

# **STABILISATION OF A HIGHLY PLASTIC CLAY SOIL FOR THE PRODUCTION OF COMPRESSED EARTH BLOCKS**

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A dissertation submitted to the faculty of Architecture, University of the Witwatersrand, Johannesburg, in fulfillment of the requirements for the degree Master of Science in Building Management.

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## Declaration

I declare that this dissertation is my own, unaided work. It is being submitted for the Degree of Master of Science in Building Management in the University of the Witwatersrand, Johannesburg. It has not been submitted before for any degree or examination in any other university.



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19<sup>th</sup> day of NOVEMBER 1998

## Abstract

Clay plays an important role in the suitability of soil for use in construction. The affinity of a clay for moisture makes it susceptible to shrinkage and swelling on drying and wetting and therefore problematic in terms of construction, where stability is required for the integrity of a structure to remain intact. It is therefore important to have methods of dealing with problematic clays in order to allow clays to be used as a construction material. One of the methods is known as soil stabilisation, where the properties of a soil and mainly the clay fraction are altered to make it more suitable for its intended purpose.

The thesis investigates the use of a highly plastic clayey soil for the production of building blocks. Due to the clayey nature and its associated properties the soil it is not suitable in its natural state. Soil stabilisation is investigated as a method to alter the properties of the soil. The principles of soil stabilisation are presented as an introduction to the laboratory testing programme.

Combinations of lime, cement and fly ash are investigated as a stabilising agent. They are investigated to determine which combination alters the properties of the soil to such an extent, that a compressed soil block produced using the combination and the clayey soil, would perform satisfactorily according to accepted standards. Combinations were designed so that comparisons could be made between blocks to determine what effect the addition of a certain stabilising agent would have on the performance of the block in terms of the tests conducted.

The engineering properties of the untreated soil which are not considered suitable for use in compressed soil blocks, are significantly enhanced by the addition of the stabilising agents. The test conducted on the blocks indicate that the addition of 4% lime, 20% fly ash and 7% cement added to the soil produces a

block that satisfies the criteria adopted for the research and that are generally in accord with international standards.

A mineralogical analysis was also undertaken in an attempt to identify changes in the mineralogy of the soil, and particularly the clay, as a result of the addition of the three stabilising agents. This analysis was done using x-ray diffraction. Due to the nature of the clay mineral and possibly the poor crystal structure thereof, the results were disappointing. No significant change in the clay was observed to correlate with the results of the improved performance as indicated by the physical tests.

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# CHAPTER ONE

## **1.0 INTRODUCTION**

Thirty percent of the world's population, or nearly 1 500 000 000 human beings, live in a home of unbaked earth. Roughly 50% of the population of developing countries, the majority of rural populations, and at least 20% of urban and sub-urban populations live in earth homes. Confronted with a twin crisis of economy and energy the industrialised countries have started a dialogue on the revival of earth as a building material and are backing research programmes and developing applications. The United States have officially recognised the use of adobe and rammed earth by integrating these construction techniques into national and regional standards. France has outlined the areas in which research into earth as a building material should be concentrated in the coming years. The budget devoted to research into these priority areas is close to 24 million francs, 83% of which will go into operational research and training. Programmes of research have also been started in Germany, Switzerland, and Belgium.

In the developing countries, construction in earth appears to be an effective means of building homes in the short term so that the greatest number of people can be housed, while at the same time encouraging the use of local resources for building materials, the training of building technicians and craftsmen, and the creation of jobs. A material is not attractive in itself but rather in terms of what it can do for society as a whole. [45]

### **1.1 Need for housing**

The demand for homes in the developing countries at the present time is immense. Recent statistics ( U.N. housing conference 1994 ) show that no less than 36 000 000 homes must be built by the year 2000 for the urban population of Africa alone. The backlog of housing in South Africa has been variously estimated as more than one million units.

## **1.2 Need for alternative construction methods and materials.**

In this age, where the natural resources are being consumed at an increasing rate, it is important to consider alternative materials and processes for the production of building materials. To solve the problem of supplying adequate housing in South Africa in order to solve the housing shortage new construction materials must also be considered, to determine if these materials can provide a cheaper alternative to conventional building materials. The use of earth and by-products from manufacturing processes must also be considered. The use of by-products can be particularly advantageous, as the materials will not require 'mining', quarrying and processing as is the case in many of the conventional building materials. This will also alleviate the need for disposal of these materials, which also has economic ramifications.

The use of earth as a construction material is in no way a new idea and has been used extensively throughout the history of mankind. However, it is regarded as an inferior building material by many who have no experience of using it but who favour more "modern" and "sophisticated" materials such as kiln-fired clay bricks, or concrete masonry units. The reason for the distrust of soil as a building material is possibly humanity's slavish adherence to fashion on the one hand and the *ad hoc* approach that has been adopted to the use of soil-based building materials on the other. [60]

## **1.3 History of earth as a construction material**

Ever since man first congregated in villages almost 10 000 years ago, unbaked earth has been one of the principal building materials used in every continent. Over one third of the world's population still lives in earth houses today. In ancient times, unbaked earth was widely used in Mesopotamia and Egypt, while later on, Romans and the Muslims built in earth in Europe, Africa and the Middle East – as did the peoples of the Indus civilizations, Buddhist monks and Chinese

emperors. During the Middle Ages, construction in unbaked earth was practiced not only in Europe, but in North America by the Indians, in Mexico by the Toltecs and the Aztecs, and in the Andes by the Mochica. The Spanish conquerors of America took with them European techniques of earth architecture and grafted them onto traditions already established there. In Africa, this art was mastered in cultures as diverse as those of the Berbers, the Dogons, and Ashanti, the Bamilikés or the Haoussas, in the kingdoms of Ife and the Dahomey and in the Empires of Ghana and Mali. [16]

Throughout the world, archaeological remains survive of cities built entirely of raw earth [62] : history's earliest city, Jericho, begun almost 10 000 years ago; Catal Hüyük in Turkey; Harappa and Mohenjo-Daro in Pakistan; Akhlet-Aton in Egypt; Chan-Chan in Peru; Babylon in Iraq; Zucheros near Cordoba in Spain; and Khirokitia in Cyprus. Very often, modern cities have been established on the sites of these ancient towns, and many recent buildings are also constructed out of raw earth. Examples of this include 'Lugdunum', the capital of Roman Gaul which later became the city of Lyon, and many of the cities born out of the Spanish conquest of the Americas, such as Santa Fe, the capital of New Mexico, and Bogotá, the capital of Columbia. This tradition has been maintained in a number of cities in Africa and the Middle East: Kano in Nigeria, Agadez in Niger, Tombouctou in Mali, Oualata in Mauritania, Marakesh in Morocco, Adar in Algeria, Ghadames in Libya, Sa'dah in North Yemen, Shibah in South Yemen and Yazd in Iran. Earth architecture in rural communities has been still more widespread and varied. Many familiar examples are to be found in Asia, Africa, and the Middle East and Latin America, while those in Europe and the USA are less well known. There are thousands of earth buildings both in the dry regions such as Spain and Italy and in the rainy countries like England, Germany, Denmark and Sweden. In France, at least 15 percent of rural buildings are still made of raw earth, and numerous examples survive near Lyons, Reims, Grenoble, Toulouse, Rennes, Avignon and even Chartres, close to Paris.

Unbaked earth has been used for thousands of years not only in rural housing, but also for the vast prestigious monuments that reflect the material and spiritual development of communities – warehouses and aqueducts, ziggurats and pyramids, monasteries, churches and mosques. In these, mankind's creative drive seemed to reach its full expression. In the 7<sup>th</sup> century BC even the famous Tower of Babylon was built of earth, its seventh level teetering at 90m – mankind's first skyscraper. That record breaking creation succumbed to the social chaos for which it has become the symbol, but the Great Wall of China, of which large parts are of unbaked earth, has managed to survive since the 3<sup>rd</sup> century BC, and testifies – as do many other monuments – to the enduring strength of this material. It is on account of these qualities that cities throughout the world have been protected by unbaked earth defences – from as early as the walls of Jericho, until 1882, when Tiznit, in Morocco, became perhaps the last city to be provided with earth ramparts.

Military strategists, too, have from ancient times recognized the virtues of earth as a strong and resistant building material. The Roman historian Pliny the Elder relates how in Spain, in about 219 BC, Hannibal constructed buildings out of unbaked earth; just as the American army used it during the Second World War for the erection of dams, military camps and airstrips.

It is not only the world's decision-makers who appreciate the architectural qualities of unbaked earth, for the greater majority of earth buildings have been, and still are, residential houses. These buildings come in a surprising variety of forms, each indicative of the cultural characteristics of their builders. All over the world, from Scandinavia to South Africa, earth constructions have been adapted to suit the most diverse types of climate. [15]

## **1.4 Earth as a construction material**

The success of soil construction is in the material itself. Soil is a building material, which is abundant throughout the world and available on most potential building sites. Soil, having proved itself as a successful building material throughout the world [1,21,29,39,46,53] may therefore offer a cheap alternative, which can be supplied at a rate equal to consumption. An essential component in soil is clay. Clay is a cohesive material that serves to bind sand particles together. Not every soil has sufficient clay content for construction purposes and some soils may contain no clay. These soils can therefore not be used in manufacturing rammed earth or soil bricks. [66]

On the opposite extreme, a soil with a high clay content also creates certain problems in the manufacture of soil based building materials. Clays tend to have a high affinity for water, which causes them to expand and contract with changes in moisture content (ibid.). Soils containing too much clay tend to fragment when dry because of the close pattern of cracking they develop. [53]

Soils contain various combinations of the components which make up a soil (i.e. gravel, sand, silt and clay). These components provide soils with the varying properties which they possess. A thorough understanding of a soil based on its properties is therefore required when considering it as a construction material to ensure a quality end-product.

## **1.5 Soil stabilisation**

The properties of a soil determine its suitability as a construction material. Depending on the type of soil and its intended use as a construction material, a soil may have properties which make it suitable as a construction material. However a soil will often have properties which do not make it suitable as a construction material in this case, a technique known as soil stabilisation can be applied to the soil. Soil stabilisation involves the modification of those properties

of a soil, which may make it unsuitable for construction purposes. The modification of the properties is due to changes in the mineralogy and structure of the soil brought about by the addition of stabilisers.

Therefore with a thorough understanding of a soil and its properties, combined with soil stabilisation techniques, a soil can be successfully used as a construction material.

This report considers the use of a clayey soil, which is a by-product of a water purification process as a potential construction material. The objectives of the research are:

- I. By application of soil science and soil stabilisation principles to produce compressed earth blocks which can be used in the construction of a building.
- II. A study of the mineralogical changes which take place in the soil as a result of the stabilisation process.

## 1.6 SOIL

Soil is a stage in a long process of deterioration of the parent rock and its physico-chemical evolution. Depending on the parent rock and climatic conditions soil appears in a myriad of forms possessing a myriad of characteristics.

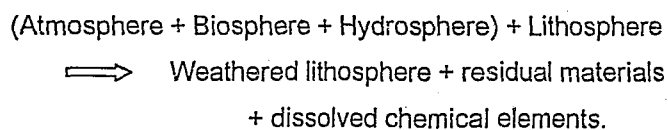
### **1.6.1 Formation of Soils**

In general, in-situ soils may be regarded as the uncemented mantle of disintegrated and decomposed rock material differentiated into horizons from the parent rock material below, in morphology and physical properties. In a broad sense, soil formation is the result of a vast geological cycle occurring continuously and relentlessly throughout the geological time scale. The three principal phases of the cycle are erosion, transportation and deposition, and earth movement. [54]

### **1.6.2 Physical and chemical weathering**

Exposed rocks are eroded and degraded by the physical and chemical process of weathering. The weathered material is transported to new locations by wind, water and glaciers. It may, finally, be deposited as soil material in various landforms. The atmosphere, biosphere and hydrosphere are the reactors in a vast chemical process with the surface of the lithosphere to produce the earth's crust. [46]

Each reactor consists of a complex chemical process that is intimately associated with the physical process. The reaction is cited as: [75]



Weathering is at its highest intensity where the interfaces between the hydrosphere, biosphere and lithosphere overlap, e.g. upper soil zones in the temperate humid climatic regions and deeper soil zones in the humid tropics.[75] The factors controlling the process of weathering of rock are the characteristics of parent rock, topography, climatic conditions, vegetation and time. The processes involved in the formation of soil by weathering of rocks include oxidation, carbonation, hydrolysis and hydration. Weathering is a spontaneous reaction involving geologic material and energy and is a change towards the decrease of the free energy of the system. [46]

### Physical weathering

Physical weathering takes place when solid rocks are broken into fragments with little or no chemical change in the rock itself. Mechanical disintegration yields mineral components of soils. Temperature change, by itself, independently can disintegrate rocks into fragments by exfoliation. Rocks may also be broken and degraded by wedging action of ice and spreading roots. Water, wind and moving ice (glaciers), although they play their primary role in the transport of soil material from one location to another, are powerful agents in scouring and eroding rock formations. The size and shape of fragmented products are influenced by structure as well as texture of the parent rock material and cementing agents[63]. In igneous and metamorphic rocks the grain size and packing influence the disintegration process. Due to the thermal expansion of the constituent minerals, in coarse-grained granular rock, disintegration is more rapid than in one of finer grains.

### Chemical weathering

Apart from the physical degradation, surface chemical reactions between rocks and environment might take place. In chemical weathering, original rock minerals are transformed into new minerals by chemical reactions. There are two types of

chemical weathering. If chemical reactions are inorganic, it is regarded as geochemical. Another type is pedochemical weathering, reflecting the biological activity as a result of organisms and organic solutions reacting with rocks [63].

The magnitude of chemical weathering is dependent on the amount of water with or without electrolytes that is available to leach rocks and on the drainage characteristics of the resultant soil minerals. Silica is slightly soluble at all pH values, whereas alumina is soluble only below pH 4 and above 8.5. Water with its content of cations and anions is the reagent that causes hydrolysis, a reaction between a salt and water to give an acid and a base. Clays, laterites and bauxites are formed by hydrolysis. Other chemical processes of decomposition are carbonation in which minerals are attacked by carbonic acid, and oxidation in which oxygen combines with the minerals of rock.

The climate of any region conditions the flora that grows there. The growth and decay of flora provides raw organic matter. This, through the action of micro-flora and chemical processes, produces soil humus. The presence of flora and the reaction of its products reacting with the mineral particles of weathered rocks changes geochemical weathering in to pedochemical weathering and affects the consequent development of soils.

## **1.7 CONSTITUENTS OF SOIL**

Soil is made up of three main constituents, namely gaseous, liquid and solid phases. The gas constituents being primarily air, the liquid mainly water and the solids primarily mineral and organic materials.

### **1.7.1 Gaseous constituents**

These form the internal atmosphere of the soil. They fill the voids in the soil and come from the outside atmosphere, organic life in the soil and the decomposition of organic material. Air's components include nitrogen, oxygen, and carbon dioxide. Gases resulting from organic decay, and respiration of living creatures

include carbon dioxide, carbon monoxide, hydrogen, and methane. There may also be traces of sulphur dioxide, hydrogen sulphide and a range of other organic gases and vapours.

### **1.7.2 Liquid constituents**

These are basically aqueous solutions of salts. These constituents are soluble in water and come from the rain and atmospheric conditions (mist, relative humidity), mankind, the weathering of rock and decay of organic material. The liquid constituents are water, soluble substances dissolved in this water such as organic compounds (sugars, alcohols, organic acids) and mineral compounds (acids, bases and salts partly dissociated into ions  $\text{Ca}^{++}$ ,  $\text{Mg}^{++}$ ,  $\text{K}^+$ ,  $\text{Na}^+$ ,  $\text{PO}_4^{3-}$ ,  $\text{SO}_4^{2-}$ ,  $\text{CO}_3^{2-}$ ,  $\text{NO}_3^{2-}$ , etc.).

### **1.7.3 Solid constituents**

The solid fraction of soil is made up of minerals resulting from the physical dissociation and chemical alteration of the parent rock material, and organic materials, which are basically the more or less decomposed remains of plant and animal organisms. [75]

#### **Organic matter**

Usually, organic matter is concentrated in the surface horizon of the soil, over a depth of between 5 and 35cm. Sometimes organic matter may contain visible plant components. Elsewhere the decomposition of the plant structure is so advanced that a black material is encountered. This is called humus.

Recently decomposed organic matter has different properties than humus. It consists of macrograins or fibres that are relatively inert from the physical or chemical point of view.

Humus is colloidal and acidic, with a very high cation exchange capacity and the ability to absorb water, which increases its volume.

Organic matter has an open and spongy structure and has only low mechanical strength. The moisture content may be very high (from 100 to 500%) eliminating all mechanical stability. The acidity of the organic components tends to trigger acid reactions in the soil, which may lead to the corrosive attack on materials with which it comes into contact. The concentration and type of organic matter has marked effects on the characteristics of a natural soil once it exceeds 2 to 4%. [39]

#### Mineral matter

The mineral or inorganic components of a soil usually represent by far the greatest part of the soil.

Two groups of minerals may be distinguished: [63]

*Unweathered minerals* or incompletely weathered minerals. These minerals are identical in composition to the parent rock from which they derive. They include pebbles and gravels and fine and coarse sands . These are the sandy elements.

*Weathered minerals* are the result of the chemical weathering of the minerals of the parent rock and are characterized by their extreme fineness (less than  $2\mu\text{m}$ ). Because of their smallness these grains of weathered mineral have the appearance of a sticky paste if they are wet. They are called colloids, which derives from the French *colle* (glue) and means 'glue-like'. They were given this name because they form the binder in the soil. The main binders are the clays and for this reason engineering geology refers to them as the clayey fraction rather than the colloidal fraction.

## **Sandy elements**

These may be silicas, silicates, or limestones.

*Silicas* These are grains of quartz resulting from the disintegration of sandstone and crystalline rocks. They are found in both the larger and finer fractions.

*Silicates* The chemical weathering of these proceeds continuously but is very slow. They are made of grains of mica, feldspar and other free minerals resulting from the disintegration of crystalline rocks such as granite and volcanic rocks. The weathering of these elements is increasingly effective as grain size is reduced.

*Limestones* This is the part of those sandy soils which are made from calcium carbonate. A distinction must be made between soils created on a limestone parent rock and the soils formed on a non-limestone parent rock. Limestone is not always present in the soil but all soils contain calcium fixed on clay in the form of calcium ions or in soil solution in the form of soluble calcium salts.

## **Classification of mineral components**

In order to facilitate their identification, the mineral components have been divided into grain fractions.

### **Pebbles**

Pebbles range in size from 20-200mm. They form a rough material which is the result of the disintegration of the parent rock from which they draw their basic characteristics. They may also have been carried from elsewhere. Young

pebbles still have sharp corners. Severely weathered pebbles, as well as those that have been carried by watercourses or glaciers are rounded.

#### Gravel

Gravel ranges in size from 2 to 20mm. It is made up of small grains of rough material, which are the result of the disintegration of the parent rock and pebbles. They may also have been carried by watercourses and thus be rounded, though angular gravel also exists. Gravel constitutes the skeleton of the soil and imposes a limit to its capillarity and shrinkage.

#### Sands

Sand ranges in size from 0.06 to 2mm. It is often made up of particles of silica or quartz. Its high internal friction marks the sandy component of a soil. The low adsorption of these surfaces limits swell and shrinkage. The open structure and permeability are typical of sands.

#### Silt

The grain size of the silt ranges from 0.002mm (2 $\mu$ m) to 0.06mm. From the physical and chemical point of view the silt component is virtually identical to the sand component, the only difference being one of size. Silt gives soil stability by increasing its internal friction. Because of their high degree of permeability silty soils are very sensitive to frost. They are subject to small-scale swell and shrinkage.

#### Clays

Clay grains are smaller than 2 $\mu$ m. They differ from sand and silt particles not only in size but also in their chemical composition and physical properties. In chemical terms they are hydrated alumino-silicates formed by the leaching process acting on the primary minerals in rock. The key to clay mineralogy is the small diameter of the mineral grains and their crystallographic habit, which is sheet-like. These two factors give clays a very high surface area relative to the mass of material in

the clay mineral crystal grains. Their specific surface is infinitely greater than that of rougher round or angular particles. This large surface area gives the properties of water absorption common to all hydrophilic materials. [39] The surface residual charges on the mineral structure attract and adsorb water in layers. Clays are very susceptible to swell and shrinkage due to their affinity for moisture which when adsorbed causes swelling and when the water evaporates shrinkage occurs.

Large clay molecules (or micelles) are fine crystals of an irregular or hexagonal shape. The latter is the most common, but there are others. These include pseudo-hexagonal wafers, cylindrical or hollow tubular fibers, thick tablets or discs. Clay micelles are made up of thin sheets or leaves, which is why argillaceous minerals (minerals composed of very fine-grained material) are referred to as phyllite ('phyllon' = leaf in Greek). Each micelle is made up of several tens or even hundreds of sheets, the structure of which determines that of the minerals as well the immediate properties of the crystal, including adsorbent properties analogous to those of colloids. These sheets have a chemical make-up which varies according to the type of clay and the degree of hydration as well as their thickness and spacing, namely from 7 to 20 Angström ( $1\text{\AA} = 10$  millionth of a mm). Some sheets are made of silica (atoms of silicon surrounded by oxygen atoms), and others are made of alumina (aluminium atoms surrounded by oxygen atoms and OH groups)

## 1.8 STRUCTURE OF BASIC CLAY MINERAL GROUPS

Clay minerals as a group are largely phyllosilicates. Silicates are made of  $\text{SiO}_4^{4-}$  ionic groups in combination with metallic cations (positively charged ions). Oxygen is much larger in size than most of the cations, so volumetrically oxygen makes up the bulk of the silicates. Silicate minerals, as well as the earth's entire crust, can be visualised as being made up of close-packed oxygens with smaller cations filling the interstices. [20]

## The Silica Tetrahedron

Silicate minerals have as their basic building block the  $\text{SiO}_4^{4-}$  silica tetrahedron consisting of a silicon surrounded by four oxygens in a tetrahedral configuration.

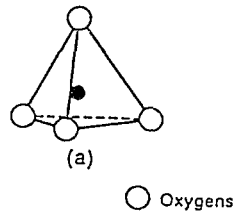


Figure 1.1: A single silica tetrahedron.

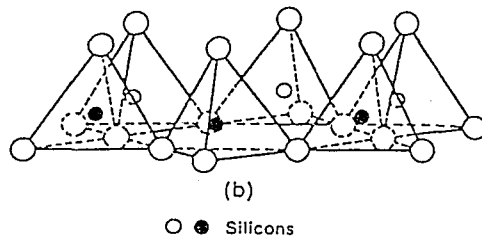


Figure 1.2: Silica tetrahedrons arranged in a hexagonal network.

