of fly ash

Element	Fly ash F	Fly ash G	Fly ash H	Units
TS	99.9	99.8	100	%
SiO2	32.5	39.3	32.6	% TS
Al2O3	5.59	6.44	20.2	% TS
CaO	23	23.3	17.5	% TS
Fe2O3	10.8	2.38	5.87	% TS
K2O	0.675	5.78	1.89	% TS
MgO	11.7	3.39	3.42	% TS
MnO	0.234	1.44	0.0934	% TS
Na2O	0.244	1.85	0.94	% TS
P2O5	0.999	2.67	0.563	% TS
TiO2	0.124	0.289	0.818	% TS
Total	85.9	86.8	83.9	% TS
LOI	10	8.2	7.5	% TS
As	14.7	<4	9.68	mg/kg TS
Ba	982	2040	1620	mg/kg TS
Be	3.21	0.818	8.36	mg/kg TS
Cd	0.898	9.66	0.586	mg/kg TS
Co	12.1	9.52	38.9	mg/kg TS
Cr	55.3	45.7	130	mg/kg TS
Cu	41	74.1	186	mg/kg TS
Hg	0.284	0.736	0.503	mg/kg TS
La	75.5	20.6	54.4	mg/kg TS
Mo	10.5	<6	7.04	mg/kg TS
Nb	<6	<6	14.8	mg/kg TS
Ni	40.8	32.1	114	mg/kg TS
Pb	23	55.1	101	mg/kg TS
S	7780	12200	28500	mg/kg TS
Sc	9.86	4.79	25.7	mg/kg TS
Sn	<20	<20	<20	mg/kg TS
Sr	258	756	974	mg/kg TS
V	55.8	33.6	244	mg/kg TS
W	<60	<60	<60	mg/kg TS
Y	103	15.4	63.4	mg/kg TS
Zn	106	1110	168	mg/kg TS
Zr	84.7	136	165	mg/kg TS
TOC	0.2	1.1	5.7	% TS
CaO Aktiv	6.0	6.5	6.3	%

Appendix D - Result of unconfined compression tests, all test specimens

Concentrations [%]					Time	Dens	W	Comp	Strength [kPa]	
Lime	Cement	F	G	H	[d]	[Kg/m2]	[%]	[%]	qu	Cu
50	50	0	0	0	7	1655.5	44.9	1.33	130.9	65.4
50	50	0	0	0	7	1670.8	45.1	1.10	123.8	61.9
50	50	0	0	0	7	1647.7	45.0	1.31	121.5	60.7
50	50	0	0	0	7	1668.7	44.8	1.09	123.0	61.5
50	50	0	0	0	28	1646.4	44.9	0.97	142.9	71.4
50	50	0	0	0	28	1691.3	44.8	1.02	169.8	84.9
50	50	0	0	0	28	1665.6	44.1			
50	50	0	0	0	28	1659.2	44.8	0.97	167.0	83.5
50	50	0	0	0	56	1634.1	46.9	0.80	163.0	81.5
50	50	0	0	0	56	1641.4	46.2	0.75	156.6	78.3
50	50	0	0	0	56	1624.1	46.9	0.80	152.0	76.0
50	50	0	0	0	56					
35	35	30	0	0	7	1665.6	46.0	1.03	102.9	51.5
35	35	30	0	0	7	1666.7	45.7	1.14	102.5	51.3
35	35	30	0	0	7	1653.2	45.9	1.00	91.7	45.8
35	35	30	0	0	7	1663.9	45.9	1.14	99.5	49.7
35	35	30	0	0	28	1662.9	47.2	1.27	138.8	69.4
35	35	30	0	0	28	1635.5	47.8	1.14	130.4	65.2
35	35	30	0	0	28	1640.6	47.9	0.94	120.5	60.2
35	35	30	0	0	28	1637.4	47.7	1.00	129.8	64.9
35	35	30	0	0	56	1636.4	47.4	0.79	138.3	69.1
35	35	30	0	0	56	1619.4	47.8	0.55	114.8	57.4
35	35	30	0	0	56	1613.0	47.5	0.80	133.1	66.5
35	35	30	0	0	56					
15	15	70	0	0	7	1668.0	46.7	1.59	83.1	41.5
15	15	70	0	0	7	1671.2	46.8	1.33	80.3	40.2
15	15	70	0	0	7	1662.0	47.0	1.58	74.7	37.4
15	15	70	0	0	7	1670.8	46.7	1.53	80.0	40.0
15	15	70	0	0	28	1673.6	48.8	1.90	98.5	49.3
15	15	70	0	0	28	1658.0	48.2	1.26	98.3	49.1
15	15	70	0	0	28	1671.8	48.1	1.40	95.1	47.6
15	15	70	0	0	28	1650.5	48.2	0.94	90.7	45.3
15	15	70	0	0	56	1649.0	47.5	0.89	109.0	54.5
15	15	70	0	0	56	1641.1	47.7	1.24	94.1	47.0
15	15	70	0	0	56	1629.4	47.6	0.84	104.7	52.4
15	15	70	0	0	56					
0	0	100	0	0	7	1660.6	48.2	12.71	17.1	8.5
0	0	100	0	0	7	1679.6	48.6	15.00	18.2	9.1
0	0	100	0	0	7	1695.7	48.2	15.00	16.5	8.2
0	0	100	0	0	7	1708.6	48.4	15.00	17.5	8.7
0	0	100	0	0	28	1680.5	51.0	15.00	22.4	11.2
0	0	100	0	0	28	1654.9	51.5	5.91	18.9	9.5
0	0	100	0	0	28	1672.9	50.9	3.81	14.9	7.4
0	0	100	0	0	28	1670.4	49.6	2.68	16.1	8.1
0	0	100	0	0	56	1694.4	49.1	3.17	26.7	13.4
0	0	100	0	0	56	1673.2	48.9	2.88	26.9	13.5
0	0	100	0	0	56					
0	0	100	0	0	56	1707.0	48.6	3.42	26.2	13.1