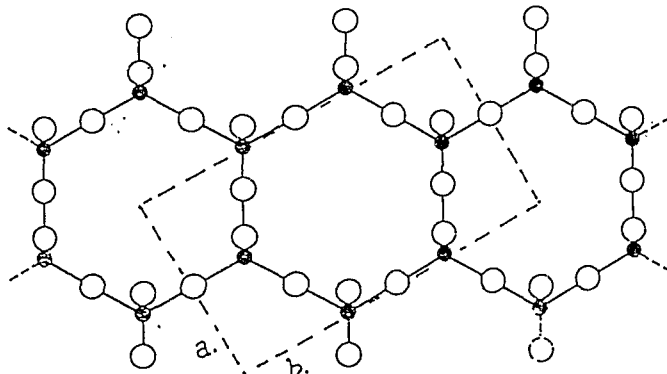


Figure 1.3: Three hexagons joined into a double chain

## The tetrahedral sheet

The great number and variety of silicate minerals result in part from the ability of silica tetrahedra to link together in various ways by sharing apical oxygens. (figure 1.2)

Tetrahedron faces and edges are rarely shared because this brings the highly charged silicons too close to each other. Note that in figure 1.1-1.5, the oxygens are shown as much too small, actually they are all touching. Phyllosilicates are characterised by silica tetrahedra linked to form sheets, and in clay minerals these sheets consist of six membered rings of tetrahedra each of which shares 3 of 4 oxygens. The unshared oxygens in one sheet all point in the same direction. Note that the sheet consists of two planes of oxygens, but there are more oxygens in the lower than the upper plane. The formula for a tetrahedral sheet is  $\text{Si}_4\text{O}_{10}^{-4}$ , and since it has negative charge, it can only exist in combination with cations and additional oxygens in actual mineral structures. Tetrahedral coordination can accommodate only the smaller cations. These are silicon, less commonly aluminum, and rarely  $\text{Fe}^{3+}$ .

## The octahedral sheet

In phyllosilicate clay minerals, the tetrahedral sheet is always combined in one of several ways with an octahedral sheet consisting of cations surrounded by 6 nearest neighbors (figure 1.4). This configuration accommodates the larger cations such as  $\text{Al}^{+3}$ ,  $\text{Mg}^{+2}$ ,  $\text{Fe}^{+2}$ ,  $\text{Fe}^{+3}$ , but not  $\text{Ca}^{+2}$ ,  $\text{Na}^{+1}$ , or  $\text{K}^{+1}$  which are too large.[24] The octahedral sheet consists of two planes of hexagonally close-packed oxygens or hydroxyls (which are nearly the same size as oxygens). If we visualise the first of these planes we will see that between every 3 balls there is a low point or hollow, this is alternately occupied by the cations, or by the oxygens (or  $\text{OH}^-$ ) of the upper plane. The octahedral designation stems from the fact that connecting the 6 nearest neighbor oxygens or hydroxyls forms an eight sided solid: an octahedron. In the octahedral sheet, the octahedron is lying on one of its faces (figure 1.4).

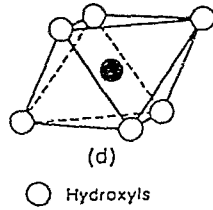


Figure 1.4: A single octahedral unit.

Unlike the tetrahedral sheet, the octahedral sheet can exist by itself. The minerals brucite,  $Mg_3(OH)_6$  and gibbsite,  $Al_2(OH)_6$  are composed entirely of octahedral sheets. The structure shown in figure 1.5 is brucite; note that all the cation positions are filled. In the gibbsite structure only 2/3 of these positions contain cations. This gives rise to an important pair of structural terms: brucite is trioctahedral (3/3 of positions filled), and gibbsite is dioctahedral (2/3 of positions filled).

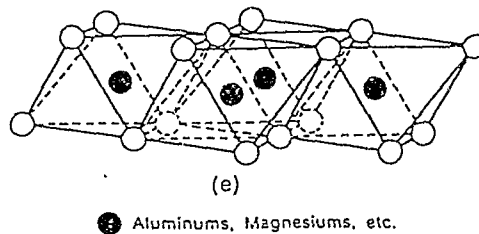


Figure 1.5: Several octahedral units joined into an octahedral sheet structure.  
[25]

### 1.8.1 Basic layer types

#### The 1:1 layer

The simplest phyllosilicate clay mineral structure combines one octahedral and one tetrahedral sheet to form a 1:1 layer. The layer can be either dioctahedral or trioctahedral. [72]

Assuming that the tetrahedral and octahedral sheets can be assembled to form a 1:1 layer the unshared apical oxygens in the tetrahedral sheet become part of the octahedral sheet, substituting for hydroxyls (figure 1.6 below). Thus, a 1:1 layer consists of three planes of large anions ( $\text{OH}^-$ , or  $\text{O}^-$ ) within which are a plane of octahedral cations and a plane of silicons. If the octahedral sheet is on the bottom, then the first plane is all  $\text{OH}^-$ , then come the octahedral cations, then a plane of both  $\text{OH}^-$  and ( $\text{OH}^-$ , or  $\text{O}^-$ ) (the tips of the tetrahedra), then silicons, then a plane entirely of  $\text{O}^-$ . The middle plane of large anions ( $\text{OH}^-$ ,  $\text{O}^-$ ) is part of both the tetrahedral and octahedral sheets. In figure 1.6 the incomplete layer of the top is part of the next 1:1 layer identical to the first.

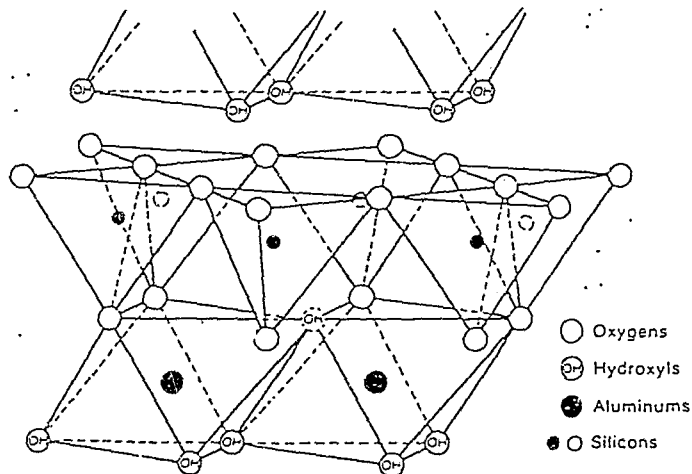


Figure 1.6: Sketch of kaolinite, 1:1 dioctahedral phyllosilicate.

[25]

## The 2:1 layer

The other major layer type consists of two tetrahedral and one octahedral sheet, which together form the 2:1 layer. This can be visualised (figure 1.7) as similar to the 1:1, but with another tetrahedral sheet inverted with respect to the first. In figure 1.7, the lowermost tetrahedral sheet is an incomplete part of the next 2:1 layer. The whole layer consists of 4 planes of large anions ( $\text{OH}^-$ ,  $\text{O}^{2-}$ ) and 3 planes of cations. The bottom is all oxygens, then a plane of silicons, then a plane of mixed  $\text{OH}^-$  and  $\text{O}^{2-}$ , then the octahedral cations, then another mixed plane, then another plane of silicons and finally a plane of oxygens. The octahedral sheet is made, in part, of the oxygens from the inward pointing tetrahedra above and below. The 2:1 layer can be either dioctahedral or trioctahedral.

Individual 1:1 or 2:1 layers (neglecting broken bonds on the edges) can be either neutral or bear a negative charge, balanced by cations between the layers in the interlayer space. The layer charge is controlled by the substitution of cations in the tetrahedral or the octahedral sheets. Layer charge arising from tetrahedral substitution is called tetrahedral charge; charge arising from octahedral substitution is called octahedral charge. Different ways of stacking identical layers produces various polytypes (a variety of polymorph or isomer). Stacking individual layers of different type, composition, or charge produces mixed layered clays. The mixed layered can be ordered or random/irregular.

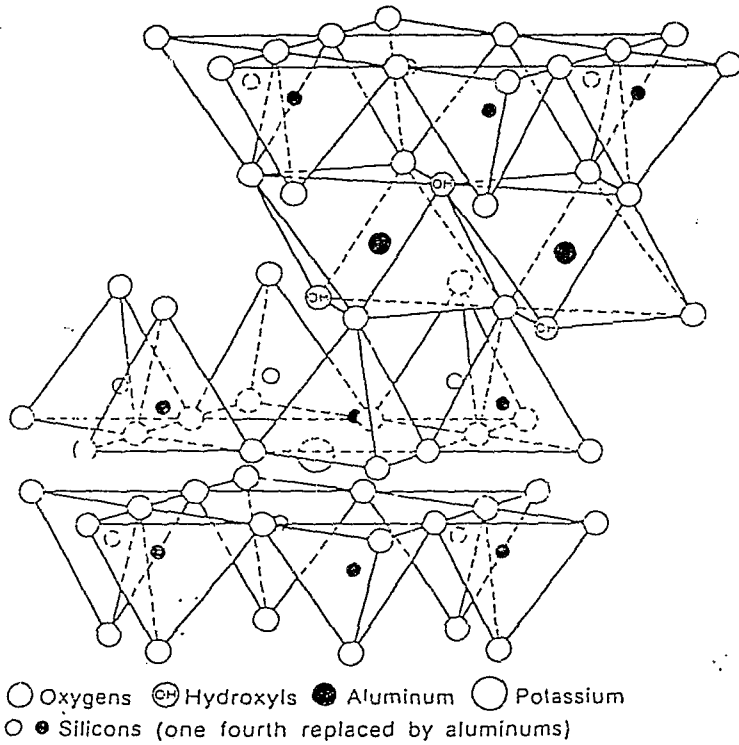


Figure 1.7: Sketch of Muscovite, a 2:1 dioctahedral phyllosilicate.

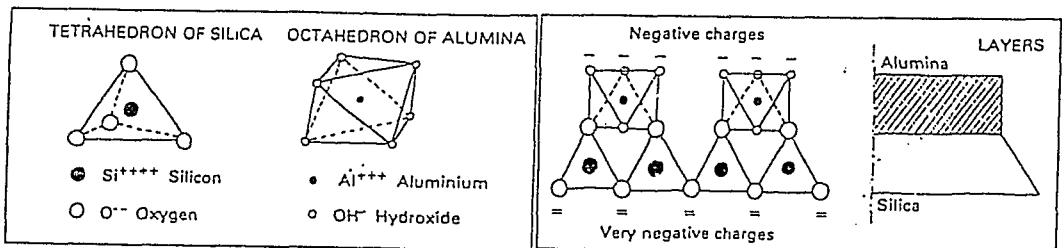
[25]

## Cation exchange capacity (CEC)

The cation exchange capacity is a measure of the capacity of a clay to exchange cations. Units of measure are milliequivalents per 100 grams of clay (meq/100g; an "equivalent" of an ion is its molecular weight divided by its valence; thus, an equivalent of  $\text{NH}_4^+$  is  $18/1 = 18$ , an equivalent of  $\text{Na}^+$  is  $23/1 = 23$ , and an equivalent of  $\text{Ca}^{+2}$  is  $40/2 = 20$ ). The CEC is a measure of the concentration of unfixed cations in the diffuse layer (interlayers and surface layers), which, as described above, depends on the magnitude of the total (structural plus surface) layer charge, which in turn varies with pH. The CEC is usually measured at a pH of 7.

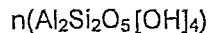
### 1.8.2 Clay mineral groups

Each clay mineral group is characterized by a particular type of layer structure and interlayer material. There are three major clay groups, these are the kaolinite group, the illite/mica group and the smectite/montmorillonite groups. Mineralogically, clay may contain a variety of components such as kaolinite, vermiculite, illite, chlorite and montmorillonite. [49]

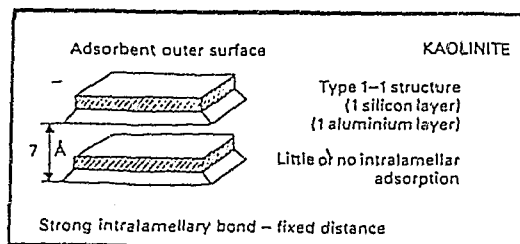


## Kaolinites

Kaolinites fall into the 1:1 layer group. The sheets found in kaolinite clays are made up of a layer of oxygen tetrahedrons with a silicon centre and a layer of oxygen (or hydroxide) octahedrons with an aluminium centre. A kaolinite sheet has the following general formula:

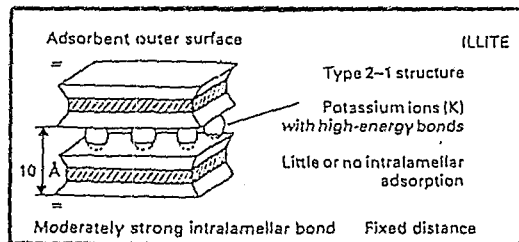


In kaolinites, bonding between successive layers is both by van der Waals forces and hydrogen bonds. This bonding is of sufficient strength that layers will not separate in the presence of water (i.e. swell). Often other elements replace the silicon and aluminium ions in the crystal. This does not effect the crystal as long as they have the same valence electrons (e.g.  $\text{Fe}^{3+}$  replacing  $\text{Al}^{3+}$ ). If they are of a lower valency (e.g.  $\text{Al}^{3+}$  replacing  $\text{Si}^{4+}$  or  $\text{Mg}^{2+}$  replacing  $\text{Al}^{3+}$ ), the crystal will be left with a net negative charge which will result in a positively charged ion from the soil water being attracted to the crystal surface. Kaolinites undergo little isomorphous substitution if any. Minerals of this group are therefore nearly inert, but cation exchange capacities of 1 to 10 meq/100g have been measured. Kaolinite is negatively charged only at the edge of the sheets and its ion fixing capacity is low when compared to the other clay groups. The distance between the sheets is constant, being 7Å. The thickness of the crystals is between 0.005 and 2µm. The outer area (OA) is approximately 15m<sup>2</sup> per gram. The internal area (IA) = 0. Kaolinite is generally stable in contact with water. [39]



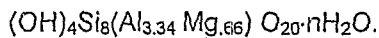
## Illites

This group has a three layer structure and forms part of the 2:1 layer group: one mainly aluminous octahedral layer, between two mainly siliceous tetrahedral layers. Magnesium (Mg) or Fe (Iron) ions may partly replace the Aluminium (Al) ions in the aluminous layer and Al ions may substitute for Silicon (Si) in the silica layer. As the sheet is non-saturated, the negative charges are balanced by the Potassium (K) ions, which bond the sheets. The distance between sheets is 10Å, and the thickness of the crystals is between 0.005 and 0.05µm. The CEC for illite is 10 – 40 meq/ 100g. The outer surface area is approximately 5m<sup>2</sup> per gram and the internal surface area is 25m<sup>2</sup> per gram. Illite is not particularly stable when in contact with water and suffers swell. [39]



## Montmorillonites

The structure of this group is comparable to that of illite, but substitution takes place in the octahedral alumina layer. The structural formula for montmorillonite is: [20]

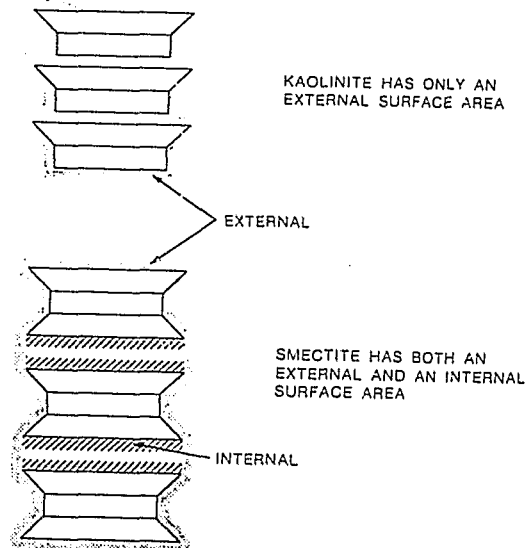
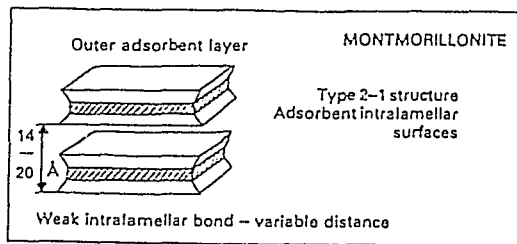


The term Mg<sub>.66</sub> indicates that substitution of Mg<sup>2+</sup> for Al<sup>3+</sup> in the octahedral sheet



is the source of charge deficiency, and that the sodium (Na<sup>+</sup>) is adsorbed to satisfy the charge deficiency. Measured ion exchange capacities range from 80 to 150 meq/100g). [11] The interlayer bonding in montmorillonite is by Van der

Waals forces and by exchangeable cations that may be present to balance charge deficiencies in the structure. These bonds are weak and can easily be separated by cleavage or adsorption of water, and balancing cations take up positions between the unit cell layers as well as on the surface of the particles. The distance between sheets ranges from 14 to 20Å. The thickness of the crystals lies between 0.001 and 0.2µm. The outside surface area is approximately 50m<sup>2</sup>/g and the inside surface area is 800m<sup>2</sup>/g. The CEC of montmorillonite/ is 80 – 150 meq/100g. Montmorillonite is not stable when in contact with water and suffers severely from swell. [39]



**Figure 1.8: Difference between external and internal surface areas of clay minerals**

### 1.8.3 Mixed-layer clay minerals

Mixed-layered (ML) clay minerals (also called interstratified or interlayered) are phyllosilicates in which two or more layer-types are intermixed in a vertical stacking sequence within a single crystal. [75] They are not simple physical mixtures of the component layers. Mixed-layering results from the fact that bonding is strong within individual layers, but weak between layers, and different types of layers have nearly identical configurations of tetrahedral oxygens bounding their outer surfaces, so can fit together relatively well. Mixed-layer clays are widespread in nature.

#### Layer Types

Most of the 1:1 and 2:1 layer-type minerals have been identified in ML clays. The most commonly encountered layer minerals are illite, smectite, vermiculite and chlorite. [11] Most natural ML clays in sedimentary rocks are composed of 2:1 layers; those involving 1:1 layers are rare. The 2:1 layers are fundamentally similar, and differ only with respect to layer charge and interlayer materials. This is illustrated in Figure 1.8 where the 2:1 layers are shown with a diagonally ruled pattern. If the interlayer contains potassium it is a mica or illite layer with a 10 Å spacing and if it contains 2 water layers it is a smectite or vermiculite, and if it contains a hydroxide sheet it is a chlorite layer; the latter two types have a 14 Å spacing. The 10 Å, potassium-containing layers have been referred to as both mica and illite, so that generally the following terms should be considered synonymous: mica/smectite, illite/smectite, mica/montmorillonite and illite/montmorillonite.[20]

## Stacking Arrangement [19]

There are four basic ways of stacking the component layers (figure 1.10):

1. Regular interstratification of two components, each of which is present in nearly equal amounts (figure 1.10a).
2. Segregation into crystallites (figure 1.10b)
3. Random Mixed-Layering (figure 1.10c)
4. Regular Mixed-Layering (figure 1.10d)

The first two of these are important, but not common on nature, and the other two describe the arrangements typical of ML clays.

- 1) Regular interstratification of two components, each of which is present in nearly equal amounts (Fig. 1.10a).

Minerals in this category have layers which are repeated regularly (figure 1.9a). Minerals in this category, because of their regularity, are given legitimate mineral names, some of which are:

Rectorite – dioctahedral mica/smectite

Hydrobiotite – trioctahedral biotite/vermiculite

Corrensite – trioctahedral chlorite/smectite or chlorite/vermiculite

Tosudite – dioctahedral chlorite/smectite

Aliettite – talc/saponite

The layers do not necessarily follow each other in equal number of layers as shown in figure 1.9a. Highly regular arrangements can be conceived in which the proportions of layers are not equal, such as the ABBABBABB..... with 33% "A" layers. Some of these have been described in natural samples, but regularity is insufficient to warrant a mineral name. Minerals such as rectorite are sometimes described as "1:1 regular interstratification" where the prefix "1:1" refers to the

50-50 proportions of layers and not to a 1:1 layer type. The ABBABBA.... Sequence given above would then be a "1:2 regular interstratification".

The stacking arrangement of ML clay minerals can be stated in probability terms. For the case of perfect 1:1 order (50-50) of A and B component layers,  $P_A$  is the probability (or proportion) of an A layer and  $P_B$  of a B layer such that

$$P_A + P_B = 1$$

The probability of an A following a B is given by  $P_{A.B}$ , and since in this example A always follows B:

$$P_{A.B} = P_{B.A} = 1;$$

And, since AA and BB pairs are never found,

$$P_{A.A} = P_{B.B} = 0$$

## 2. Segregation into crystallites (figure 1.10b)

Where stacks of identical layer-types are many layers thick they simply become physical mixtures of the two components. Stacks just a few layers thick mixed in various ways show XRD patterns unlike either of the components, but stacks of identical layers perhaps 10 or more layers thick begin to show XRD peaks characteristic of two separate phases. The probability expressions for this are:

$$P_{A.A} = P_{B.B} = 1, \text{ and}$$

$$P_{B.A} = P_{A.B} = 0.$$

These expressions are based on the stacks being large, and do not consider the A-B junction in the middle of figure 1.9b.

### 3. Random Mixed-Layering (figure 1.10c)

This type of interstratification is wide spread in nature. The components are simply randomly mixed like the red and black cards in a shuffled deck. The XRD peaks are characteristic of neither of the component phases. The probability expressions for this are:

$$P_{A.A} = P_{B.A} = P_A, \text{ and}$$

$$P_{B.B} = P_{A.B} = P_B.$$

That is, the probability of occurrence of an A following an A or a B, or of a B following an A or a B simply depends on their abundance ( % of component layers ) in the crystal, and does not depend on what preceded it. Random mixed-layering is common in illite/smectites that have less than 50% illite layers.

### 4. Regular Mixed-Layering (figure 1.10d)

Interstratification can be perfectly ordered on an ABAB scheme even where the proportion of component layers is not 1:1. This type of ordered mixed-layering is very common, particularly with illite/smectites. If  $P_A > 0.5$  (greater than 50% A layers), this ordering is given by the expression:

$$P_{B.B} = 0$$

This means that the probability of a B being followed by a B is zero; that is there are no BB pairs in the crystal. Such a clay would consist of perfect ABABAB.... ordering with the excess A randomly distributed such that there would be some AA pairs or AAA triads, etc., but no BB pairs. Such a scheme is shown in figure 1.9d.

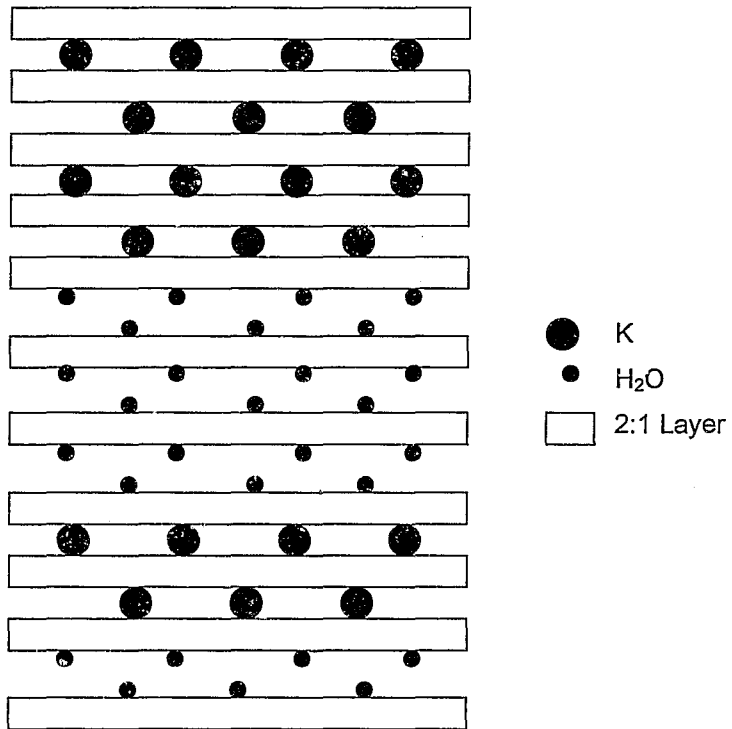
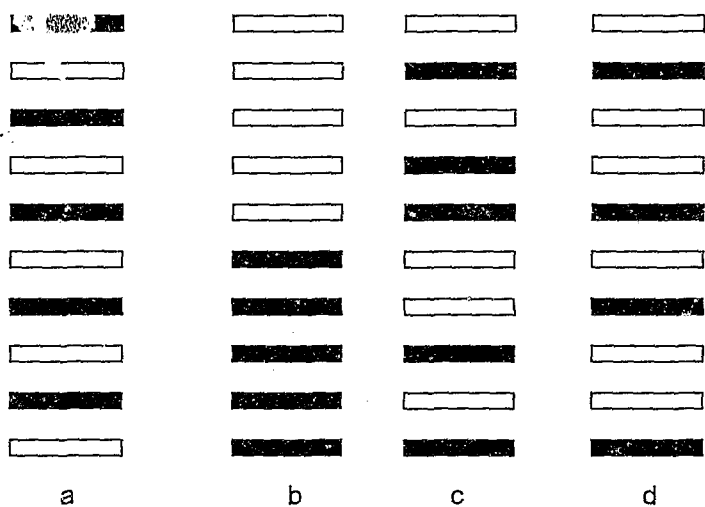


Figure 1.9 : Mixed-Layer Illite/Smectite



- a. regular interstratification
- b. segregation into crystallites
- c. random mixed layering
- d. regular mixed layering

**Figure 1.10: Graphical representations of stacking arrangements in mixed-layered clays.**

## 1.9 PROPERTIES OF SOILS

The properties of soils may vary considerably from one soil to another. They depend on the complex nature of the mix of the various granular fractions. The characteristics of the soil depend on the proportions of pebbles, gravel, sand, silt, clays, colloids, organic matter, water and gas. It is often the predominant fraction in the soil, which governs the fundamental properties of a material.[40]

### Physical properties

These properties fall into four main categories: [7]

1. Texture or grain (particle) size distribution of the soil in percentages of pebbles, gravel, sands, silts, clays and colloids. The grain size distribution of a soil is measured by particle size analysis for the coarse fraction (gravels, sands and silts) and by sedimentation analysis for the fine fraction (clays). Gravels and sands give the material its strength, whilst the clays bind it together; silts fulfil a less clear intermediate function. The texture of a soil is usually presented in the form of a particle size distribution curve. Figure 1.11 – 1.14 are examples of the particle size distribution curves of typical soils with varying percentages of constituent materials.

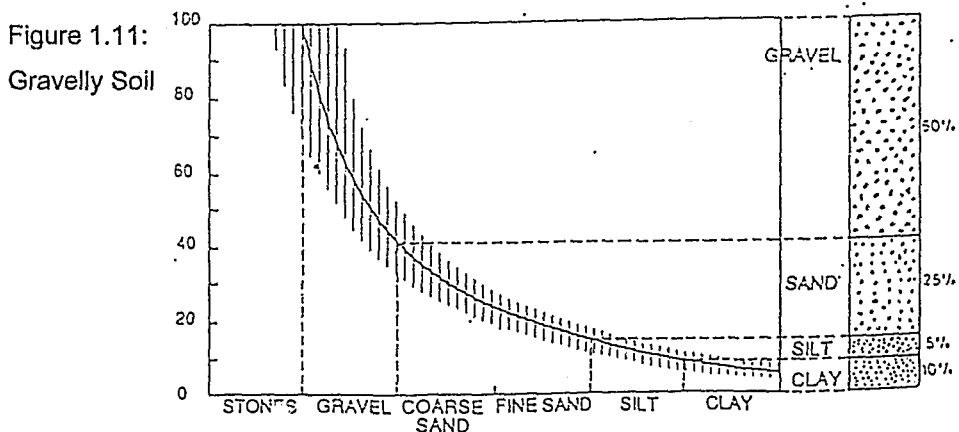


Figure 1.12:  
Sandy Soil

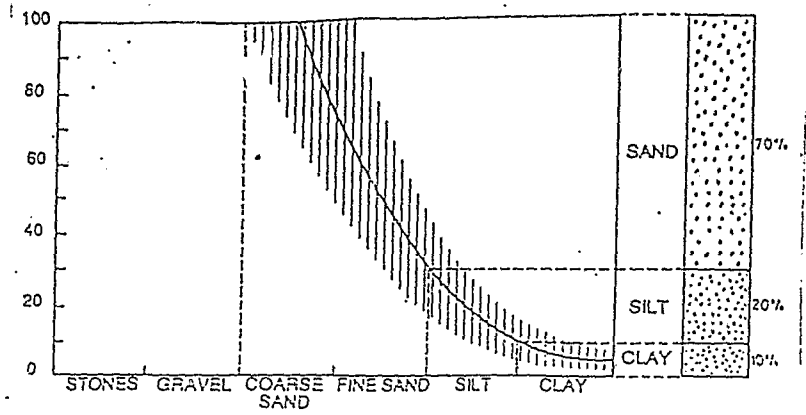


Figure 1.13:  
Silty Soil

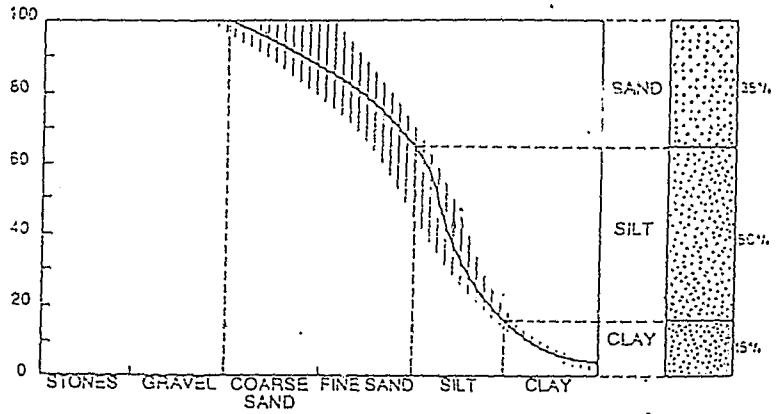
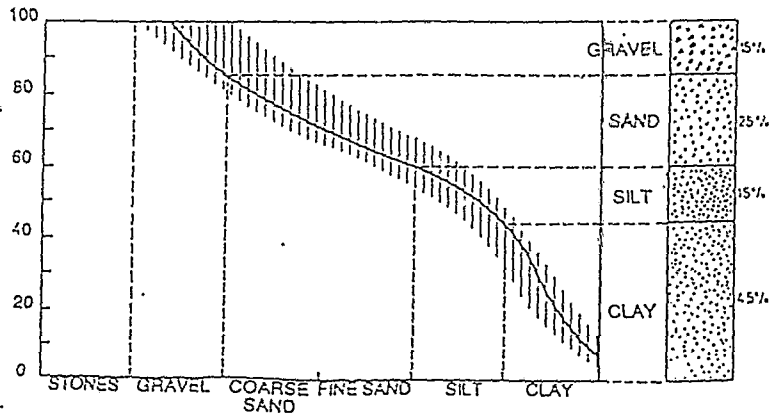


Figure 1.14:  
Clayey Soil.



## 2. Atterberg limits

Plasticity defines the extent to which a soil can be distorted without any significant inelastic reaction, typically cracking or crumbling, occurring. The plasticity of the soil, as well as the limits between different states of consistency, are defined by measuring the "Atterberg Limits". These are carried out on the finer fraction of the soil (particle size diameter less than 0.4 mm). The amount of water expressed in percentage terms, corresponding to the point at which the material passes from a liquid to a plastic state is known as the Liquid Limit (LL). The point, at which it passes from a plastic to a solid state, is known as the Plastic Limit (PL). At the Liquid limit, the soil begins to display some resistance to shearing. At the Plastic limit, the soil ceases to be plastic and becomes crumbly. The Plasticity Index (PI), determines the extent of the plastic behaviour of the soil, where:

$$\text{Plasticity Index (PI)} = \text{Liquid Limit} - \text{Plastic Limit}$$

The Atterberg Limits also include:

- Shrinkage limit
- Adsorption Limit
- Limit of adhesion

The most important of the limits are the Plastic limit and the Liquid limit, while the other three are only rarely used.

## 3. Compressibility

The compressibility of a soil defines its maximum capacity to be compressed for a given amount of compaction energy and at a given moisture content. When a force is applied to a quantity of soil, the material is compressed and the

proportion of voids decreases. The more the density of a soil can be increased, the lower its porosity will be and the more difficult it will be for water to penetrate. This property results from the tighter overlapping of the particles, which lowers the risk of the structure being modified in the presence of water. The moisture content must be high enough to lubricate the particles and enable them to move around in such a way as to occupy as little space as possible. At the same time, the moisture content must not be too high, or the voids would be full of water, and therefore impossible to compress.

4. Cohesion or the ability of the soil grains to remain in association.

The cohesion of a soil is an expression of the capacity of its grains to remain together when a tensile stress is imposed on the material. The cohesion of the soil depends on the adhesive or cementation properties of its coarse mortar (grain size of diameter larger than 2mm) which binds the inert grains together. This property thus contributes to the quantity and adhesive quality of the clays.

## 1.10 SOIL CLASSIFICATION

### The Unified Soil Classification System

This system was developed in the United States of America in the 1940's.

The symbols used to describe the soils consist of a primary and a secondary descriptive letter. These are :

Primary Letter	Secondary Letter
G : Gravel	W : Well graded
S : Sand	P : Poorly graded
M : Silt	M : With non-plastic fines
C : Clay	C : With plastic fines
O : Organic Soil	L : Of low plasticity (LL<50)
Pt : Peat	H : Of high plasticity ( LL>50)

Table 1.2: USCS symbols used to classify soils [8]

If the soil sample is coarse (contains more than 50 percent sand and gravel i.e.: more than 50 percent is retained on the #200 sieve [0,075mm]) complete classification requires a grain size distribution curve [62]. From this curve one can determine the percentage composition of the various constituents, as well as the coefficients of uniformity and curvature. If a coarse soil contains more than 12 percent fines (silt and clay), the liquid limit and the plasticity index must also be determined in order to undertake complete classification. If the soil is fine grained (ie : more than 50 percent passes the # 200 sieve ), both the liquid limit and the plasticity index must be determined.

The procedure for classifying a soil is to begin at the beginning of the soil classification chart and make a decision at each point according to the criteria

presented. At the conclusion of the process an appropriate group symbol is obtained.

For fine soils, a distinction is made between silts, clays, high plasticity and low plasticity and low compressibility based on the plasticity chart.

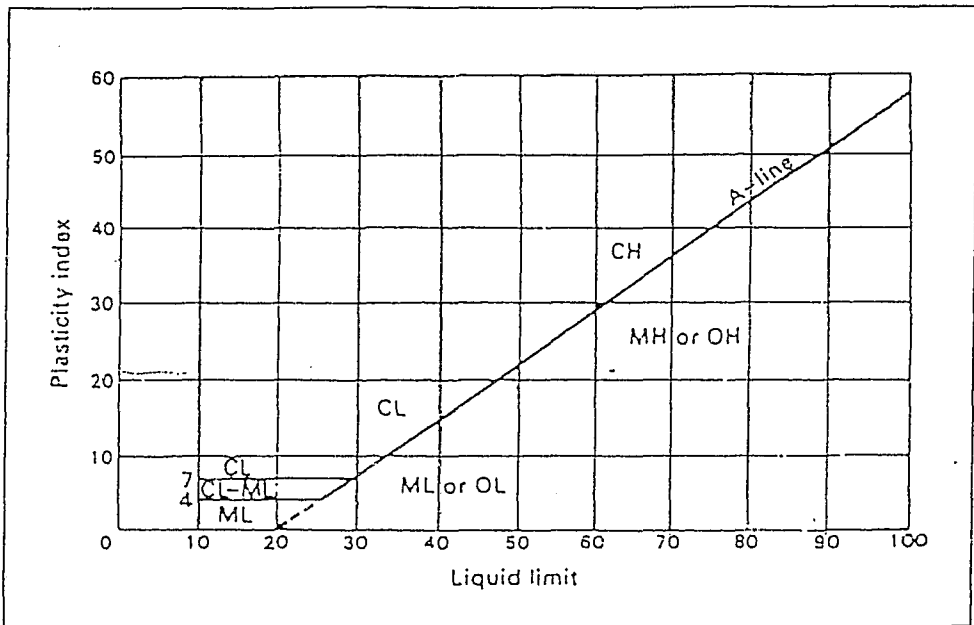


Figure 1.15 : Plasticity chart for the Unified Soil Classification System [8]

Primary Letter	Secondary Letter
M : Silt	L : Of low plasticity (LL<50)
C : Clay	H : Of high plasticity (LL>50)
O : Organic Soil	

The following example will illustrate how the Unified Soil Classification System works: Assume there is a soil A with a gradation curve as illustrated in Figure 1.16: [62]

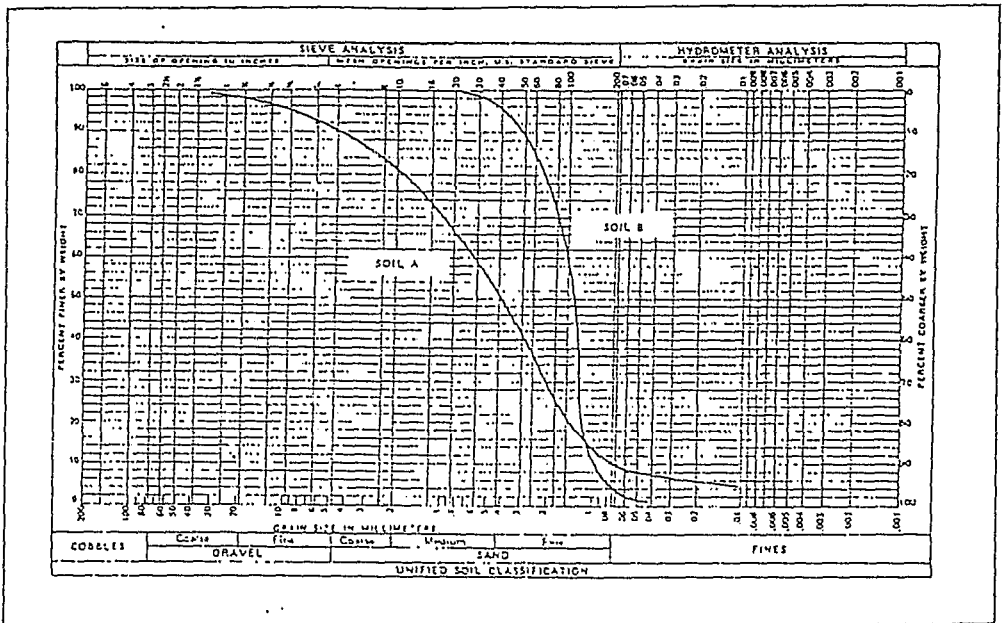


Figure 1.16 : Gradation curves for Soil Classification example [62]

Look at the sieve on the top of the graph and find the #200 sieve. Read down the 200 sieve line until it intercepts with the gradation curve for soil A. Read the corresponding "percentage finer by weight" reading from the left hand side of the graph. This reading for soil A is approximately 10 percent i.e.: approximately 10 percent of soil A is smaller than the #200 sieve. Looking at the gravel part of the curve, (that which is greater than the # 4 sieve [4,75mm]), only about 10 percent of soil A falls into the gravel category (this is approximately 11 percent of the total coarse fraction). From the classification table, if more than 50 percent of the

coarse fraction passes the #4 sieve the soil is sand. As the fines fraction was about 10 percent of the sample by weight, the soil is regarded as a borderline case requiring a dual classification (borderline cases are those soils with fines between 5 and 12 percent). Because of the dual classification, the soil needs to be further classified using the gradation curve. By determining the percentage finer by weight at 60 percent, 10 percent and 30 percent and finding the corresponding grain size from the gradation curve for soil A, the uniformity and curvature coefficients can be calculated.

This is done as follows:

$$D_{60} = 0,6\text{mm}$$

$$D_{10} = 0,074\text{mm}$$

$$D_{30} = 0,2\text{mm}$$

$$\begin{aligned} \text{Coefficient of Uniformity: } C_u &= \frac{D_{60}}{D_{10}} = \frac{0,6\text{mm}}{0,074\text{mm}} \\ &= 8,1 \end{aligned}$$

$$\begin{aligned} \text{Coefficient of Concavity: } C_v &= \frac{(D_{30})^2}{D_{60} \times D_{10}} = \frac{(0,2)^2}{(0,6 \times 0,074)} \\ &= 0,9 \end{aligned}$$

According to the Unified Soil Classification Chart (Table 1.2) the soil does not meet the requirements for a well graded soil.

Based on Wagner, A. A. (1957) *Proceedings of the Fourth International Conference SMFE, London, Vol. 1*. Reproduced by permission of Butterworth & Co.

Description	Group Symbols	Laboratory criteria					
		Fines (%)	Grading	Plasticity	Notes		
Coarse grained (more than 50% larger than 63 $\mu$ m BS or No. 200 US sieve size)	Gravels (more than 50% of coarse fraction of gravel size)	Well graded gravels, sandy gravels, with little or no fines	GW	0-5	$C_u > 4$ $1 < C_g < 3$	Dual symbols if 5-12% fines. Dual symbols if above A-line and $4 < PI < 7$	
		Poorly graded gravels, sandy gravels, with little or no fines	GP	0-5	Not satisfying GW requirements		
		Silty gravels, silty sandy gravels	GM	> 12			Below A-line or $PI < 4$
		Clayey gravels, clayey sandy gravels	GC	> 12			Above A-line and $PI > 7$
	Sands (more than 50% of coarse fraction of sand size)	Well graded sands, gravelly sands, with little or no fines	SW	0-5	$C_u > 6$ $1 < C_g < 3$		
		Poorly graded sands, gravelly sands, with little or no fines	SP	0-5	Not satisfying SW requirements		
		Silty sands	SM	> 12			Below A-line or $PI < 4$
		Clayey sands	SC	> 12			Above A-line and $PI > 7$
Fine grained (more than 50% smaller than 63 $\mu$ m BS or No. 200 US sieve size)	Silt and clays (liquid limit less than 50)	Inorganic silts, silty or clayey fine sands, with slight plasticity	ML	Use plasticity chart			
		Inorganic clays, silty clays, sandy clays of low plasticity	CL	Use plasticity chart			
		Organic silts and organic silty clays of low plasticity	OL	Use plasticity chart			
	Silt and clays (liquid limit greater than 50)	Inorganic silts of high plasticity	MH	Use plasticity chart			
		Inorganic clays of high plasticity	CH	Use plasticity chart			
		Organic clays of high plasticity	OH	Use plasticity chart			
Highly organic soils	Peat and other highly organic soils	Pt					

Table 1.2 : Unified Soil Classification System Chart [8]

This is because soil A has a  $C_u$  greater than 6 and a  $C_c$  of less than 1. The soil is therefore poorly graded sand. Limits according to figure 2.6 for a well graded sand is where  $C_u$  is greater than 6 and  $1 < C_c < 3$ . Because fines content (that which is smaller than 0,075mm) is between 5 and 12 percent, it is necessary to determine whether the fines are clayey or silty. The laboratory tests indicated that the liquid limit (LL) was 63, the plastic limit (PL) was 42 and that the plasticity index (PI) therefore was 21. From the plasticity chart it is found that the point at which the plasticity index of 21 and the liquid limit of 63 for soil A cross, falls below the A- line and would therefore be classified as a silty sand.

The total description for soil A would therefore be SP- SM.

If the same exercise is followed for soil B, it is found that the appropriate description for soil B is SP.

The same process used to illustrate the example has been used in section 3.5 to classify the soil used in this research.

# CHAPTER TWO

## 2. SOIL STABILISATION.

### 2.1 Definition.

The term soil stabilisation is applied to any process which improves the properties of a soil and which enables it to perform and sustain its intended engineering use. [74] Soil stabilisation is not a recent discovery. The use of stabilisers to improve the performance of soil as a construction material dates back to ancient Egypt where the discovery of lime and gypsum as binding agents aided the construction of structures like the pyramids.

The Romans used the knowledge they had gained from the Greeks and the Etruscans on cementing materials to construct many buildings in the Roman Empire. In the America's the Spaniards first introduced adobe mixed with pigs' blood and other ingredients to produce more durable (when compared with pure adobe) building materials. [66]

In the early nineteenth century, the development of "Portland cement" by Joseph Aspdin, led to its use in soil stabilisation in a similar way to its use for binding aggregates in concrete. [53]

#### 2.1.2 Effect of soil stabilisation on the properties of the soil.

The action of soil stabilisation can be directed at only two characteristics of the soil itself, i.e. its texture and structure [2,33,40,49]. There are three courses of action, which can be aimed at texture and structure:

- Reducing the volume of interstitial voids: acts on porosity
- Filling the voids which cannot be eliminated: acts on permeability
- Improvement of the bonding between grains: acts on mechanical strength

The main objectives aimed at are: [16]

- Achieving better mechanical characteristics: improving dry and wet compressive strength;
- Achieving better cohesion;
- Reducing porosity and changes in volume: shrink and swell due to water.
- Improvement of resistance to wind and rain erosion: reduction of surface abrasion and increasing waterproofing.

These objectives can be achieved in the following ways: [39]

1. Increasing density by creating a dense environment which blocks pores and capillary channels.
2. Reinforcing, where the soil is reinforced by the addition of various types of fibres of organic origin, (straw), animal origin (hair, wool), mineral or synthetic origin (synthetic fibres). This approach creates a network of omni-directional fibres, which notably improves tensile and shearing strengths and helps to reduce shrinkage.
3. Cementation: a strong, inert, three-dimensional matrix is introduced into the soil. This causes consolidation by cementation, which coats the particles and resists movement within the material. The main consolidation reactions occur within the stabiliser itself and between the stabiliser and the sandy fraction of the soil. Secondary reactions, however, can be observed between the stabiliser and the clay fraction. Clay affects the efficiency of the stabiliser and can modify the mechanical behaviour of the material.
4. Bonding: an inert matrix is introduced into the soil in the form of clays. Two mechanisms giving the same results are known. The first is an inert matrix formed by the clays where the negative and positive charges of the plate-like clay particles, or their chemical composition, are used to bind them together through the intermediary of a stabiliser, which acts as a binding

agent or catalyst in this binding effect. Certain chemical stabilisers work in this way, including certain acids, polymers and flocculants. The second is where an inert matrix is formed with the clays. A stabiliser reacts with the clay and precipitates a new, insoluble, inert material: a kind of cement. This pozzolanic reaction, notably is obtained with lime. This slow reaction depends essentially on the quantity and quality of the clay present.

5. Water-proofing

Water-proofing involves the reduction of water erosion and of the swelling and shrinkage that might occur as a result of repeated alternate wet-dry cycles. There are two known ways of waterproofing. The first involves the filling of all voids, cracks and crazing with a material which is not water sensitive. This method of stabilisation is particularly well-suited to sandy soils which display good volume stability and which are little affected by water. It can also be used for silty and clayey soils, which demand more stabiliser because their specific surface area is greater. The second method is where a material which expands and seals off access to pores, as soon as it comes into the slightest contact with water, is dispersed throughout the soil. An example of such a material is bentonite.