35	35	0	30	0	7	1636.1	50.6	1.54	82.3	41.2
35	35	0	30	0	7	1642.5	49.9	1.64	88.6	44.3
35	35	0	30	0	7	1635.9	49.7	1.49	88.7	44.3
35	35	0	30	0	7	1665.3	50.0	1.41	87.2	43.6
35	35	0	30	0	28	1685.5	47.1	1.00	143.1	71.6
35	35	0	30	0	28	1667.0	47.1	1.14	142.4	71.2
35	35	0	30	0	28	1664.1	47.0	0.89	136.7	68.3
35	35	0	30	0	28	1673.6	47.7	1.04	140.6	70.3
35	35	0	30	0	56	1584.9	44.4	0.75	87.1	43.6
35	35	0	30	0	56	1578.4	44.1	0.65	98.5	49.2
35	35	0	30	0	56	1576.5	44.8	0.60	95.1	47.6
35	35	0	30	0	56	1562.1	44.6	0.74	84.6	42.3
15	15	0	70	0	7	1681.6	48.3	2.04	66.0	33.0
15	15	0	70	0	7	1683.5	49.0	2.14	67.2	33.6
15	15	0	70	0	7	1683.2	48.3	1.69	67.5	33.8
15	15	0	70	0	7	1654.5	48.6	1.50	57.8	28.9
15	15	0	70	0	28	1662.2	47.2	1.90	85.8	42.9
15	15	0	70	0	28	1672.9	47.7	1.79	77.9	39.0
15	15	0	70	0	28	1659.2	47.8	1.47	89.9	45.0
15	15	0	70	0	28	1672.8	48.2	1.35	82.4	41.2
15	15	0	70	0	56	1620.3	49.1	1.00	88.8	44.4
15	15	0	70	0	56	1618.4	48.7	1.30	93.3	46.7
15	15	0	70	0	56	1571.0	48.9	0.80	93.4	46.7
15	15	0	70	0	56	1643.6	49.1	1.11	100.0	50.0
0	0	0	100	0	7	1705.1	50.4	15.00	18.1	9.1
0	0	0	100	0	7	1706.1	49.4	15.00	18.0	9.0
0	0	0	100	0	7	1774.2	49.6	15.00	17.9	9.0
0	0	0	100	0	7	1698.2	49.8	15.00	17.0	8.5
0	0	0	100	0	28	1717.2	48.5	2.23	21.9	11.0
0	0	0	100	0	28	1694.3	49.0	3.02	20.8	10.4
0	0	0	100	0	28	1703.1	48.7	2.40	21.6	10.8
0	0	0	100	0	28	1703.1	49.5	7.79	23.6	11.8
0	0	0	100	0	56	1703.7	50.8	2.39	29.5	14.7
0	0	0	100	0	56	1674.6	50.1	2.79	30.1	15.1
0	0	0	100	0	56	1694.1	50.2	3.25	26.5	13.3
0	0	0	100	0	56	1642.0	50.4	3.02	22.3	11.1
35	35	0	0	30	7	1670.1	49.0	1.64	122.7	61.4
35	35	0	0	30	7	1656.6	50.0			
35	35	0	0	30	7	1188.6	49.0	1.00	132.9	66.4
35	35	0	0	30	7	1667.2	48.9	1.49	125.3	62.6
35	35	0	0	30	28	1657.5	47.7	0.89	175.9	88.0
35	35	0	0	30	28	1671.1	48.1	1.06	168.5	84.2
35	35	0	0	30	28	1645.0	48.6	0.98	178.2	89.1
35	35	0	0	30	28	1639.2	48.3	0.80	168.8	84.4
15	15	0	0	70	7	1633.3	50.3	1.39	109.3	54.7
15	15	0	0	70	7	1648.3	50.4	1.15	116.1	58.0
15	15	0	0	70	7	1661.8	50.7	1.24	112.0	56.0
15	15	0	0	70	7	1634.6	50.4	1.34	110.1	55.1
15	15	0	0	70	28	1638.2	49.1	0.79	168.3	84.2
15	15	0	0	70	28	1635.4	49.0	0.90	173.0	86.5