6. Water dispersal

The state of the interstitial water is modified and the sensitivity of the plate-like clay particles to water is reduced. This process uses chemical products (calcium chloride, acids, quaternary amines or resins) or ion exchange resins to eliminate as much absorption of water as possible.

## 2.2 SOIL STABILISATION METHODS

Soil stabilisation can be achieved by a variety of methods, which include mechanical stabilisation, physical stabilisation and chemical stabilisation. [33,52]

### 2.2.1 Mechanical Stabilisation

The strength and durability of a soil can be increased by mechanical means. The mechanical method of stabilisation involves soil compaction and densification by application of mechanical energy. A clear relationship exists between dry density and mechanical strength. The more compact a material, the higher its mechanical strength [27,49,52]. For example, given the same composition and dosage, the denser concrete is, the stronger it will be. Similarly, the densest stones have the highest mechanical resistance. The same applies to earth, only the densities achievable remain relatively low.

The main effect of compaction is to tighten up the soil particles which results in:  
[27]

- An increase in the number of points of contact between soil particles;
- A reduction in the proportion of spaces, that is in the porosity of the soil.

For clay soils, the small size of the particles and their wide specific surface area make them behave in a particular way. Depending on the relative importance of the forces of attraction and repulsion, two fundamental types of structure can be observed: [16]

- A dispersed structure indicating predominantly repulsive forces: the clay plates, which are separated one from the other, tend to be parallel to each other.
- A flocculated structure indicating predominantly attractive forces: the clay plates draw together and form sharp angles between themselves (figure 2.1).



Dispersed structure



Flocculated structure

In these conditions, the effects of compaction differ widely according to the structure: a flocculated state generally corresponds to lower water contents, whereas a dispersed state is characteristic of high water contents [12]. In particular along the compaction curve the structure could be more dispersed to the right of the optimum and more flocculated to the left (figure 2.2).

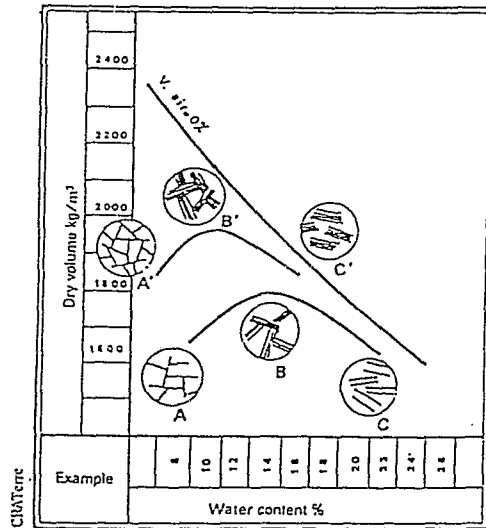


Figure 2.2: Structure

In A, the particles come together and tend to flocculate: as the water content increases they disperse and adopt more regular patterns (B and C). The optimum compaction thus appears to be at a state when the forces of attraction remain sufficient to allow good compaction whilst the forces of repulsion facilitate a certain ordered arrangement of the particles. [16]

The effects of compaction carried out under good conditions can be observed in a reduction in permeability, compressibility, water absorption and swelling, as a result of water in humid conditions and in an increase in mechanical strength.

### 2.2.2 Physical Stabilisation

Physical stabilisation involves the controlled mixing of various fractions of soil to obtain a soil with a particular grain size distribution. Limits have been established which indicate optimum particle size distribution for soils for specific uses. [19] These limits are intended to act as a guide to the amount of each constituent of the soil that should be present, i.e. the amount of sand, silt and clay. These limits are presented in the form of two curves on a graph. [16] If the soil is considered suitable, its particle size distribution curve will fall in between the curves. Therefore if the curve of a soil is not partly or totally within these limits it is possible to mix it with another which has a high content of the elements in which it is deficient in order to obtain a satisfactory product.

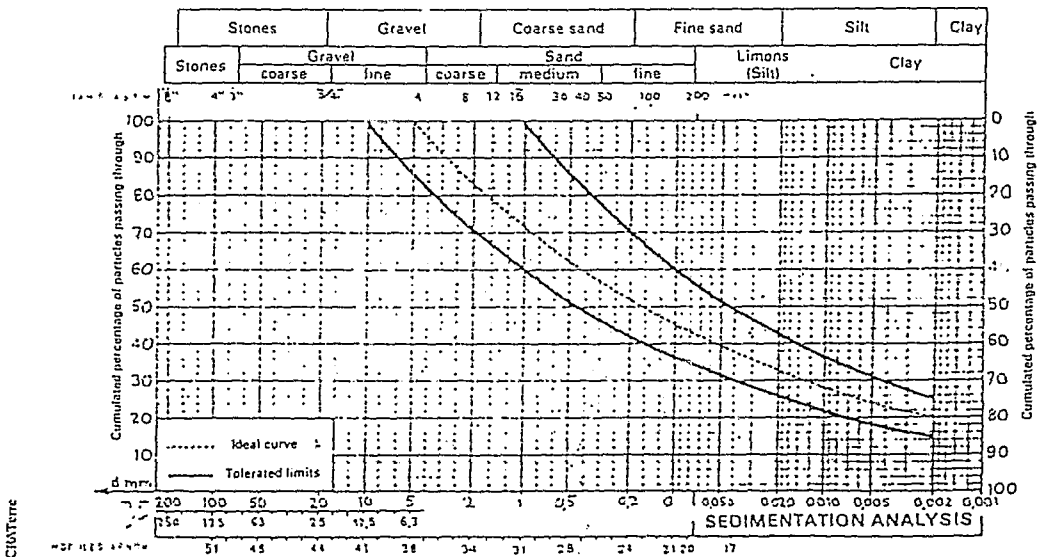


Figure 2.3: The ideal curve for stabilised soil and its limits of tolerance [16]

### 2.2.3 Chemical Stabilisation

Chemical stabilisation is the addition of various chemicals to a soil to modify its properties, either by a physico-chemical reaction between the grains of the soil and the added product, or by creating a matrix which binds or coats the grains. [24] A physico-chemical reaction can lead to the formation of a new compound: for example a pozzolana resulting from a reaction between clay and lime. Whereas the changes, which are effected as a result of mechanical stabilisation, can be undone by moisture i.e. mechanical strength, the effects of chemical stabilisation are permanent. [40]

#### Chemical Stabilisers

As described there are various effects, which can be achieved by the addition of various chemicals to a soil. The following are the stabilisers which may be used:

#### Lime

There are three main types of lime: high calcium limes (referred to hereafter as "limes"), hydraulic limes and dolomitic limes.

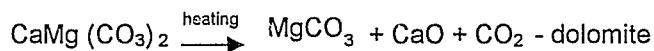
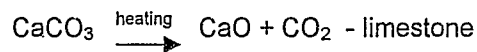
High calcium lime is obtained by calcination at 900°C to 1000°C of a relatively pure, clay-free calcareous rock. A calcium carbonate content of 97% would normally be considered the minimum for production of high calcium limes. During baking, this carbonate dissociates and part of it escapes in the form of carbon dioxide. This leaves only calcium oxide, known as quicklime, the proportion of which determines the quality of the final product. Pure high calcium lime will contain more than 97% CaO; fat lime, more than 85%; and lean lime, less than 85%. When it is placed once more in contact with the carbon dioxide present in the atmosphere, the lime again becomes calcium carbonate.

Hydraulic limes gain greater strength, and at a faster rate, than high calcium limes. They also set under water and produce a generally more durable product. Their behaviour, the way they work and their characteristics closely resemble those of lower strength Portland cements. [55]

Dolomitic lime is similar to a high calcium lime except that it contains a proportion of magnesium oxide (MgO) combined with calcium oxide (CaO). This lime needs more care in usage compared with a high calcium lime. It is particularly important to ensure that it has been properly burned and slaked, that is, it is burned at a lower part of the temperature range for lime production and a longer time is allowed for slaking it. [5]

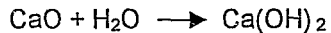
There are two main forms of lime:

Quicklime (CaO): quicklime is produced directly by firing limestone (CaCO<sub>3</sub>) or Dolomite [CaMg (CO<sub>3</sub>)<sub>2</sub>] in a kiln;



Quicklime is extremely hygroscopic (i.e. it attracts water) and must be protected from moisture. The sensitive conditions of storage and maintenance it requires can limit its use. It is a material which is highly exothermic in its reaction with water reaching temperatures of up to 150°C during the hydration stage.

Hydrated lime [Ca(OH)<sub>2</sub>] : Hydrated lime is obtained by combining quicklime (calcium oxide) with water under controlled conditions;



The hydration reaction, also known as 'slaking', is exothermic and is accompanied by an increase in volume. Hydrated lime has less available CaO per unit mass than unslaked lime and is therefore less reactive and easier to store. [2]

### Lime-clay Reactions

When lime is mixed with a moist clay soil, two basic though complex reactions apparently take place: [2,5,17,69]

- Cation exchange reactions and flocculation of the clay which begin almost instantaneously and produce rapid changes in the soil's plasticity and workability – referred to as *modification reactions*.
- Depending on the characteristics of the soil being stabilised, a clay-lime pozzolanic reaction may occur. The pozzolanic reaction, which is time dependent, results in the formation of various cementing agents, which increase the strength, and durability of the material – called *cementation*.

### Ion exchange and flocculation

Clay particles adsorb cations to neutralise the negative charge resulting from isomorphous substitution. The adsorbed cations, together with other influences such as pH and electrolyte concentration, influence the amount of water attracted to a clay particle, and thus the interaction between clay particles. [18]

These adsorbed cations are exchangeable, i.e. they can be replaced by other cations with the general rule for replaceability being that multivalent cations are adsorbed in preference to monovalent cations. By replacing adsorbed cations with cations of another type, the properties and amount of water surrounding a clay particle can be altered and therefore replacing adsorbed cations with cations of another type can modify the properties of the clay.

A simplified explanation for the modification of clay by lime [13] suggests that:

- The  $\text{Ca}^{2+}$  ions from the lime water solution replace the  $\text{H}^+$  ions and other monovalent ions such as  $\text{Na}^+$  ions adsorbed to the clay.
- The  $\text{Na}^+$  ions each 'hold' 89 molecules of water when hydrated while the  $\text{Ca}^{2+}$  ions only hold 2 molecules each, and thus the quantity of water attracted to the clay particle is vastly reduced.
- If sufficient lime is added to the clay, overcrowding of the  $\text{Ca}^{2+}$  cations onto the clay particle surfaces results in the particles becoming flocculated forming silt sized agglomerations within which clay particles adhere to one another.

The ion exchange and flocculation processes materially change the number of electrical charges on the surface of the clay particle. Because the bond between the clay particles is dependent on the charge and the size of the ions, the preponderance of the divalent calcium ions that have replaced the univalent ions attract the soil particles together. The ion exchange reactions and the flocculation of the clay begin almost immediately, resulting in most cases in an increase in the plastic limit, at the same 'in-situ' water content. [40] Generally, the plastic limit increases, while the liquid limit either remains unchanged or increases or decreases slightly. The lime treatment makes the soil more open textured and the material becomes more friable and more workable. [24]

The decrease in interparticle repulsion resulting from ion exchange of for example  $\text{Ca}^{2+}$  for  $\text{Na}^+$  can result in flocculation of clay particles, where the individual particles aggregate to form larger 'floc units'. The result is in effect an increase in the size of 'particle', which will remain stable, i.e. will not disperse into smaller particles, when placed in water.

Ingles [41] defines flocculation as the 'bridging' of clay particles into loose random structure (figure 2.4). The randomly oriented structure is more porous (and hence has a higher permeability) than the unflocculated structure. The bridged structure resists shear and thus yields a lower compacted density for the same compactive effort. He describes flocculation as "effectively converting the clay to the mechanical equivalent of a fine silt".

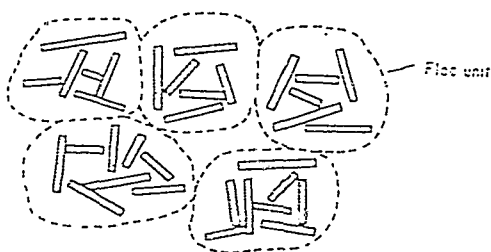


Figure 2.4: Floc Units in Random Orientation

The changes, which take place in the Atterberg limits of a montmorillonite clay soil with the addition of lime, are a significant reduction in plasticity resulting from an increase in the plastic limit and a reduction in the liquid limit.[37]

The increase in the plastic limit can be ascribed to the flocculation of the clay, which behaves more like a granular material. Hilt and Davidson [38] explain the rise in plastic limit as being the result of additional water being required to break the stronger bonds between clay particles within the flocs. In other words, a higher water content creates a more dispersed structure, which is easier to remold.

The stabilised material still exhibits a liquid limit i.e. some cohesive strength, but will exhibit little plasticity because of the overriding influence of weakly attracted silt size flocs. Although the soil may appear to be non-plastic, this does not mean that the plastic limit is zero, but that the plastic limit is very close to the liquid limit or cannot be measured. The higher the clay content (untreated), the further the plastic and liquid limits diverge from one another because of the increasing influence of the clay.

Thus as the lime content is increased, more clay becomes flocculated, reducing the range of water over which the soil will behave plastically. The lime itself may also contribute to some extent to the raising of the plastic limit when it hydrates, i.e. additional water is required for the hydration of the lime.

The same trend cannot be expected for all soils, and laboratory determinations of the Atterberg limits must be conducted to confirm any assumed changes. Kaolinitic clays often show increases in the liquid limit with increasing lime content, sometimes showing increased plasticity. [40] However, the increased plasticity can be shown to result when the liquid limit increases by a greater amount than the plastic limit, i.e. the liquid and plastic limit curves diverge with increasing lime content.

The quantity of lime required to complete the ion exchange and flocculation reactions (known as the 'initial lime demand' or 'lime fixation capacity'), the rate of the reaction and the extent to which the properties of the clay are altered by the reaction, are dependent on the clay type. [61]

Kaolinites have low cation exchange capacities and thus low initial lime demands (1 to 2% lime by dry weight). The ion exchange and flocculation reactions are normally completed fairly rapidly i.e. in less than 24 hours. Montmorillonite clays have high cation exchange capacities and the initial lime demands may range from 5% to as high as 12%. The ion exchange and flocculation reactions occur at a slower rate and may take some days to complete. [18]

Because of its small particle size and the expanding lattice structure of these particles, montmorillonite is notorious for the large volume changes which, occur when it is wet or dried. On the other hand, kaolinite is relatively insensitive to changes in water content and does not swell and shrink very much.

### Cementation

After the short term ion exchange and flocculation reactions have occurred, any excess or 'free' lime reacts chemically with the clay minerals or with any other fine pozzolanic component such as hydrous silica or alumina. These reactions produce "tough, water insoluble" hydrated calcium silicate and aluminate gels, which cement the soil particles. [17,18,37,40,69]

The reaction products are similar to those present in Hydrated Portland cement (i.e. calcium silicate hydrates). However, the hydration of cement is independent of the soil type, while with lime the gel is formed only after attack and removal of silica from the clay minerals of the soil. [40,69]

The type and amount of minerals in the soil that react with the lime (known as pozzolans) vary for different soil types and determine the long term properties of the soil-lime. Possible sources of silica and alumina include clay minerals, quartz, feldspars, micas and other similar silicate or alumino-silicate minerals.

In addition to the formation of calcium and aluminium silicates, it is also believed that the lime reacts chemically with surfaces of the clay minerals [69]. The same reference suggests the possibility that cementation occurs as a combination of the two processes.

These reactions occur only under a condition of high alkalinity ( $\text{pH} > 11$ ) [2,19,38,49,69,74] at which the solubilities of silica and alumina are greatly increased. This condition is achieved by adding sufficient lime (the pH of lime

saturated water is approximately 12.4) to ensure that the pore water remains calcium saturated for the projected duration of the pozzolanic reactions. Ingles and Metcalf [40] note that the reactions proceed only whilst water is present and able to carry calcium and hydroxyl ions to the clay surface. The reaction thus ceases on drying (just as cement hydration would cease on drying), and very dry soils will not react with lime. Ingles and Metcalf [40] describe the cementing mechanism as shown in figure 2.5 whereby the gel immediately begins to coat clay lumps in the soil and to block off the soil pores. In time the gel crystallizes on the sides of the clay lumps into calcium silicate and aluminium hydrates. Water for the reactions is 'withdrawn' from the soil pores until there is insufficient water in the voids and the reaction ceases. Correct emphasis is placed on the fact that clay lumps are stabilised on their outside surfaces only. Thus together with the unused lime and the weaker uncrystallized gel, the interior of the lumps represents local weaknesses in the 'matrix' of stabilised soil. Natural soils contain both granular (silt, sand and gravel) and clay materials. Some of the granular particles may be capable of reacting with the lime to form cementitious gel. It follows logically that there must be sufficient lime reactive clay or other materials to produce sufficient cementing material to both encase the clay lumps and to bind granular particles together at points of contact. [41]

Although the cementing reactions may begin immediately, the rate of reaction is very slow, and it may take some years before sufficient crystalline products form to produce strengths comparable with soil cement.

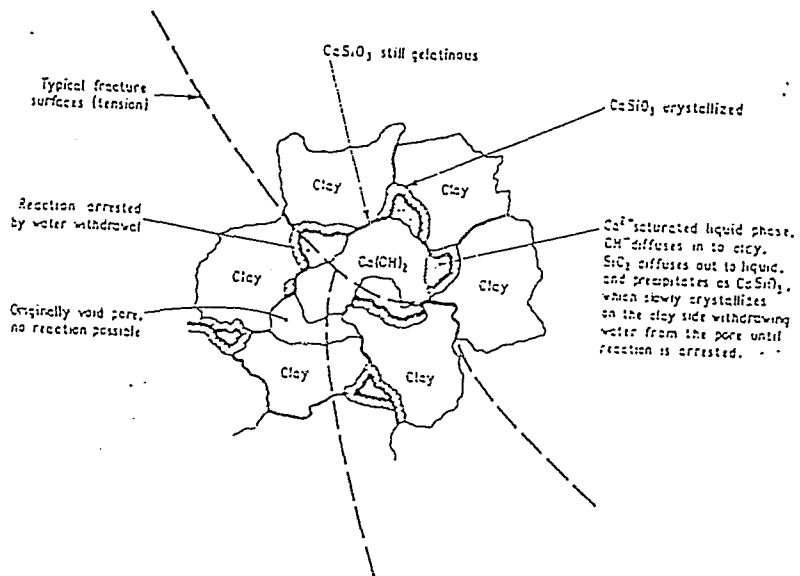


Figure 2.5: Mechanism of Lime Stabilisation of Clay soils. [40]

## Portland Cement

Ordinary Portland Cement (OPC) is manufactured by burning a mixture of calcareous, argillaceous and/or other silica, alumina or iron oxide bearing materials at a temperature of some 1450°C. The clinker (material leaving the burning zone) is then interground with small proportions of gypsum. [22] Many other materials can be used in the manufacture of Portland cement but the addition of these materials is restricted by economic and physical requirements set by various specifications.

Compound	Formula	Abbreviation	% by mass in cement
Tricalcium silicate	$3\text{CaO} \cdot \text{SiO}_2$	C <sub>3</sub> S	35 – 55
Dicalcium silicate	$2\text{CaO} \cdot \text{SiO}_2$	C <sub>2</sub> S	20 – 40
Tricalcium aluminate	$3\text{CaO} \cdot \text{Al}_2\text{O}_3$	C <sub>3</sub> A	5 – 12
Tetracalcium Aluminoferrite	$4\text{CaO} \cdot \text{Al}_2\text{O}_3 \cdot \text{Fe}_2\text{O}_3$	C <sub>4</sub> AF	5 – 10
Magnesia	MgO	M	0.3 – 0.4
Gypsum	Raw material	-	4 – 7
Free lime	CaO	-	0.5 – 2.5

Table 2.1 – Compound Composition of South African Portland Cement. [22]

Portland cement is an extremely complex mixture of compounds.

These are:

- a) Tricalcium silicate: this contains all the essential properties of Portland cement. It is chiefly responsible for the early strength build up of cement-water pastes.

- b) Dicalcium silicate: this hardens slowly and is responsible for most of the strength build up of Portland cement after 28 days.
- c) Tricalcium aluminate ( $C_3A$ ): this liberates large amounts of heat during the first few days of hardening. It accounts for the initial set of the material and contributes slightly towards the initial strength of the material. It is largely responsible for the susceptibility of the matrix to sulphate attack because the greater the tricalcium aluminate content ( $C_3A$ ) in the cement, the lower the sulphate resistance.
- d) Tetracalcium aluminoferrite ( $C_4AF$ ) : this hydrates rapidly and helps in developing a little strength in the material. It is mainly used as a flux during manufacture.
- e) Magnesia: this is a minor constituent of most cementitious materials. If it is present in excessive quantities and the structure is exposed to moist conditions, it may cause expansion of the cement paste and therefore lead to deterioration of the structure of the concrete after it has hardened. The magnesia reacts with water but will only be deleterious if the magnesia is present in the crystalline form. [22]
- f) Gypsum: This is added to the clinker to control the rate of setting of the cement. " When gypsum ( $CaSO_4 \cdot 2H_2O$ ) is present, some of this gypsum dissolves in the gauging water and reacts with the hydration products of  $C_3A$  forming an insoluble calcium sulphate aluminate called ettringite. This forms a skin around the individual  $C_3A$  crystals, but as it is formed, it tends to trap a little water between itself and the  $C_3A$ . This water reacts with the  $C_3A$  giving an increase in volume. This results in the bursting of the skin, which is then again sealed off, by a new skin but again trapping some water. In this manner the hydration of the  $C_3A$  is controlled, giving the user a chance to work with the cement before it sets." [76] When this happens in hardened concrete damage can occur.(see above)
- g) Free lime: this is present due to an incomplete reaction in the kiln. If it is present in excess of four percent, it can cause the cement to be unsound (excessive volume changes) and lead to a deterioration of the structure. This

however is only relevant if the volume changes take place after the concrete is no longer plastic. If it is present as calcium hydroxide, it has no deleterious effect. Free lime is regarded as an undesirable constituent of cement as it contributes to the susceptibility of the matrix to attack by aggressive agents such as acid and pure water. (ibid.)

### Types of cements

The specifications for cement have recently been changed in South Africa to fall in line with the specifications being used in Europe. The table 2.2 details the new types of cements in the specification and their equivalents in the old specifications. The most common cement used in soil stabilisation is ordinary Portland cement although other cements such as sulphate resisting cement, portland blastfurnace cement or masonry cement are used to stabilise soils [2]. Rapid hardening cements are not readily used as they do not produce a particular improvement when compared to ordinary portland cements and are more expensive. [2,39]

### Cement Stabilisation

#### Soil-cement reactions

When water is added to the soil-cement mix the cement starts to hydrate, the reactions, which occur are similar to those presented in the case of clay-lime: [64]

- The hydration of the calcium silicates and aluminates release lime as  $\text{Ca}(\text{OH})_2$ . This lime and the free lime (as  $\text{CaO}$ ) present in the cement react with the clay in plastic soils as described in clay-lime reactions resulting in a fairly rapid reduction in plasticity and an increase in strength (ability to resist external forces, stress or pressure exerted on it).

The major contribution to the strength of soil-cement comes however from the hardened cement which bonds the soil particles together providing early and long-term strength, and thus determining the engineering properties of cement treated materials.

Ingles [41] notes that granular soils require cementation at points of contact to maintain their dense state after compaction, while clay lumps must be encased by a 'shell' of cement which provides a moisture barrier and restricts volume changes. He also points out however that a disadvantage of the early strength type of structure is that once the structure has been ruptured it does not readily reform. This implies that high cement contents may be required to effectively stabilise expansive or shrinkable clays.

Nearly all soils except those containing an excessive amount of organic material can be treated with cement, which significantly improves their properties [2,33,39,40,49,66]. Salt-rich soils are also difficult to stabilise with cement, but increasing the proportion cement can often yield good results. Soils with high percentages of clay (>30%) are often difficult to mix and therefore high percentages of cement are required to ensure good distribution of the cement throughout the soil. The addition of a small percent of lime causes the clay to flocculate and become more friable, thereby making mixing easier. Therefore by pre-treating a soil which has a high percentage of clay with lime, the quantity of cement required for stabilisation can be reduced as the addition of lime make the clay soil more friable and better mixing is achieved. [24]

Soils which contain sulphate contents in excess of 2 to 3% are also not suitable for cement stabilisation as the sulphates have very harmful side-effects. Calcium sulphate (anhydrite and gypsum) in particular reacts with the hydration products of  $C_3A$  to form ettringite which is expansive and results in the destruction of the hardened cement from the inside of the cement-soil resulting in an increase in the sensitivity of the clay to moisture. [25]

Cement type	Description	Old product nomenclature	New product nomenclature
I	Portland cement	OPC	CEM I 32.5 CEM I 32.5R
	Rapid hardening cement	RHPC	CEM I 42.5 or CEM I 42.5R
II	Portland slag cement	PC15SL	CEM II/A-S 32.5 or CEM II/A-S 32.5R CEM II/A-S 42.5
		RH30SL	CEM II/B-S 32.5R CEM II/B-S 42.5
	Portland fly ash cement	RH15FA	CEM II/A-V 42.5 or CEM II/A-V 42.5R CEM II/A-W 42.5 or CEM II/A-W 42.5R
		PFAC	CEM II/B-V 32.5 or CEM II/B-W 32.5
		PC15FA	CEM II/A-V 32.5 or CEM II/A-V 32.5R CEM II/A-W 32.5 or CEM II/A-W 32.5R
III	Blastfurnace cement	PBFC	CEM III/A 32.5 or CEM III/A 32.5R
		RH40SL	CEM III/A 32.5R or CEM III/A 42.5R

**Table 2.2: Common cement types**

## Bitumen

Bitumen is a product made up of a filler and at least 40% heavy hydrocarbons. In its natural state, bitumen is too thick to be usable. It is either heated, or a "cutback" or emulsion is prepared. For stabilisation purposes, these two methods are used. [25]

"Cutback" is obtained by mixing the bitumen with a volatile solvent such as diesel fuel, kerosene, or naphta. Emulsions are obtained by dispersing bitumen in water with the help of an emulsifier, which is either of anionic or cationic. The emulsions from the latter are more common and are compatible with many soil types.[25] Emulsions are very fluid and are easily mixed with pre-moistened soils.

Cutback and emulsions come in the form of microscopic droplets in suspension in a solvent or water. The stabiliser is mixed into the soil. And when the water or solvent evaporates the droplets of bitumen spread out to form strong, very thin films, which adhere to and coat the soil particles. Bitumen improves the water-resistant properties of the soil (less absorption by clays) because it is intrinsically hydrophobic and can improve the cohesion of naturally non-cohesive soils [25]. Organic matter and sulphates in the soil hinders the efficiency of bitumen stabilisation as their adhesion to the grains prevents the adhesion of the bitumen. Mineral salts are very harmful. When bitumen-stabilisation is carried out on an industrial scale, salt contents of more than 0.2% are not accepted, an exception is sodium chloride (NaCl), where up to 6% can be accepted. [55]

## Synthetic Products

Certain synthetic products can also be used for soil stabilisation for earth construction. They consist of either synthetic products, or of by products. They are also often industrial wastes.

The following are some of the products which are used: [25]

1. Acids : Acids modify the pH of the soil resulting in flocculation, the effects of which are often reversible. Some acids act as catalysts to form insoluble phosphates. Hydrochloric acid and nitric acid result in moderate stabilisation. Hydrofluoric acid is very effective in all soils except those with a high aluminium contents, inducing a reaction which brings about the formation of insoluble and strong silica fluorides. If phosphoric acid is incorporated, a hydration reaction is set off with the formation of phosphoric anhydrides which reacts with clayey minerals, and the creation of an insoluble gel of aluminium and iron phosphates, which cement the grains together.
2. Sodas: Sodas induce cementation by reacting with minerals, which produce insoluble silicates and aluminates. Caustic soda acts as a dispersant by degrading the minerals by alkaline attack. Barium hydroxide, calcium hydroxide, potassium hydroxide and lithium hydroxide are often used. [78]
3. Salts: Salts acting on soils induce colloidal reactions, alter the characteristics of the water and lead to flocculation. By increasing the attraction between fine soil grains, salts help to create larger particles. This flocculation reaction leads to a reduction in density and an increase in optimum moisture content, in permeability and strength, but also to a reduction in plasticity. Salts act on pore water and reduce the loss of water from the soil, slowing evaporation, and reducing water absorption of soil treated. The effectiveness of treating soil with salt depends on the extent of moisture movements in the stabilised soil. The treatment is not always permanent as the salts may be leached out and dissolved when the soil is moistened. The four main salts used are:
  - a. sodium chloride: flocculant and compaction aid; effective in non-saline soils
  - b. calcium chloride: impermeabilizing agent;
  - c. ferric chloride: powerful coagulant and flocculant;

- d. aluminium chloride: electrolytic coagulant, electrochemical consolidation of soil.

Salts should never be used in conjunction with cement. [32]

4. Silicates: Sodium silicate acts as an impermeabilizing agent, particularly where a surface treatment of the materials is required. Silicates are generally used with soils which lack cohesion. Sodium silicates are highly soluble, but can be rendered insoluble by allowing them to react with slaked lime. Other silicates, which can be used, are potassium silicate and calcium silicate.
5. Stearates: Stearates are salts or esters of the stearic acid contained in animal fats. They act as impermeabilizing agents. Aluminium and magnesium stearates as well as zinc stearate may be suitable.
6. Paraffins: Paraffins are a mixture of solid saturated hydrocarbons characterised by their inertness in the presence of chemical agents. They can be used as a compaction agent but must first be dissolved in a fatty medium.
7. Waxes: Industrial waxes can be used as an aid to compaction. They are often added to other stabilisers.
8. Latexes: Industrial latex emulsified in water and added at the rate of 3 to 15% can give good results. These products are binders and impermeabilizing agents.
9. Industrial wastes:
  - a) Blastfurnace slag – these are siliceous/aluminous slags which can approach Portland cement in composition. They are often used with ordinary Portland cement as an additive in the form of a latent hydraulic binder as an extender for the cement.
  - b) Lignin and Lignosulphates – these are by-products of the paper pulp industry. Soluble in water, they can be rendered insoluble by mixing with chrome salts (sodium

or potassium bichromate) resulting in a thick gel, known as chromolignin. They are good water proofing agents.

- c) Molasses – a product of the sugar industry, this improves compressive strength and reduces water sensitivity but cannot be used with cement.
- d) Pozzolanas: These are materials, which contain silica, and/or alumina, e.g. fly ash (from power generating plants using coal), burnt clay (rejects from brick production) and rice husk ash. They are not cementitious themselves, but when finely ground and mixed with lime, the mixture will set and harden at ordinary temperatures in the presence of water, like cement.

The use of these products often depends on their local availability. The majority of these products usually have a high unit cost, which limits their extensive use. The most commonly used stabilisers are cement and lime, due to their availability and to a lesser extent bitumen. [39]

### 2.3 Soil Suitability

"Immense experience has been acquired with building with earth in recent times. It is, however, far from being complete. The criteria for the suitability of a soil currently in use are far from final and should not be too literally interpreted." [39]

The suitability of a soil as a raw material for earth construction is a function of a number of factors. As discussed in chapter one, soils have a number of properties, which affect their behaviour and therefore their performance as a construction material. To successfully stabilise a soil, proper understanding and knowledge of the soil sample must be sought. Due to variations in soil types, various soils will require different quantities of stabilizer (cement, lime etc.) to achieve a satisfactory building material. Therefore, before using a specific soil as a building material a comprehensive study must be conducted to determine these properties and enable the prospective user to understand the material and to predict its behaviour in the final product.

As a result of extensive work originally done in the field of road construction, procedures to determine these properties have been established. Although stabilised soil in road construction differs in its application to its use in the construction of buildings, many of the testing procedures performed on stabilised soil, for road construction, are relevant to the material used for building construction for example grading analysis, liquid limits and plastic limits. [60] Relevant research undertaken in road construction has therefore in addition to more specific research conducted in the field of earth constructed buildings been used to produce suitability criteria and reference nomograms which act as guidelines. These guidelines enable the user to make a decision with respect to the suitability of a soil as a construction material, based on its properties.

The laboratory tests conducted on the soil to determine the properties of the soil investigated in this report are detailed in chapter 3.

The general guidelines, which have been set out for the suitability of a soil according to its properties have been suggested by CRATerre-EAG, the International Centre for Earth Construction and School of Architecture at Grenoble, which has accumulated a vast knowledge of earth construction through experience over twenty years in more than fifty countries and is considered to be the world leader in earth related construction. These guidelines have also been incorporated into a standard known as the CIB W90 document, which has been established for use with compressed earth blocks. The standards are the fruit of the work of committee W90 (Compressed Earth Block Technology) of the International Council for Building Research Studies and Documentation (CIB). The guidelines indicate what the particle size distribution and plasticity of the soil should be for the soil to be considered a suitable soil.

#### Guidelines for soil suitability (CIB W 90)

The particle size distribution and plasticity of the soil should preferably fall within the limits of the shaded area on the appropriate diagram which follows. (figure 2.6 and figure 2.7).

"The limits of the recommended zones are approximate. Acceptable tolerances may vary widely. The current state of knowledge does not permit precise limits to be applied. It is generally agreed that many soils which do not fall within the recommended zones, give acceptable results in practice. However, soils which do conform to these, in most cases give satisfactory results. The zones indicated are intended to guide the user and not be applied as rigid specifications." [39]

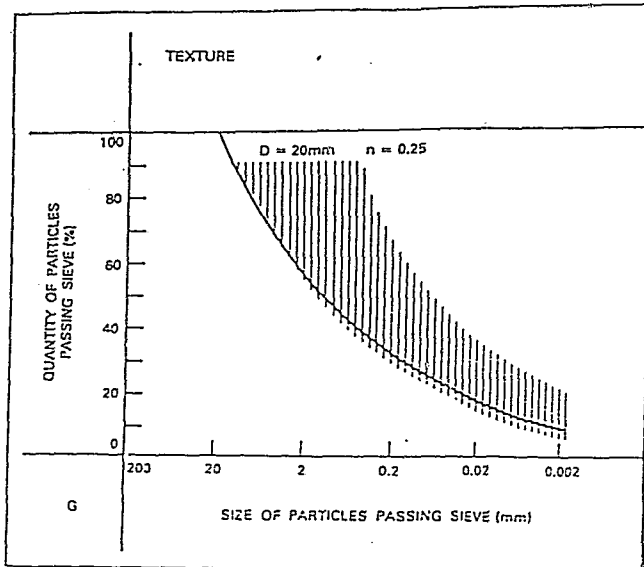


Figure 2.6: Recommended Particle size distribution

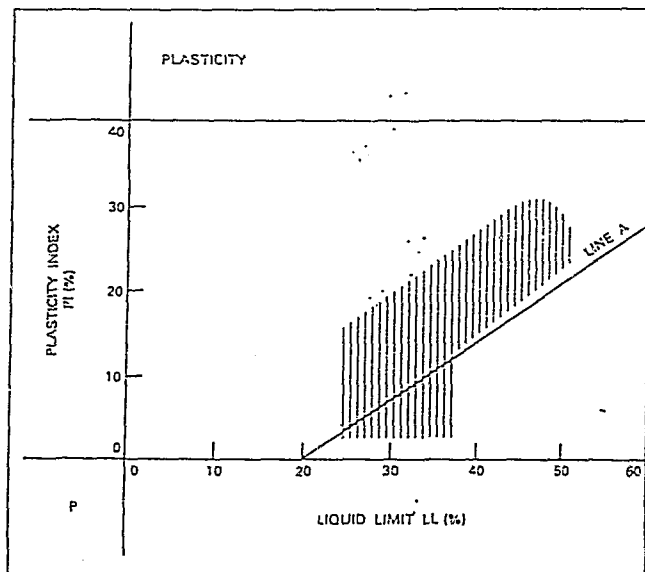


Figure 2.7: Recommended Plasticity Limits

## 2.4 The need for and practice of soil stabilisation

Soil stabilisation as a science started to evolve about 60 years ago. [24] The growth of soil stabilisation has economic roots. Its economic importance arises from the possibility of improving soils at a site of construction for use as foundations or as materials of construction. If an unstable soil has to be removed and replaced with other foundation material such as gravel or crushed rock the cost can be high owing to the weight and bulk of the materials involved. Similarly if soil or stabilised soil can be used as a construction material transportation costs are greatly reduced as fewer building materials have to be transported to the site. Soil stabilisation has been used in the building of roads and aircraft runways, earth dams and embankments, in erosion control, in the reduction of frost heaving and for the construction of buildings.

When a project is encountered which involves the use of soil as a construction material and the soil on the site is considered unsuitable, possible alternatives are: [33]

1. Avoid the particular site. Relocate a planned highway or development site.
2. Design the planned structure accordingly.
3. Remove and replace unsuitable soils.
4. Attempt to modify the existing ground. (soil stabilisation)

As more and more land becomes subject to urban or industrial development, good construction sites and borrow areas are difficult to find and the soil improvement alternative more frequently becomes the best option, technically and economically.

Soil stabilisation techniques are also increasingly being applied in the rehabilitation of hazardous-waste disposal areas. There emphasis may be on the fixation of dangerous pollutants, but at the same time a geotechnically

engineered design should impart a minimum strength and volume stability to the waste mass. Of increasing concern to environmentally conscious engineers is the constructive use of high-energy waste materials such as slag and fly ash. Part of this concern arises from the estimated increase in ash production in the years to come. Current costs of fly ash disposal are staggering, considering that fly ash could be used in combination with lime or cement, as a stabilising agent for use with soils and as a cement extender in concrete construction.

The current housing backlog in South Africa is an area which may benefit from the application of soil stabilisation techniques. This is especially true in the more rural areas where the inhabitants often live a distance away from suppliers of conventional building materials. The ability to use soil from the site on which these people live, as a reliable construction material could be a suitable solution to some of the housing problem. The popular use of corrugated iron roof sheets as a walling material provides these homes with highly undesirable thermal conditions due to a lack of adequate thermal insulation in both summer and winter conditions.

## **2.5 Earth construction techniques**

There are numerous methods which have been used and are currently being used for the construction of buildings using earth. Among the most widely known and practical construction methods are rammed earth in formwork, bricks moulded in raw earth and baked by the sun or 'adobe' as it is known, and compressed earth blocks [39]. The rammed earth and adobe methods are discussed briefly with a more in depth discussion of compressed earth block technology, as it is the technique adopted in this research.

### **2.5.1 Rammed earth**

Pisé or rammed earth construction makes use of formwork in a similar way to that for casting in-situ concrete. Two shutter boards are erected parallel to each

other, with a gap of approximately 300mm between them (this gap may vary depending on the wall strength required). Hollow spacers are placed between the panels to ensure that the required gap is maintained. Steel rods are placed through these spacers, connected and clamped up against the formwork from both sides. This prevents the formwork from bulging. The moistened soil or soil-stabilizer mix is then placed between the formwork in layers of approximately 100mm loose material and rammed either manually or mechanically. [53] It has been found that if layers of greater depth than 100mm are rammed, the lower part of each layer is insufficiently compacted. (ibid.) As each short length is completed, the shutters are unclamped, advanced and reclamped until a complete layer, the length of the wall being constructed, has been formed. Shutters are then raised to cast the following layer. This form of construction was investigated by Roberg. [60] Roberg investigated the construction of walls using three soils found in the Witwatersrand area. He showed, using a scientific investigation, that the soils stabilised with cement could be used to produce rammed earth walls which exhibited sufficient strength and durability to confidently be used in the construction of a single storey building.

### **2.5.2 Adobe construction**

Adobe construction is a technique which uses sun dried clay bricks in the construction of buildings. To manufacture adobe bricks, small amounts of water are added to the soil which is mixed with the aid of a shovel and by treading on the material with bare feet until a homogenous dough-like consistency is obtained. Often short pieces of straw are added to increase the tensile strength of the bricks and to avoid excessive cracking of the bricks on drying out (if straw or other fibres are added it is known as cob, whereas if just soil is used it is known as adobe). Once the material is thoroughly mixed, the mixture is tamped into wooden moulds and the excess material removed from the top surface of the mould. The mould is then removed and the bricks are left to dry. To avoid excessive cracking of the bricks due to rapid drying of the material, various materials may be placed over the bricks to reduce the rate of drying, for example

straw, canvas or plastic sheets. Once the blocks have dried they are then laid in a similar way to fired clay bricks using mud as the mortar.

### **2.5.3 Compressed earth block technology.**

The compressed earth block is the modern descendent of the adobe block. The idea of compacting earth to improve the quality and performance of moulded earth blocks is, however, far from new, and it was with wooden tamps that the first compressed earth blocks were produced. This process is still used in some parts of the world. The first machines for compressing earth probably date from the 18<sup>th</sup> century. In France, Francois Cointeraux, inventor and fervent advocate of "new pisé" designed the "crecise", a device derived from the wine-press. But it was not until the beginning of the 20<sup>th</sup> century that the first mechanical presses, using heavy lids forced down into moulds, were designed. Some examples of this kind of press were even motor-driven. The fired brick industry went on to use static compression presses in which the earth is compressed between two converging plates. But the turning point in the use of presses and in the way in which compressed earth blocks were used for building purposes came only with effect from 1952, following the invention of the famous CINVA-RAM press designed by engineer Paul Ramirez at the CINVA centre in Bogota, Columbia. This was used throughout the world. With the '70s and '80s there appeared a new generation of manual, mechanical and motor-driven presses, leading to the emergence today of a genuine market for the production and application of the compressed earth block. [6]

Since its emergence in the '50s compressed earth block (CEB) production technology and its application in building has continued to progress and to prove its scientific as well as its technical worth. Research centres, industrialists, entrepreneurs and builders have developed an extensive body of knowledge, making this technology the equal today of competing construction technologies. CEB production meets scientific requirements for product quality control, from identification, selection and extraction of the earth used, to quality assessment of

the finished block. This scientific body of knowledge ensures the quality of the material.

The setting up of compressed earth block production units, whether on a small scale or at industrial level, in rural or urban context, is linked to the creation of employment generating activities at each production stage, from earth extraction in the quarries to building work itself. The use of the material for social housing programmes, for educational, cultural or medical facilities, and for administrative buildings, helps develop societies' economies and well being. CEB production can form part of development strategies for the public and the private sector which underline the need for training and new enterprise and thus contributes to economic and social development. This was the case in the context of a programme on the island of Mayotte, in the Comoros archipelago, for the construction of housing and public buildings, a programme today regarded as an international reference. [6] The use of compressed earth blocks, which followed the setting up of an island production industry proved pivotal in Mayotte's development, founded on a building economy generating employment and local added value in monetary, economic and social terms. Compressed earth blocks represent a considerable improvement over traditional earth building techniques. When guaranteed by quality control compressed earth blocks products can very easily bear comparison with other materials such as the sand-cement blocks or the fired bricks.

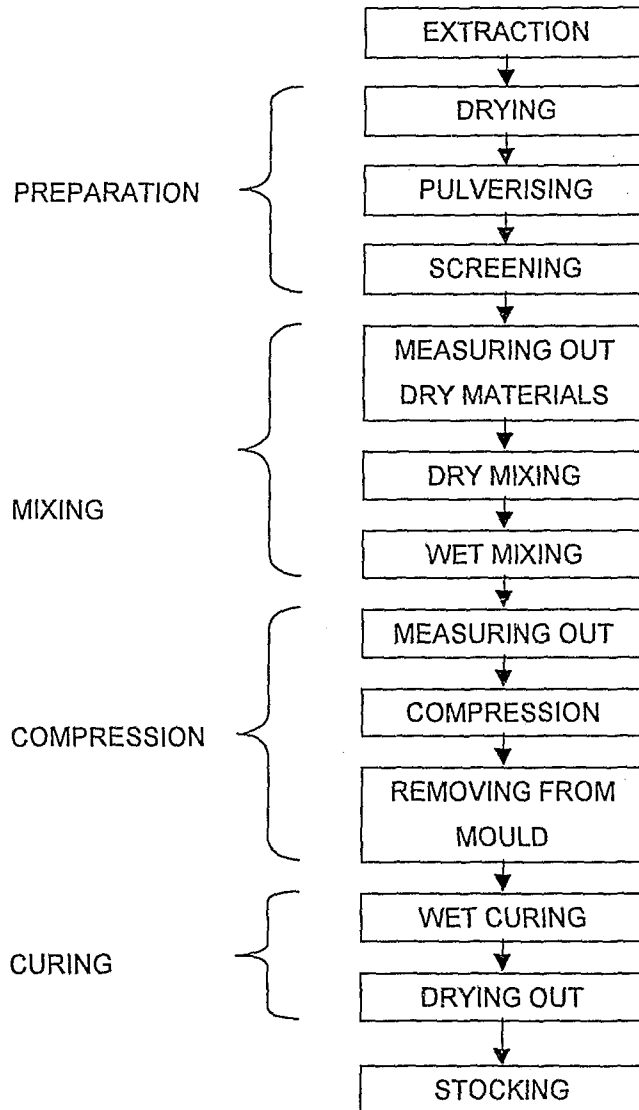
Compressed earth block technology has made great progress thanks to scientific research, to experimentation, and to architectural achievements which form the basis of a wide range of technical documents and academic and professional courses. A major effort is now being devoted to the question of norms and this should help to confer ultimate legitimacy upon the technique in the coming years.

[6]

## Production of compressed earth blocks

The production of compressed earth blocks can be regarded as similar to that of fired earth blocks produced by compaction, except that there is no firing stage.

The following flow diagram describes the production of stabilised soil blocks:



### **Excavation**

This is the starting point of the compressed earth block production cycle. The soil is excavated from a suitable site either manually or by machine.

### **Drying**

After excavation the soil is dried by spreading it in thin layers or passing through a hot-air cyclone.

### **Pulverising**

It is sometimes desirable to break up, crush or pulverise a fragmented or consolidated soil. Pulverisation is often indispensable in the case of compressed, cement or lime stabilised blocks and is largely responsible for ensuring the quality of the product. [39] When a soil is well graded into required distinct grain fractions and sieved, the grains, water and powder stabiliser will be evenly distributed and the product will be of good quality.