15	15	0	0	70	28	1643.1	48.7	1.00	166.1	83.0
15	15	0	0	70	28	1646.6	48.7	0.80	160.4	80.2
0	0	0	0	100	7	1666.7	50.5	15.00	18.0	9.0
0	0	0	0	100	7	1679.8	50.3	12.43	18.6	9.3
0	0	0	0	100	7	1689.5	50.8	12.35	18.8	9.4
0	0	0	0	100	7	1702.1	51.2	15.00	20.4	10.2
0	0	0	0	100	28	1718.7	49.0	5.47	20.1	10.1
0	0	0	0	100	28	1697.4	49.4	4.40	22.9	11.5
0	0	0	0	100	28	1716.2	49.5	7.71	23.3	11.6
0	0	0	0	100	28	1707.5	55.0	6.35	23.6	11.8

Appendix E - Result of unconfined compression tests, mean values

Concentrations [%]					Time	Dens	W	Comp	Strength [kPa]	
Lime	Cement	F	G	H	[d]	[Kg/m2]	[%]	[%]	qu	Cu
50	50	0	0	0	7	1660.7	44.9	1.21	124.8	62.4
50	50	0	0	0	28	1665.7	44.9	0.99	159.9	79.9
50	50	0	0	0	56	1633.2	46.7	0.78	157.2	78.6
35	35	30	0	0	7	1662.3	45.9	1.08	99.1	49.6
35	35	30	0	0	28	1644.1	47.6	1.09	129.9	64.9
35	35	30	0	0	56	1622.9	47.6	0.71	128.7	64.4
15	15	70	0	0	7	1668.0	46.8	1.51	79.6	39.8
15	15	70	0	0	28	1663.5	48.3	1.37	95.7	47.8
15	15	70	0	0	56	1639.8	47.6	0.99	102.6	51.3
0	0	100	0	0	7	1686.1	48.4	14.43	17.3	8.7
0	0	100	0	0	28	1669.7	50.8	6.85	18.1	9.0
0	0	100	0	0	56	1683.8	49.0	3.03	26.8	13.4
35	35	0	30	0	7	1645.0	50.0	1.52	86.7	43.3
35	35	0	30	0	28	1672.5	47.2	1.02	140.7	70.3
35	35	0	30	0	56	1575.5	44.5	0.69	91.3	45.7
15	15	0	70	0	7	1675.7	48.6	1.84	64.6	32.3
15	15	0	70	0	28	1666.8	47.7	1.63	84.0	42.0
15	15	0	70	0	56	1613.3	48.9	1.05	93.9	46.9
0	0	0	100	0	7	1720.9	49.8	15.00	17.8	8.9
0	0	0	100	0	28	1704.4	48.9	3.86	22.0	11.0
0	0	0	100	0	56	1678.6	50.4	2.86	27.1	13.5
35	35	0	0	30	7	1545.6	49.2	1.38	127.0	63.5
35	35	0	0	30	28	1653.2	48.2	0.93	172.9	86.4
15	15	0	0	70	7	1644.5	50.5	1.28	111.9	55.9
15	15	0	0	70	28	1640.8	48.9	0.87	166.9	83.5
0	0	0	0	100	7	1684.5	50.7	13.69	18.9	9.5
0	0	0	0	100	28	1709.9	50.7	5.98	22.5	11.2

Appendix F - Result of unconfined compression tests, mean values in %

Concentrations [%]					Time	Dens	W	Comp	Strength [kPa]	
Lime	Cement	F	G	H	[d]	[Kg/m2]	[%]	[%]	qu	Cu
35	35	30	0	0	7	100	102	89	79	79
35	35	30	0	0	28	99	106	110	81	81
35	35	30	0	0	56	99	102	91	82	82
15	15	70	0	0	7	100	104	125	64	64
15	15	70	0	0	28	100	108	139	60	60
15	15	70	0	0	56	100	102	127	65	65
0	0	100	0	0	7	102	108	1194	14	14
0	0	100	0	0	28	100	113	695	11	11
0	0	100	0	0	56	103	105	387	17	17
35	35	0	30	0	7	99	111	126	69	69
35	35	0	30	0	28	100	105	103	88	88
35	35	0	30	0	56	96	95	88	58	58
15	15	0	70	0	7	101	108	152	52	52
15	15	0	70	0	28	100	106	165	53	53
15	15	0	70	0	56	99	105	134	60	60
0	0	0	100	0	7	104	111	1241	14	14
0	0	0	100	0	28	102	109	392	14	14
0	0	0	100	0	56	103	108	366	17	17
35	35	0	0	30	7	93	110	114	102	102
35	35	0	0	30	28	99	107	95	108	108
15	15	0	0	70	7	99	112	106	90	90
15	15	0	0	70	28	99	109	88	104	104
0	0	0	0	100	7	101	113	1133	15	15
0	0	0	0	100	28	103	113	607	14	14

All values in % of reference value (50/50/0)