### **Screening**

This operation is absolutely essential when:

- I. the removal of excessively large elements or organic matter is required,
- II. after pulverising there are still particles larger than that required.

### **Measuring out dry materials**

The soil, stabiliser and water are measured out by weight or volume according to design parameters.

### **Dry mixing**

This is a particularly important stage in order to maximise the effectiveness of a stabilizer in powdered form by ensuring even distribution throughout the soil. The importance of obtaining a good mix cannot be overstated.

**Wet mixing**

The optimum amount of water is added to the soil-stabilizer mix by either sprinkler, or a mist spray, or by means of pressurised steam.

**Measuring out**

The correct amount of the moist stabilised soil is measured out to ensure the dimensions of the blocks remain the same throughout production.

**Compression**

The mixed material is then compacted using human energy (the manual press) or mechanical energy (motorised presses). The moulding pressure or compactive effort is measured in megapascals (MPa) and is the theoretical pressure applied to the soil expressed as a ratio between the effective force and the area over which it is applied.

**Wet and dry curing**

The concept of curing is discussed in the following section.

### **Curing of stabilised soil blocks**

The curing of stabilised soil blocks is one of the factors which contribute to the strength and durability of the finished product. Curing is the period after the soil and stabiliser mix has been compacted during which principally two types of phenomena occur: [6]

- Physicochemical reactions between the various components of the mix and above all the soil and the additives resulting in the stabilisation of the block: during this phase, humidity and temperature of the curing environment are crucial.
- Drying which consists of the gradual removal, by evaporation, of the moisture needed for the manufacture of the blocks.

For cement or lime stabilised blocks, the presence of water within the block is crucial for the soil-stabiliser mix to attain its maximum strength. Cement stabilised blocks must be kept in a humid environment for at least seven days. [7] The surface of the blocks must not be allowed to dry out too quickly, as this may lead to shrinkage cracks. The blocks must be sheltered from direct sun and wind and kept in conditions approaching 100% relative humidity by covering them with waterproof plastic sheets. The humidity and temperature of the curing environment plays an important role in the rate of curing and strength obtained. For example, compared with 14 days' curing at 100% relative humidity, blocks cured for 7 days at 100% relative humidity and 7 days at 95% relative humidity blocks will achieve 25% less strength. Blocks cured for 7 days at 40°C will be 1.5 to 2 times stronger than blocks cured for 7 days at 20°C. [6]

After curing, a drying period should be allowed so that the water is allowed to evaporate and the clay fraction to shrink. This drying period is to allow the block to shrink before it is built into a structure. To prevent shrinkage occurring too quickly, exposure to wind and direct sun must be reduced.

Curing conditions for blocks stabilised with lime are identical to those stabilised with cement, i.e. hot humid conditions. The period of time, which is suggested, for curing is different, blocks stabilised with cement are cured for a minimum of seven days whereas a block stabilised with lime should be cured for a minimum of twenty-eight days. [16,39]

# CHAPTER THREE

### **3.0 THE SOIL SPECIFIC TO THIS INVESTIGATION**

To successfully stabilise a soil a thorough knowledge of the nature and properties of that soil must be gained. By knowing the properties of the soil, it is possible to make a decision as to its suitability for the production of compressed earth blocks. Chapter 3 details the nature and properties of the soil which was investigated in this report.

#### **3.1 Origin of the soil.**

The soil, which was investigated in this research, is a by-product of the water purification process used at Rand Water's Panfontein purification plant. The water, which is pumped out of the Vaal River has to be purified to make it fit for human consumption. The raw water has silt and clay particles in suspension. These particles have to be removed. The purification process involves the flocculation of these particles by the addition of calcium hydroxide to the water. The calcium ions attach themselves to the silt and clay particles making these particles heavy and causing them to sink forming a "sludge". The clean water and the sludge are then separated. The sludge is pumped out to settling ponds where it dries out to form a soil material. The sludge is produced at about 600 tons per day. At present, there is no use for the sludge and it is simply being stockpiled.

##### **3.1.1 Reasons for the choice of the soil**

The soil is a by-product and is not presently being utilised. As discussed, the sludge from the purification process is pumped into settling ponds where it dries out. Once the water has evaporated from the sludge the settling ponds are filled up with more sludge and allowed to dry, eventually the settling ponds are filled with the dry material. Once a pond is full of dry material it can no longer be used and new ponds are built to accommodate the production of sludge. This requires that new land be purchased to build the new ponds on. The land surrounding the Panfontein plant is agricultural land and as a result of settling ponds being built

on it, it is not being used productively. By finding a use for the dried sludge a number of advantages can be realised. Firstly, the use of a by-product from one process as a raw material for another will eliminate the need for disposal which is often costly and the use of agricultural land for waste management will also be stopped. Secondly, a material, which can potentially be used for the production of compressed earth blocks, is readily available and does not have to be 'mined' as do the materials for fired clay bricks. If a block production plant can be set up to utilise the sludge, jobs will also be created in the local community.

Research has been conducted into the possible use of the dry sludge for the production of fired-clay bricks. However this was not successful as the clay is difficult to extrude and the blocks crack during the initial drying period before firing in the kiln.

### **3.2 Laboratory analysis of the soil**

A laboratory analysis was conducted on the Panfontein soil to determine some of the soil's properties and its mineralogy. The mineralogical analysis was conducted to determine the type of clay mineral present in the soil. The physical properties of the soil were analysed according to standard procedures used in the field of soil science as described in "Technical Methods for Highways 1." Properties that were determined included:

1. Atterberg limits, which consists of the
  - liquid limit,
  - plastic limit,
  - plasticity index
  - and the linear shrinkage of the soil.
  
2. Particle size distribution, which consists of the determination of the grain size of the soil by means of sieve and hydrometer analysis.

Description of the testing procedures for the determination of the Atterberg Limits and the particle size distribution are detailed in Appendix 1.

The mineralogy of the soil was examined using x-ray diffraction analysis.

The results from these analyses were used to determine the suitability of the soil as a material for the production of compressed earth blocks and to classify the soil.

### **3.2.1 Results of the Atterberg Limits tests**

The following table shows the results of the Atterberg Limits tests of the soil used in this report.

LIQUID LIMIT	64
PLASTIC LIMIT	29
PLASTICITY INDEX	35
LINEAR SHRINKAGE	15.5
PI OF THE WHOLE SAMPLE	35

The values obtained from the Atterberg Limits are used to classify the soil.

### **3.2.2 Particle size analysis**

A combined analysis was used to determine the soil grain size distribution. Such an analysis is a routine way of classifying soils, but grain size is not as useful as it was once believed it may be. [26]. The grain size curves of coarser grained soils provide more useful information than those of fine-grained soils.

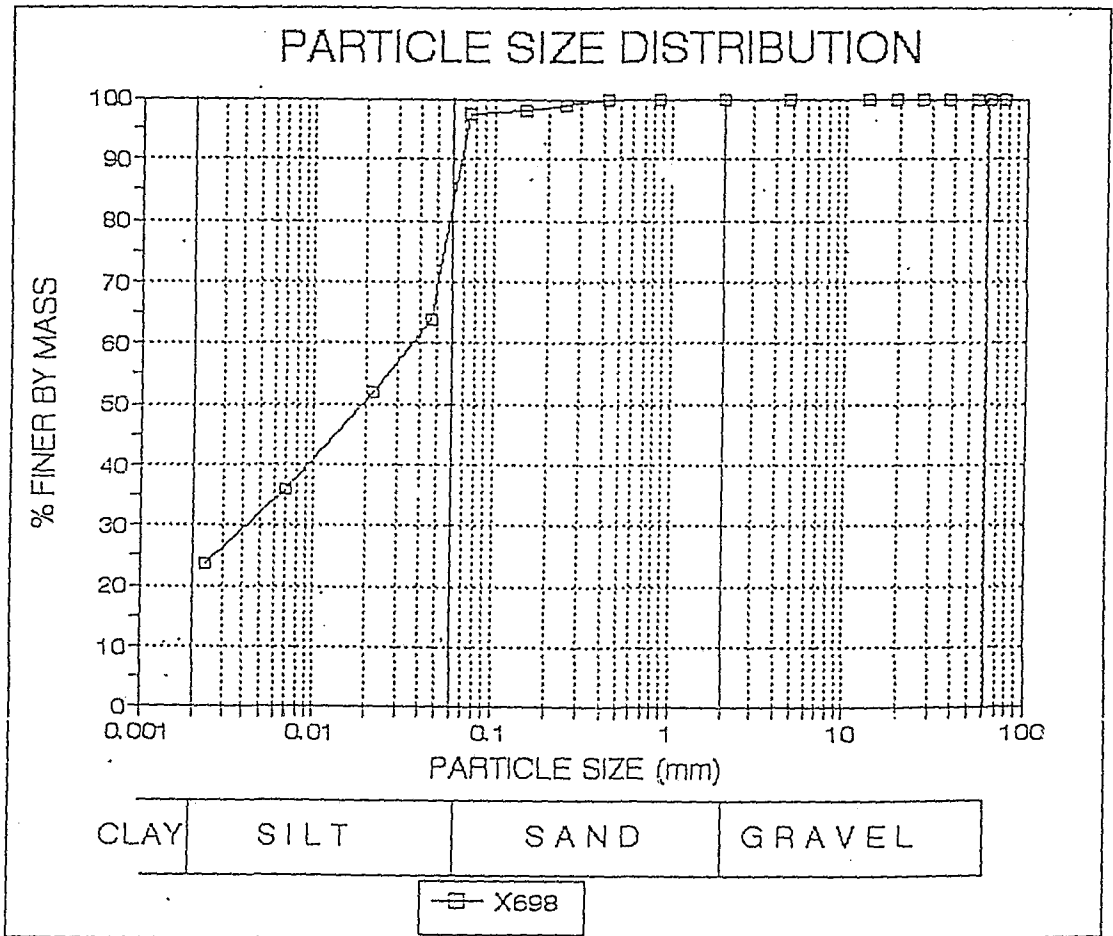
The analysis is carried out by shaking the soil through a stack of sieves with known opening sizes - in order to classify the larger particles and by using a hydrometer and Stoke's equation for the velocity of a free falling sphere to determine the range of fine particles.

### 3.2.3 Particle size analysis results

The following table and graph represent the results of the particle size analysis.

Particle Size (mm)	% Passing (by mass)	Description Type	%
37.50	100	GRAVEL	0
26.50	100		
19.00	100		
13.20	100		
4.75	100		
2.00	100		
0.850	100		
0.425	100	SAND	18
0.250	99		
0.150	98		
0.075	97		
0.060	82	SILT	59
0.046	64		
0.022	52		
0.007	36		
0.002	23	CLAY	23
Total % passing =			100

**Table 3.1: Results from the particle size analysis of the sieve and hydrometer methods**



**Figure 3.4 : Graphical representation of the particle size distribution**

### 3.2.4 Mineralogical analysis of the soil

In addition to determining the physical properties of the soil, the soil was also subjected to x-ray diffraction (XRD) analysis. X-ray diffraction is the analytical tool most commonly used to identify clay types in terms of their mineralogy. [72]

In x-ray diffraction analysis two parameters are determined. These are: [24]

- a) the spacings between the planes of atoms in the crystals (basal spacing), and
- b) the intensities of the x-ray reflections from the corresponding planes.

X-ray diffraction produces diffraction patterns or spectra, which are used to identify minerals. These spectra are graphical representations of peaks, with the intensity of the peaks on the y-axis and the angle of diffraction on the x-axis. The position of these peaks in relation to the x-axis indicates the spacing between successive sheet layers of atoms (known as the basal spacing or 00L spacing) in a crystal structure.

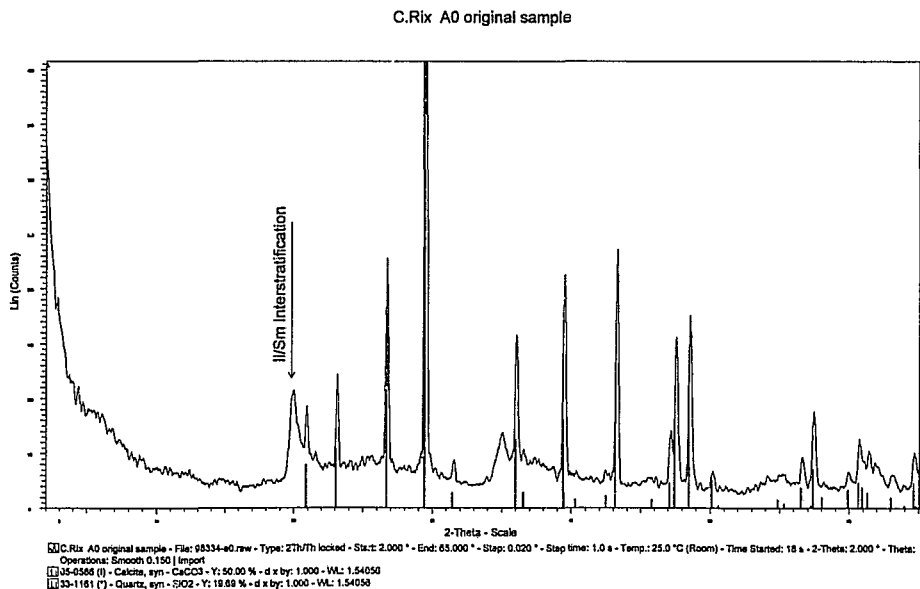
The spacing is used to determine the nature of the crystal structure and thereby the nature of the material undergoing x-ray diffraction analysis. [11] The soil samples were analyzed specifically to determine what the type of clay mineral makes up the clay fraction of the soil. This is important as the type of clay mineral in soil plays an important role in the effectiveness of the different stabilising agents used.

The XRD analysis was performed at the Council for Geoscience by Dr. Dieter Böhman, a renowned scientist in the field of clay mineralogy in South Africa. The results of the XRD are mainly qualitative, where the spectra produced during the analysis are used to identify clay minerals, although the introduction of computer software packages enables semi-quantitative analyses to be conducted.

## Results of the mineralogical analysis

Figure 3.5 shows the spectrum of the soil produced by the XRD analysis. The spectrum shows a peak at approximately  $20^{\circ}2\theta$  which is associated with an illite/smectite clay mineral. The illite/smectite clay mineral is of an irregular stacking arrangement as determined by Böhman. Quartz and calcite are also identified in the spectrum. Calcite is the mineralogical name for calcium carbonate. The semi-quantitative analysis indicates that the soil consists of 20% illite/smectite, 10% quartz and 70% calcite.

The high percentage of calcite is assumed to be as a result of the carbonation of the calcium hydroxide which is used during the flocculation process of the sludge. The amount of clay mineral correlates approximately with the result from the particle size distribution test, which indicated that the soil consisted of 23% clay.



**Figure 3.5: Diffraction spectrum of the unstabilised Panfontein soil.**

### **3.3 Classification of the Panfontein soil**

The physical properties of the soil as determined by the laboratory tests were used to classify the soil according to the Unified Soil Classification System. The procedure used is explained in section 1.6.

From the particle size distribution curve (figure 3.4) and Table 3.1 :

Approximately 97 percent of the soil passes the 0,075mm sieve. Therefore as can be seen on the classification chart (figure 2.6) the soil is fine grained and can be classified as MH, OH, or CH.

Because the fine fraction is greater than 12 percent, the soil is not subject to dual classification and the secondary symbol can be obtained using the plasticity chart. The plasticity index (liquid limit - plastic limit) for the soil is 35 and the liquid limit is 64. If the relevant point is plotted on the plasticity chart (figure 2.4), it is seen that the point falls above the A-line in the CH zone with the C indicating the soil is a clay and the H indicating a liquid limit of higher than 50 percent. The classification for this soil is thus CH, which on the classification chart (figure 2.6) is described as an inorganic clay of high plasticity.

### **3.4 Problems associated with the soil according to its properties**

As discussed in chapter 2, the suitability of a soil as a raw material is dependent on its properties. The particle size distribution of a soil, which is suited for use in the production of soil blocks, is shown in figure 2.5. When comparing the results of the particle size distribution curve of the sludge to the recommended limits, it is evident that the grading does not fall within the limits. The sludge has a narrow grading curve, which means it is lacking in one or more of the components which, make up a soil. In this case it is the sand fraction that is missing.

The plasticity index and the liquid limit of the sludge also fall outside the limits proposed in figure 2.6. This high plasticity index of the soil indicates that the soil

is highly susceptible to volume changes in the presence of moisture. This is obviously undesirable if the soil is to be used for the production of compressed earth blocks, as volume changes can destroy the bonds between soil particles and may lead to eventual disintegration of the block. This was confirmed when the blocks were made of the unstabilised soil.

Although the amount of clay in the soil falls within the recommended limits, the actual nature of the clay mineral is the problem i.e. the smectite in the illite/smectite interstratification. Smectite is a highly expansive clay, whereas illite does not expand much when exposed to moisture. The inclusion of the smectite therefore makes the sludge unsuitable as a construction material. If the soil consisted of 20% kaolinite or 20% illite, the soil would not be problematic.

#### **3.4.1 Modification of problematic properties.**

The objective of soil stabilisation in this case is therefore to alter the problematic properties of the sludge detailed above. The particle size distribution not being ideal should be corrected to fall within the limits recommended. If however it is calculated how much of the sandy fraction is to be added, bearing in mind that the optimum mix would also have to be economic, a large amount of sand would have to be imported as a minimum of 50% of the mass of the dry sludge in the form of sand between 5mm and 0.075mm will have to be added to the sludge. It was decided not to correct the grain size distribution. The ideal solution however would be to correct the texture with the addition of sand.

The plasticity of the soil, if not altered, would have a detrimental affect on the mechanical properties of the compressed block due to volume changes when exposed to moisture. The plasticity of the soil is related to the percentage of and type of clay mineral present in the soil. Therefore by reducing or modifying the clay fraction the plasticity of the soil could be reduced. The plasticity can be modified by treatment with lime resulting in a reduction of the liquid limit and an

increase in the plastic limit can be achieved through the reactions detailed in chapter 1 (lime-clay reactions). As discussed the addition of lime to a clayey soil results in a modification reaction and if sufficient lime is added, cementation of the clay particles will occur. The objective of the addition of the lime was therefore to achieve the modification reaction. This reaction is expected to result in changes in the plasticity index. The lime was added to bring the plasticity index and liquid limit of the soil within the limits recommended for soils for compressed earth blocks that are to be stabilised with cement.

To achieve these changes in the properties of the sludge further tests involving the addition of lime to the sludge were conducted. The tests conducted were undertaken to determine the following:

1. The amount of lime required by the soil in order for modification of the clay fraction to take place :

This is determined using a test known as the initial consumption of lime (ICL) test, which gives an indication of the amount of lime required by a soil to satisfy the soil's affinity for calcium. The result indicated by the test will be the amount of lime required for modification of the clay (see section 2.3) and the quantity of lime indicated by the ICL test is also known as the soil's lime fixation point. This quantity of lime is considered to be the optimum quantity. The testing procedure used for the ICL test is detailed in Appendix 2.

2. The effect that the optimum quantity of lime has on the Atterberg limits of the soil:

This was conducted to determine if the optimum quantity of lime would reduce the plasticity index of the soil sufficiently to bring it within the limits recommended for soils to be stabilised with cement. The time period required for the change in the plasticity index of the sludge was also determined. These two characteristics were determined by adding the optimum quantity of

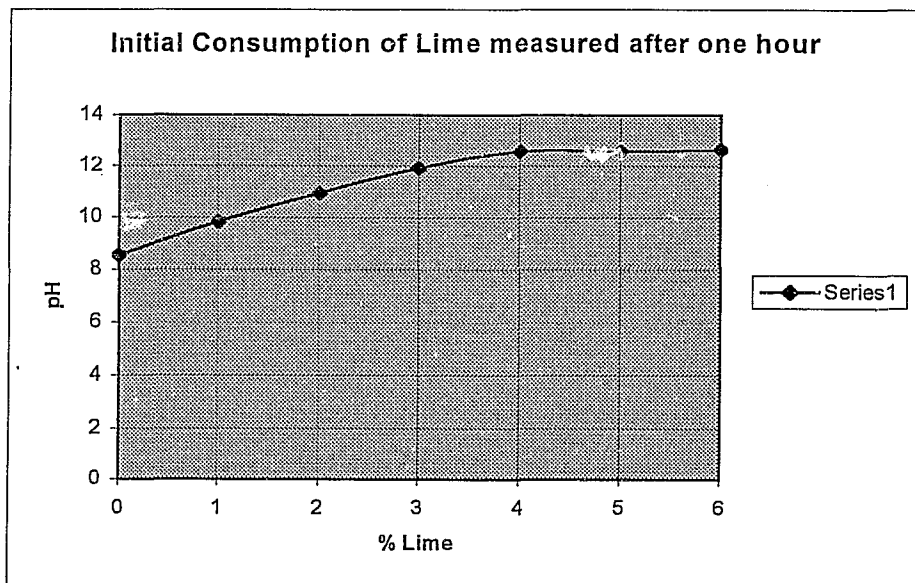
lime to the soil and then allowing the mix to cure. The Atterberg limits of the lime-soil mix were then determined after 2, 6, 24 and 48 hours of curing.

### 3.5 Laboratory test results

#### 3.5.1 Initial Consumption of Lime test results

The initial consumption of lime or 'lime fixing point' of the soil is that percentage of lime at which the pH of the lime-soil mix remains constant. The results of the test conducted are as follows:

Percentage lime	0%	1%	2%	3%	4%	5%	6%
pH	8.54	9.82	10.94	11.9	12.57	12.58	12.65



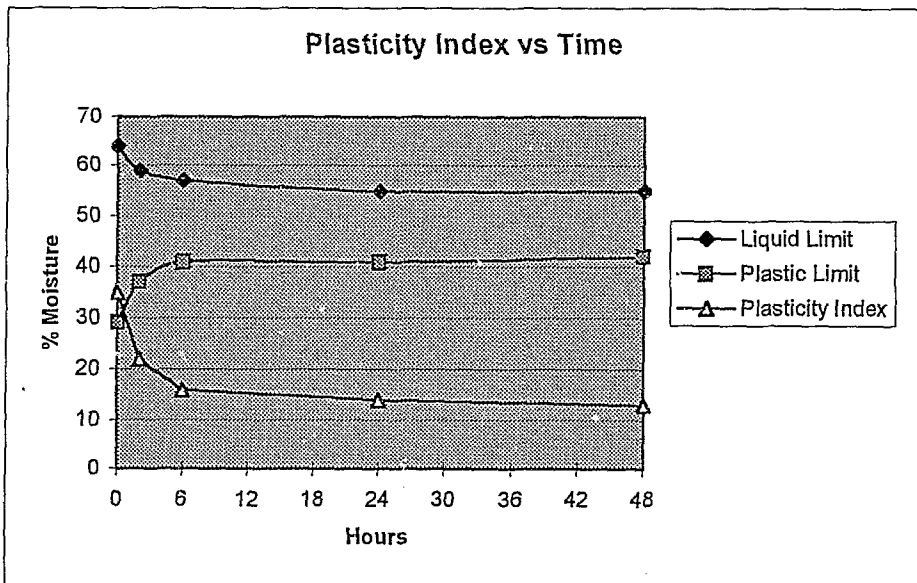
**Figure 3.6: ICL test results**

From the graph, the fixing point of the sludge is 4%. The 4% lime will therefore be the minimum amount of lime which will result in modification of the sludge.

Samples of soil were then treated with 4% of lime and their liquid limit and plastic limit determined after different time intervals.

**3.5.2 Results of the effect of curing period on the Atterberg Limits of the lime modified sludge**

MODIFICATION PERIOD	0hr	2hr	6hr	24hr	48hr
LIQUID LIMIT	64	59	57	55	55
PLASTIC LIMIT	29	38	41	41	42
PLASTICITY INDEX	35	21	16	14	13



**Figure 3.7: Changes in the Atterberg Limits after addition of 4% lime.**

The results show that the addition of 4% lime reduces the Plasticity index of the soil from 35 to 16 in 6 hours after which it only reduces to 13 after an additional 42 hours. The modification of the clay, as indicated by the changes in the

plasticity index and liquid limit, proceeds rapidly within the first 6 hours of curing after which the changes are relatively small.

The table below shows the change in the linear shrinkage with the addition of 4% lime as a function of curing time. The linear shrinkage of the unstabilised sludge is 15.5 %. The linear shrinkage of the soil is also reduced markedly after the addition of 4% lime with the linear shrinkage of the material being reduced from 15.5 to 8.5 in 6 hours and a further reduction to 6.5 in a further 42 hours. Linear shrinkage results:

Modification period	0hr	2hr	6hr	24hr	48hr
Linear Shrinkage	15.5	9.5	8.5	7.5	6.5

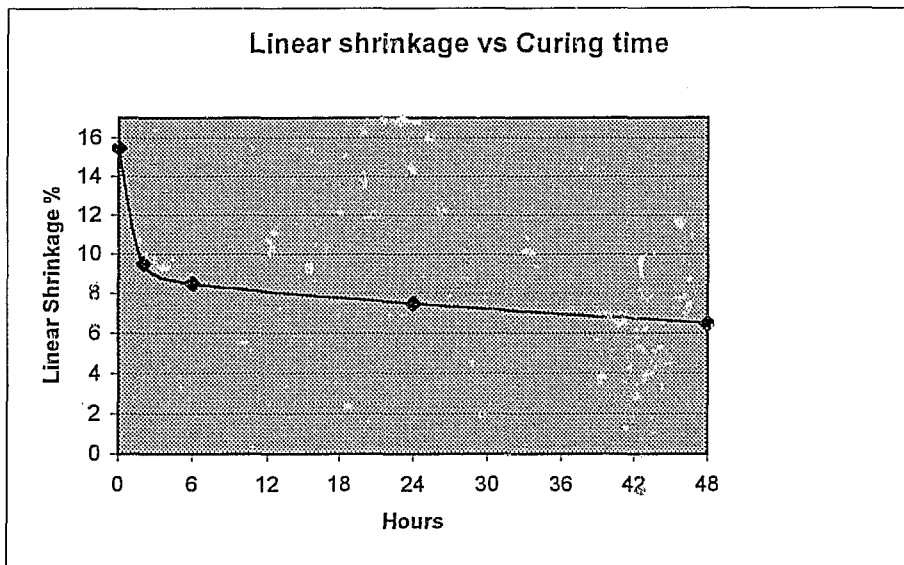


Figure 3.8: Change in Linear shrinkage after addition of 4% lime.

### 3.5.3 Mineralogical changes due to addition of lime.

In order to gain a fuller understanding of the modification of the soil by lime, x-ray diffraction analysis was conducted on the soil treated with 4% lime, in an attempt to correlate the changes in the physical properties with a change in the mineralogy of the soil. As described in chapter 1 the modification reaction results in the exchange of cations. A question often asked in this research was why the modification of the clay does not occur when lime is added during the water purification process. After the lime is added, the pH of the water is corrected by bubbling carbon dioxide through the water. The author believes that this results in the carbonation of the calcium ions and thereby not allowing the ions to take part in the modification reactions. The time period between the addition of the lime and the correction of the pH is not sufficient for the required reactions to take place. The cations which are exchanged are those which are in the interlayer space. Kaolinite therefore does not undergo significant modification as there are no interlayer cations in kaolinite and it has a low cation exchange capacity (1-10 meq/100g). Illite has potassium ions in the interlayer space and these are not readily replaced by the calcium cations, the cation exchange capacity of illite is between 10 and 40 meq/100g. Smectite minerals have a high cation exchange capacity as the bond between the layers in the minerals is weak and the cations are loosely held and therefore easily exchanged. The cation exchange capacity may range from 80 to 150 meq/100g.

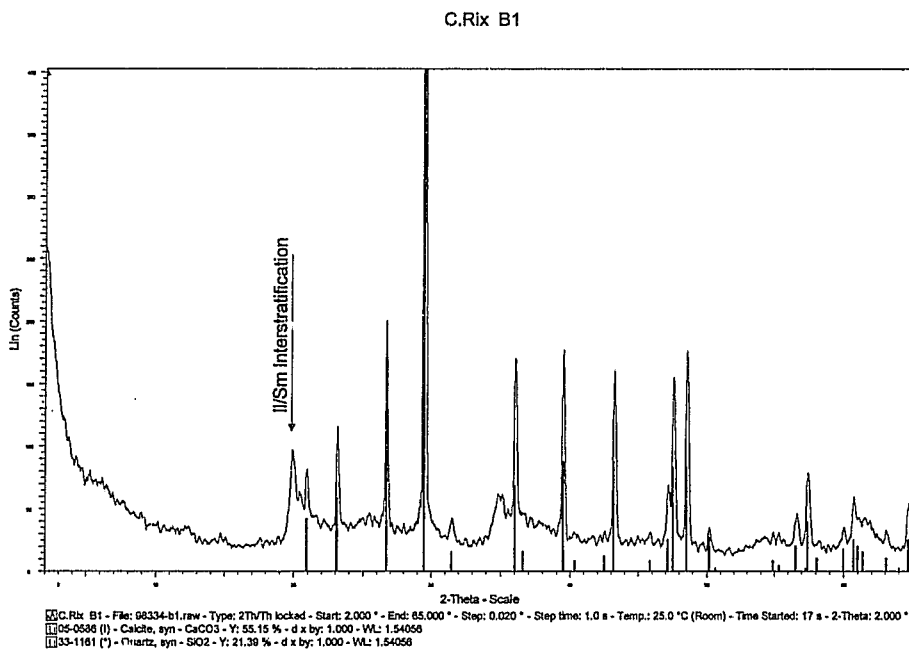
In an illite/smectite clay mineral cation exchange would take place in the interlayer space between smectite layers. Therefore, if the sodium cations are replaced by larger calcium cations the interlayer space which is determined by the cation adsorbed into it would increase. The peak in the XRD diffraction pattern would then shift to the left (basal spacings decrease from left to right on the x-axis). The result hoped for was a decrease in the intensity (height) of illite/smectite peak, indicating that modification had taken place by cation exchange. The results of the XRD analysis were then compared with the XRD results of the unstabilised soil (figure 3.5) with the XRD spectrum of the soil after a 4% addition of lime.

Work done by Croft [10], Hilt and Davidson [37], Eades and Grim [18] included the x-ray diffraction of illite and montmorillonite(smectite) which had been treated with lime. They concluded that the reaction of lime begins with a replacement of existing cations between the silicate sheets with  $\text{Ca}^{++}$ . Following the saturation

of the interlayer positions with  $\text{Ca}^{++}$  the whole clay mineral structure deteriorates with an accompanied reduction in the peak intensity related to the clay mineral.

This reduction in peak height was not evident in this research. However the changes which take place in the physical properties are evidence of distinct changes which take place in the clay.

The XRD was conducted on the lime-stabilised soil after the mix had been allowed to 'cure' for 6 hours. Six hours was chosen as the curing period as the change in the Atterberg limits slowed markedly after 6 hours of curing and it was assumed that the majority of the modification had taken place in this period of time.



**Figure 3.9 XRD spectrum of 4% lime stabilised sludge**

To determine what affect a lime addition higher than indicated by the ICL test would have on the clay a XRD analysis was conducted on a sample stabilised with 7% lime. Figure 3.10 shows the spectrum.

The presence of Portlandite  $\{Ca(OH)_2\}$  indicates that there is excess lime which was not used up in the modification reaction. The semi-quantitative analysis showed the concentration of the Portlandite to be 3%. This reinforces the findings of the ICL test which indicated a lime fixing point of 4% for this soil.

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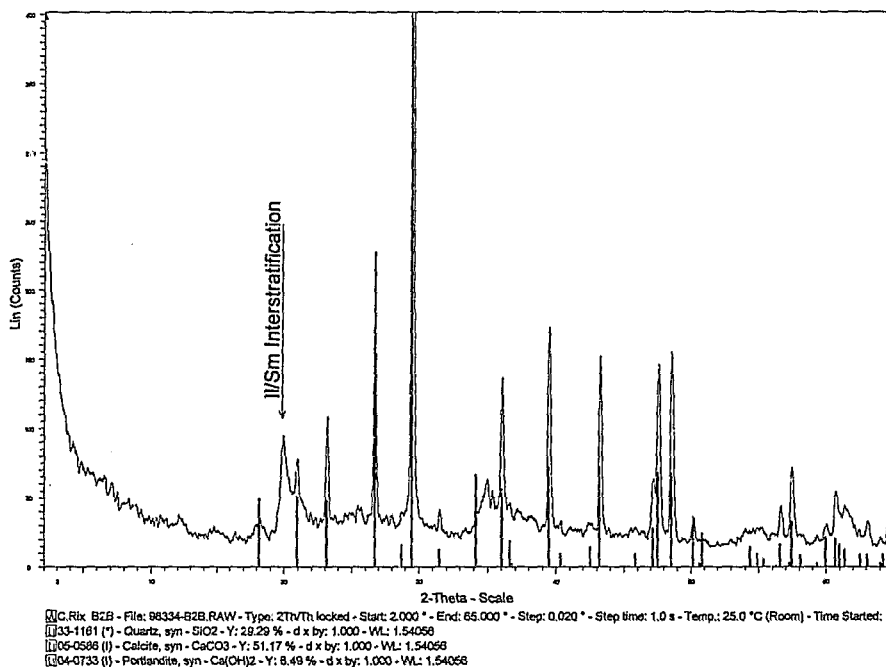


Figure 3.10: Diffraction pattern of Panfontein soil stabilised with 7% lime.

## **Conclusions**

The changes which have occurred in the physical properties are not shown by the XRD spectra. The author believes this is due to the clay mineral having a poor crystalline structure. The change in the physical properties as indicated by the Atterberg limits are therefore assumed a more reliable source of evidence of the modification reactions associated with lime stabilisation in this particular soil.

# CHAPTER FOUR

## 4.0 MATERIALS PREPARATION AND TEST BLOCK MANUFACTURING PROCESS

### 4.1 STABILISING AGENTS

#### 4.1.1 Cement

Ordinary Portland cement (CEM I 42.5) was used throughout the project. Sufficient cement for the entire project was sealed in plastic bags, which were placed together in airtight containers. Cement retains its properties over fairly long periods of time if correctly sealed from the ingress of carbon dioxide and water vapour. [80]

Compound	%
SiO <sub>2</sub>	22.9
Al <sub>2</sub> O <sub>3</sub>	4.2
Fe <sub>2</sub> O <sub>3</sub>	3.8
Mn <sub>2</sub> O <sub>3</sub>	0.4
TiO <sub>2</sub>	-
CaO	65.0
MgO	3.3
P <sub>2</sub> O	-
SO <sub>3</sub>	0.2
Na <sub>2</sub> O	0.1
K <sub>2</sub> O	0.6

Table 4.1: Chemical analysis of Ordinary Portland cement

#### 4.1.2 Lime

A lime commonly used for soil stabilisation was selected for this project. The lime is described as a "super air-separated dry hydrated lime". The chemical analysis of the lime is shown in table 4.1. To prevent carbonation after opening the bag of lime, the lime was divided into smaller quantities and stored with bags of dessicant in airtight containers. In this way, the lime was minimally exposed before use. In considering the type of lime to use, unslaked lime was rejected because of its' caustic nature.

Available CaO	67%
Total Calcium as CaO	73.05%
Total Magnesium as MgO	0.63%
Silica + Insolubles as SiO <sub>2</sub>	2.08%
Other Metals as R <sub>2</sub> O <sub>3</sub>	0.94%
Carbon Dioxide at Works	0.25%
Loss on Ignition	23.23%
Residue on 0.075mm sieve	1.50%

**Table 4.2: Chemical analysis of hydrated lime**

#### 4.1.3 Fly ash

The fly ash used in this research is known commercially as 'pozzfill' available from ash resources. The particle size is sub 110 micron. Fly ash is not reactive with the atmosphere as is the case with lime and cement, and was stored on wooden palettes in the laboratory. The 'pozzfill' was delivered by special arrangement with Ash Resources in 25 kg bags to facilitate storage. When a bag was opened the total contents were usually used and therefore it was felt that

special storage considerations were not required. The chemical analysis of the fly ash produced at the Lethabo and Matla power stations are shown in table 4.2 . Fly ash from the Lethabo power station was used in this research.

		Lethabo Fly Ash
Silicon	- SiO <sub>2</sub>	53.8%
Aluminium	- Al <sub>2</sub> O <sub>3</sub>	34.2%
Iron	- Fe <sub>2</sub> O <sub>3</sub>	3.6%
Manganese	- Mn <sub>2</sub> O <sub>3</sub>	0.1%
Calcium	- CaO	4.4%
Magnesium	- MgO	1.1%
Phosphorous	- P <sub>2</sub> O <sub>5</sub>	0.3%
Potassium	- K <sub>2</sub> O	0.5%
Sodium	- Na <sub>2</sub> O	0.4%
Titanium	- TiO <sub>2</sub>	1.6%
Sulphur	- SO <sub>3</sub>	0.1%
Loss on Ignition	- LOI	0.8%

**Table 4.3: Chemical analysis of fly ash**

## 4.2 STABILISER CONTENTS

### 4.2.1 Stabiliser combinations used for test blocks

The mix design parameters used in the research were chosen to determine what the optimum combinations of the various admixtures are to produce compressed earth blocks with certain physical characteristics. The parameters were also chosen to determine what affects the various admixtures would have on the physical properties of the clay soil.

The mixtures used in the research are indicated in the tables below:

#### Soil-lime combinations

MIX	SOIL	LIME	CEMENT	FLY ASH	OMC
M	100%	0%	0%	0%	24.6%
L1	96%	4%	0%	0%	25.4%
L2	93%	7%	0%	0%	25.6%
L3	91%	9%	0%	0%	25.9%

Table 4.3: Lime content for lime-stabilised blocks

#### Soil-lime-cement combinations

MIX	SOIL	LIME	CEMENT	FLY ASH	OMC
K1	90%	4%	6%	0%	25.1%
K2	89%	4%	7%	0%	24.8%
K3	88%	4%	8%	0%	24.6%
K4	87%	4%	9%	0%	24.3%

Table 4.4: Stabiliser contents for lime-cement stabilised blocks.

**Soil-lime-cement combinations**

MIX	SOIL	LIME	CEMENT	FLY ASH	OMC
N1	87%	7%	6%	0%	25.3%
N2	86%	7%	7%	0%	24.9%
N3	85%	7%	8%	0%	24.8%
N4	84%	7%	9%	0%	24.6%
O1	85%	9%	6%	0%	25.7%
O2	84%	9%	7%	0%	25.5%
O3	83%	9%	8%	0%	25.4%
O4	82%	9%	9%	0%	25.2%

**Table 4.5: Stabiliser contents for lime-cement stabilised blocks.**

**Soil-lime-fly ash-cement combinations**

MIX	SOIL	LIME	CEMENT	FLY ASH	OMC
I1	70%	4%	6%	20%	23.6%
I2	69%	4%	7%	20%	23.4%
I3	68%	4%	8%	20%	23.0%
I4	67%	4%	9%	20%	22.7%
H1	60%	4%	6%	30%	21.3%
H2	59%	4%	7%	30%	21.1%
H3	58%	4%	8%	30%	20.8%
H4	57%	4%	9%	30%	20.6%
J1	50%	4%	6%	40%	19.1%
J2	49%	4%	7%	40%	18.6%
J3	48%	4%	8%	40%	18.4%
J4	47%	4%	9%	40%	18.2%

**Table 4.6 : Stabiliser contents for lime-cement-fly ash blocks**

### 4.3 PRODUCTION OF TEST BLOCKS

The test blocks, which were manufactured for the purpose of this study, were made using a M5 Hydraform block-making machine. The machine consists of a vertically orientated mould, which is filled with the soil-stabiliser mix by hand from above. A hydraulically operated gate closes the top of the mould, before the material is compacted from below using a hydraulic ram. Once the compaction has taken place the top gate is retracted and the compacted soil block is pushed up out of the mould and removed by the operator.

The unit is powered by a 10 kW diesel engine and is mounted on wheels which makes it portable despite its 850kg mass. The nominal dimensions of the blocks manufactured by the Hydraform block-making machine are 220mm wide by 115mm high. The length of the block is dependent on the amount of material put in the mould as well as the compactive effort applied. The stress applied by the ram during compaction of the block can be varied.

#### 4.3.1 Pre-processing of the sludge

When the sludge dries out in the settling ponds it forms clods of various sizes. These clods first had to be broken down into smaller particles to ensure that the stabilisers could be mixed in well. The sludge clods were therefore crushed so that the particles would pass through a sieve with 2mm openings. This resulted in lumps of clay (maximum size 2.0mm) and clay 'powder'. The decision to pulverise the clods to a maximum size of 2.0mm was based on results achieved during preliminary testing where the sludge received had been reduced to sub 2.0mm particles. The sludge samples were those which had been prepared for an investigation of the possible use of sludge in the production of fired-clay bricks. The blocks which were produced using the sub 2.0mm sludge proved to have a good surface finish. This was compared with blocks produced by the author during an undergraduate research report where the sludge was sieved

using a sieve with 5.0mm openings. The blocks produced had a poor surface finish and did not compact well. Based on this observation it was decided to use the sludge with a maximum particle sizes of 2.0mm. After the sludge had been pulverised it was sieved, thoroughly mixed and placed in 25kg bags in the laboratory.

#### 4.3.2 Pre-conditioning the soil with lime

As discussed in chapter 3 the plasticity of the sludge had to be reduced. This was achieved by pre-conditioning the sludge before the addition of cement and fly ash. Before the cement and fly ash components of a mix were added, the sludge was treated with 4% lime (as indicated by the results of the ICL test) to allow the modification of the clay to take place. The lime and soil were weighed out and placed in a pan mixer and dry mixed for 4 minutes. Water was then measured out using a measuring cylinder so that 10% moisture was added to the lime-soil mix. The combination was then mixed for a further 6 minutes. The soil-lime combination was then put into a plastic bag and sealed to prevent carbonation and loss of moisture. The combination was left for a 6-hour period to allow the modification reaction to take place. The test conducted into the effect of the 4% lime addition on the Atterberg limits (section 3.7.2) showed that the majority of the modification reaction takes place in the first 6 hours after mixing. Further curing results in very small changes. From a production point of view, therefore the modified soil can be used after 6 hours.

#### 4.3.3 Determination of the optimum moisture content. (OMC)

The optimum moisture content of a soil or soil stabiliser combination can be defined as the moisture content at a particular compactive effort which will result in the highest density of the compacted material being achieved. The optimum moisture contents for the various combinations used were determined by firstly applying the 'drop' test and then refining this by producing blocks which could be

successfully handled without disintegrating. The drop test is a field test used in the field of stabilised soil blocks to determine the optimum moisture content for a particular combination of soil and stabiliser. Moisture is added to a soil-stabiliser mix until a ball can be formed in the hand. This ball is dropped from a height of one metre onto a hard surface. The soil-stabiliser mix is considered to be near its optimum moisture content when the ball breaks up into 4 or 5 pieces and does not shatter. The moisture content is too high if the ball remains fully intact.

Blocks are then produced using the moisture content indicated by the drop test. These blocks were then examined to determine if the block could be handled without the corners or any edges crumbling. The blocks were all compacted at a compactive effort of 4 MPa as the blocks compacted at lower pressures crumbled when handled and blocks produced at higher pressures showed pressure cracks as a result of over compaction. These cracks formed as soon as the block was removed from the mould. It was also desirable to keep the compactive effort the same for all the combinations so compactive effort could be eliminated as a variable when blocks produced using the various combinations were compared. Once the author was satisfied with the degree of compaction achieved, a sample of the soil was oven-dried at 110°C to determine the exact moisture content. The moisture contents (OMC) used in the production of the blocks are shown in table 4.3-4.6 where details of the combinations used are given.

#### 4.3.4 The manufacturing of the blocks

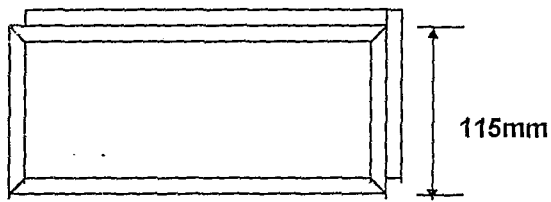
After the 6-hour modification period, the lime-soil combination was returned to the pan mixer where the measured amounts of cement or the cement and fly ash were added. These combinations were then mixed for approximately five minutes until a mix with uniform colour was obtained. The balance of the water required to bring the mix to its optimum moisture content was then added by sprinkling and mixing for a further five minutes.

After the mixing the pan mixer was stopped and the soil-stabiliser combination was poured into the compaction cylinder of the Hydraform machine. The mix was then compressed and the block removed. This process was followed for all the design mixes. After the blocks were produced, they were cured under plastic for a period of fourteen days after which they were removed from the plastic, dry cured for a further 14 days, and then removed to be tested.

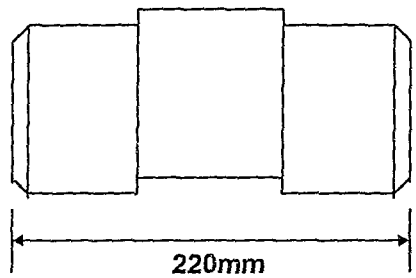
For all the blocks manufactured for this research an applied stress of four MPa was used.

A total of thirty test specimens were produced for each mix. A total of 840 specimens were produced for the research.

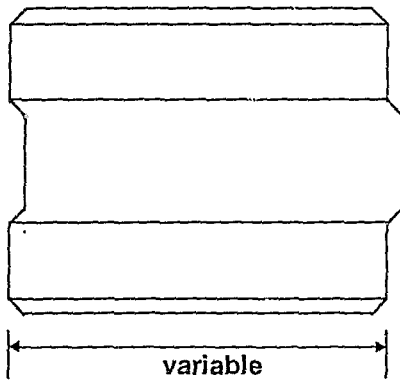
Figure 4.1 shows the dimensions of a typical block produced in the Hydraform block press. The key, which is provided to facilitate dry stacking of the blocks, can also be seen.



**SIDE ELEVATION**



**FRONT ELEVATION**



**TOP ELEVATION**

**Figure 4.1 : The Hydraform Block in elevation**

# CHAPTER FIVE

## 5.0 Tests conducted on the blocks

### 5.1 Introduction

Chapter five covers the physical tests conducted on the blocks produced from Panfontein soil and the three stabilising agents. XRD analysis was also undertaken on samples taken from the test blocks in an attempt to evaluate the changes in mineralogy as a result of the addition of fly ash and cement. This was undertaken to correlate the changes in the performance of soil with changes in mineralogy. The XRD analysis is discussed at the end of the chapter.

The physical tests conducted on the stabilised soil samples were chosen to evaluate the effect various combinations of the three stabilising agents have on the performance of the sludge when used as a raw material for the production of building blocks. The tests chosen are those which were expected to be the most likely indicators of the stabilised soil blocks' performance as a material under field conditions. The results of the tests are used to obtain an indication of the effectiveness of a particular combination when compared with other combinations and in some cases, generally accepted standards used in the field of stabilised soil blocks. Another objective of the tests was to determine an optimum combination of the stabilizers, which will satisfy the standards set, as well as being economical in terms of material costs.

The blocks were tested to determine their physical properties with respect to the following:

1. Dry compressive strength
2. Wet compressive strength
3. Drying shrinkage
4. Water absorption
5. Resistance to water induced erosion.
6. Expansion due to water absorption.

### 5.1.1 Standards adopted for assessment of blocks

As the field of compressed earth blocks is still a relatively new one in South Africa, there are no standards by which the performance of the blocks can be judged. It was therefore decided to use a standard developed by CRATerre and the Center for Development of Industry in Europe known as CIB W 90. The standards were developed as a framework for individual countries to adopt and customise to local conditions. The standard suggests compressive strength and water absorption limits for building blocks to be used in certain conditions. CRATerre also suggests a limit for durability in terms of an accelerated water erosion test. The standard has various requirements in terms of mechanical, hygrometric and physical characteristics for ordinary blocks (intended to be covered) and facing blocks (intended to remain visible). The standard used was that applicable to ordinary blocks. The ordinary blocks are divided into 9 classes each with different requirements. The criteria adopted was for a block which would be used in a *structural element resistant to small external live loads* e.g. a single-storey building made of load bearing structural elements. The environmental category which the block falls into is *structural elements resistant to water damage by vertical penetration and lateral spraying* e.g. walls exposed to rain, external walls unprotected from capillary rise and internal walls unprotected from water leaking through the roof. The block falls into the category designated by the following number CEB OC2. Blocks in this category must have the following physical properties:

Dry compressive strength	>4 MPa
Wet compressive strength	> 2 MPa
Water absorption	< 10%

The durability of the block is determined by the accelerated water erosion test. The recommendation made by CRATerre as to the blocks' performance with

regard to this test are that depth of the eroded pockets, which develop during the accelerated erosion test, must not exceed 15mm in depth.[39]

The following table details the number of specimens tested for each of the mixes produced:

<u>TEST</u>	<u>NO. OF SPECIMENS PER MIX</u>
Compressive strength – dry	28 days - 5
- wet	7 days - 5 * 28 days - 5
Drying shrinkage	3
Water absorption	3
Water induced erosion	3
Expansion due to water absorption	3
<b>TOTAL</b>	<b>27</b>

\* The 7 day compressive strength is designated "wet" as the block had only lost 15% of the manufacturing moisture and the blocks were still moist after 7 days of curing under plastic.

## **5.2 UNCONFINED COMPRESSIVE STRENGTH**

The objective of determining the unconfined compressive strength is to establish the effect the various degrees of stabilisation have on the resistance of the blocks to compressive loads. The stress at failure after 28 days is an important indicator of a masonry unit's structural usefulness.

The blocks were tested at 7 and 28 days after being produced. This was done so that an indication of the rate of strength build-up can be determined.

### **Scope**

This method describes the procedure for determining the unconfined compressive strength (wet and dry) of Hydraform stabilised soil blocks.

### **Apparatus**

- (i) A suitable location for curing the blocks. The area should consist of a hard clean surface, which is not exposed to direct light or dramatic temperature fluctuations.
- (ii) A cube crushing machine with platens capable of accommodating the longest side of the block (generally 200mm)
- (ii) A measuring rule graduated in millimeters, at least 300 millimeters long.
- (iii) Two metal plates measuring 15mm thick, 50mm wide and the same length as the block.

### **Method**

The specimens are standard Hydraform blocks manufactured using the designed mixes and at the optimum moisture contents required to produce suitable blocks.

### **Curing of specimens**

The blocks are cured under plastic for the first fourteen days to allow the hydration of the cement to take place in a moist environment (this is the minimum period suggested in literature consulted). After the fourteen days the blocks were removed from the plastic and allowed to "cure" for a further 14 days in the laboratory. The temperature in the laboratory fluctuated between 21°C and 26°C and the relative humidity between 30% and 45%. These conditions were monitored using a thermo-hygrograph.

### **Crushing the blocks**

Five blocks from each design mix are removed from the curing area at seven and twenty-eight days and brought to the crushing machine. The dimensions of the shoulders of the blocks are measured and recorded in millimeters. The blocks are then placed on the platens in the machine in the way that they are laid in a wall and orientated so that they are in the center of the platens. A metal plate measuring 50mm wide by 12 mm thick is then placed on each shoulder of the block (see figure 5.1). The plate is placed on the shoulder of the block to simulate the way the block will be loaded in a wall, as there is no contact between the intrusion and protrusion of blocks laid in a wall. The crushing rate is set at 150 kN/minute. The force reading on the cube crusher should be zeroed and the block crushed in accordance with the operating instructions. The force at which the block fails is recorded. The block is removed from the machine and the next block crushed in the same manner.

## **Determination of the maximum applied stress**

### **Calculations**

Find the effective area resisting the compressive force as follows:

$$A = (L) \times 2(W)$$

where

A = the effective area in mm squared

L = the length of the block in mm

W = the width of the shoulder of the block in mm

Determine the stress at failure using the following formula :

$$\sigma = F / A$$

where:

$\sigma$  = the ultimate compressive stress in MPa

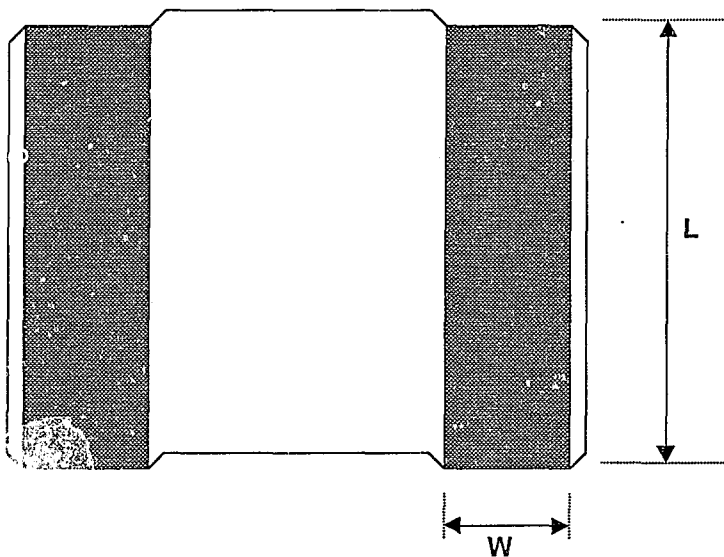
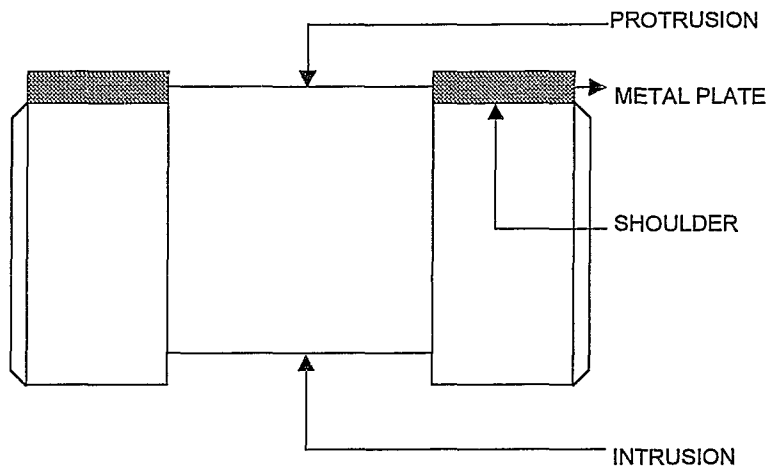
F = the ultimate compressive load in kN

A = the effective area in mm squared

These results are reported graphically against stabilizer content.

## **WET COMPRESSIVE STRENGTH**

For the wet compressive strength, the blocks are cured for 14 days under plastic and then cured for a further 13 days in the laboratory and then immersed in water for a period of 24 hours before being subjected to the test.



TOP ELEVATION

FIGURE 5.1

### 5.2.1 Results of the unconfined compressive strength tests

For mix design parameters see section 4.2

#### Mix L

The 28 day wet compressive strength could not be tested for this design mix as the blocks disintegrated to various degrees when placed in water.

MIX	7 day UCS(MPa)	28 day wet UCS (MPa)	28 day dry UCS(MPa)
M	0.5		3.5
L1	1.0		3.0
L2	1.0		3.0
L3	1.0		2.5

Table 5.1: Unconfined compressive strength for unstabilised and lime – stabilised blocks.

#### Mix K

MIX	7 day UCS (MPa)	28 day wet UCS(MPa)	28 day dry UCS(MPa)
K1	2.5	2.5	5.5
K2	3.0	2.5	6.0
K3	3.0	2.5	6.0
K4	3.0	3.0	6.5

Table 5.2: Unconfined compressive strength (4% lime, variable cement)

#### Mix N

MIX	7 day UCS (MPa)	28 day wet UCS(MPa)	28 day dry UCS(MPa)
N1	2.5	2.5	6.0
N2	2.5	2.5	6.0
N3	3.0	2.5	6.5
N4	3.0	3.0	6.5

Table 5.3: Unconfined compressive strength (7% lime, variable cement)

**Mix O**

MIX	7 day UCS (MPa)	28 day wet UCS(MPa)	28 day dry UCS(MPa)
O1	2.5	2.5	5.5
O2	3.0	2.5	6.0
O3	3.0	2.5	6.0
O4	3.0	3.0	6.5

**Table 5.4: Unconfined compressive strength (9% lime, variable cement)**

**Mix I**

MIX	7 day UCS (MPa)	28 day wet UCS(MPa)	28 day dry UCS(MPa)
I1	2.5	3.0	5.0
I2	3.0	3.5	5.0
I3	3.5	3.5	5.5
I4	4.0	4.0	6.0

**Table 5.5: Unconfined compressive strength (4% lime, 20% fly ash, variable cement)**

**Mix H**

MIX	7 day UCS (MPa)	28 day wet UCS(MPa)	28 day dry UCS(MPa)
H1	2.5	3.0	5.0
H2	3.0	3.0	5.0
H3	3.0	3.5	6.0
H4	3.0	3.5	6.5

**Table 5.6: Unconfined compressive strength (4% lime, 30% fly ash, variable cement)**

### Mix J

MIX	7 day UCS (MPa)	28 day wet UCS(MPa)	28 day dry UCS(MPa)
J1	3.0	3.0	5.0
J2	3.0	3.5	5.5
J3	3.5	4.0	6.5
J4	4.0	5.0	6.5

**Table 5.7: Unconfined compressive strength (4% lime, 40% fly ash, variable cement**

### 5.2.2 Discussion of the UCS test results

When considering the results of the unconfined compressive strength it is important to compare the strength of the blocks in a dry state and a wet state.

#### **Soil-lime**

The addition of lime to the sludge reduces the compressive strength of the sludge in the short-term i.e. 28 days, which is evident when the lime-stabilised blocks are compared with blocks produced from the unstabilised sludge. This trend may be as a result of the increasing flocculation of the clay which results in an increased porosity of the material and therefore a reduction in strength. Lime stabilisation was not considered as a viable option due to its characteristic slow strength build up. Lime was added to modify the clay and not for the purposes of actually increasing the soil block's strength.

It is generally accepted that soils stabilised with lime are tested at 3 months and not 28 days as with cement. The lime-stabilised blocks however were tested at 28 days to be able to make comparisons with other combinations, which included lime. The UCS results for the lime-stabilised blocks indicate that the optimum quantity of lime in terms of dry compressive strength is 7%. This is suggested by

the fact that the strength increases up to a point i.e. 7% lime addition after which, increases in lime reduce the strength. The addition of 7% lime results in a compressive strength of 3.0 MPa being achieved whereas with 9% lime, only 2.5 MPa is attained.

The wet compressive strength could not be determined as the test blocks disintegrated when placed in the water. This indicates that sufficient cementitious bonds had not been created to increase the soils' wet strength. These bonds however might increase if a longer curing period was allowed as the reactions related to strength increase in lime stabilised soil only take place in the long term.

### **Soil-lime-cement**

A similar trend to that of the soil-lime combinations is apparent in the soil-lime-cement combinations, with the exception of increased strength. The results show that for a given amount of cement there exists an optimum amount of lime. An increase in the amount of lime resulting in reduced strength. It is possible that some of the added lime or the lime produced during hydration of the cement is not used up in ion exchange and flocculation reactions and prevents the cementing together of soil properties.

The results of these combinations show that the strength is highest with a 7% addition of lime. An important indicator of the effect of the cement addition when compared to the lime-stabilised blocks, is the fact that the lime stabilised blocks disintegrate when placed in water and the blocks which included cement did not. The wet strength of the lime-cement stabilised blocks is therefore evidence of an improvement in mechanical performance when compared to lime stabilised blocks. The increase in strength that the addition of cement to the soil-lime combination brings about can clearly be seen when comparing mix L1 to K1, L2 to N1 and L3 to O1. The increase in dry strength is 50%, 49% and 52%

respectively. The increase in cement addition results in an increase in the wet and dry compressive strength.

### **Soil-lime-cement-fly ash**

The addition of fly ash to the soil-lime-cement combinations enabled comparisons with combinations of soil-lime-cement to be made to determine what affect the addition of fly ash has on the strength. The results show that an increase in the percentage fly ash leads to a marginal increase in the compressive strength. The increase in strength when comparing mixes with the same percentage of cement is most likely as a result of a pozzolanic reaction between the lime produced during hydration of the cement, and the fly ash.

If a comparison is made between the K series, which is identical to the I, H and J series except for the fly ash component, the dry compressive strength of the K series is higher. This is probably as a result of a higher sludge/clay content, where the clay contributes to the strength in the dry state. However, if the wet compressive strengths are compared, the effect of the sludge/clay is evident, and the strength is reduced considerably. This is because clay has very low strength in a wet state. The reduction in strength of the blocks when comparing dry and wet strength gives an indication of the role clay plays in the dry compressive strength of the various combinations investigated. The reduction in strength for the K series from a dry state to a wet state is 57% on average, whereas the reduction for the I, H and J series is 40%, 37% and 34% respectively.

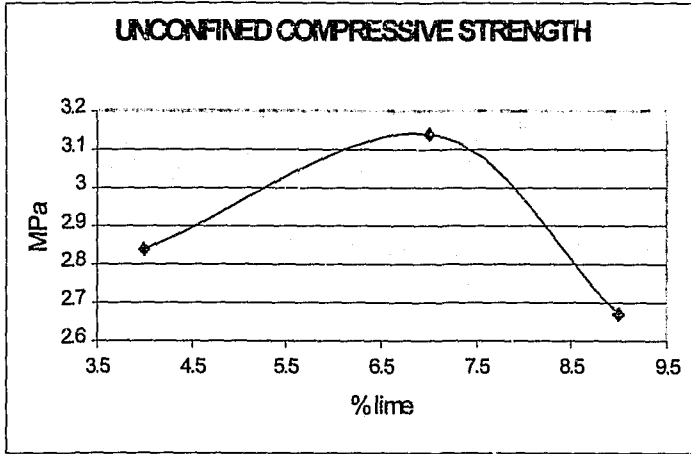


Figure 5.2: UCS results for lime-stabilised blocks

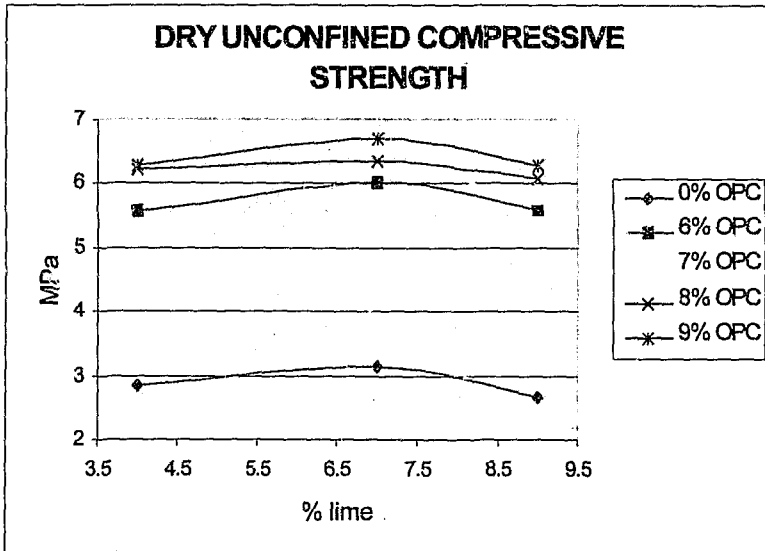
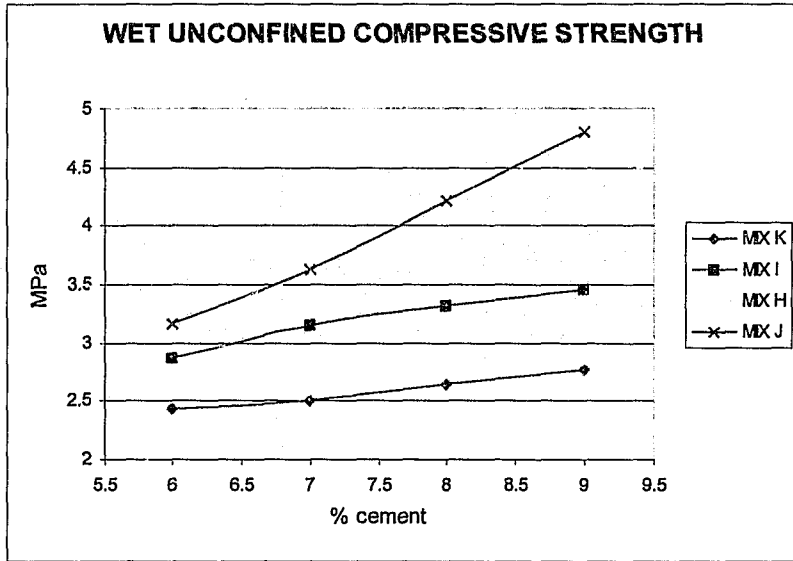


Figure 5.3: UCS results lime-cement stabilised blocks.



**Figure 5.4: Wet UCS of the lime-cement-fly ash stabilised blocks.**

### **5.3 THE DRYING SHRINKAGE TEST**

#### **Objective**

The objective of this test is to determine what effect the various stabilizers have on the drying shrinkage of the block over a period of twenty-eight days.

#### **Scope**

This method covers the determination of the average drying shrinkage of the block at a constant temperature as it dries or cures.

#### **Apparatus**

- (i) A suitable location for curing blocks.
- (ii) Two targets per block, consisting of 2mm thick aluminum rectangular plates (15mm x 35mm).
- (iii) An epoxy resin based adhesive capable of adhering aluminum to block surfaces, mixing cups and mixing rods.
- (iv) A digital vernier caliper.

#### **Method**

##### **Preparation of the specimens**

The specimens are standard Hydraform blocks manufactured using the designed mixes and at the moisture contents required to produce suitable blocks.

##### **Curing of specimens**

The blocks are cured under plastic for the first fourteen days to allow the hydration of the cement to take place in a moist environment (this is the minimum

period suggested in literature consulted). After the fourteen days the blocks were removed from the plastic and allowed to "cure" for a further 14 days in the laboratory. The temperature in the laboratory fluctuated between 21°C and 26°C and the relative humidity between 30% and 45%. These conditions were monitored using a thermo-hygrograph which records temperature and relative humidity.

### **Adhering the targets and using the Vernier caliper**

Two blocks of each design mix are taken one hour after manufacture to be prepared for the test. The block is placed on a hard clean surface in the curing location. A fluid mix of the epoxy-based adhesive is applied in two areas slightly larger than the targets dimensions, centered on the long axis of the block on either end of the block. While these priming applications begin to set another more viscous epoxy mix is prepared. Just before the first applications harden, the plastic slide is used to scrape the epoxy that has not been absorbed by the block, from the block surface. The epoxy adhesive is then applied to the targets which are attached to the block. Once the epoxy-based adhesive has hardened preparations can be made for taking the initial readings. The vernier caliper is then used to take the zero reading across the targets attached at the ends of the block. Readings are taken at 24 hours intervals for the first seven curing days and then at seven day intervals for the remaining 21 days.

### **Determination of linear shrinkage**

The linear shrinkage for a certain period of time is determined by subtracting the reading on the vernier caliper from the zero reading and then dividing this value by the zero reading taken at the beginning of the test period.

### Calculations

The shrinkage for each block at each time interval is calculated as follows:

$$S = [(R_0 - R_T) / R_0] \times 100$$

Where:

S = % shrinkage

R<sub>0</sub> = zero reading (initial test reading)

T = test reading at time interval T

Calculate the average shrinkage for each type of block using the formula:

$$S_{ave} = \frac{\sum S}{n}$$

S<sub>ave</sub> = average shrinkage for each type of block at time interval

∑S = the sum of shrinkage readings for that type of block

n = the number of blocks of that type tested

The results are recorded graphically against time.

### **5.3.1 DRYING SHRINKAGE TEST RESULTS**

The linear shrinkage of the specimens is determined over the 28-day curing period. The results are reported as a percentage of the original length of the block.

<b>Drying Shrinkage (%)</b>				
<b>Mix</b>	<b>7 DAYS</b>	<b>14 DAYS</b>	<b>21 DAYS</b>	<b>28 DAYS</b>
M	0.67	4.45	6.14	7.37
L1	0.2	1.29	3.12	3.26
L2	0.1	1.12	2.32	2.45
L3	0.07	0.97	2.05	2.38

**Table 5.8: Drying shrinkage (mix M & L)**

<b>Drying Shrinkage %</b>				
<b>Mix</b>	<b>7 DAYS</b>	<b>14 DAYS</b>	<b>21 DAYS</b>	<b>28 DAYS</b>
K1	0.15	0.76	2.49	2.78
K2	0.09	0.63	2.28	2.36
K3	0.06	0.48	1.98	2.28
K4	0.06	0.29	1.96	2.13

**Table 5.9: Drying shrinkage (mix K)**

<b>Drying Shrinkage %</b>				
<b>Mix</b>	<b>7 DAYS</b>	<b>14 DAYS</b>	<b>21 DAYS</b>	<b>28 DAYS</b>
N1	0.12	0.68	2.25	2.36
N2	0.08	0.48	2.19	2.22
N3	0.06	0.38	2.04	2.11
N4	0.05	0.32	1.79	1.85

**Table 5.10: Drying shrinkage (mix N)**

Drying Shrinkage %				
Mix	7 DAYS	14 DAYS	21 DAYS	28 DAYS
O1	0.1	0.53	2.19	2.28
O2	0.06	0.39	2.1	2.16
O3	0.04	0.26	1.65	1.79
O4	0.03	0.2	1.63	1.70

**Table 5.11: Drying shrinkage mix O**

Drying Shrinkage %				
Mix	7 DAYS	14 DAYS	21 DAYS	28 DAYS
I1	0.12	0.42	1.03	1.09
I2	0.1	0.4	0.97	1.02
I3	0.08	0.3	0.83	0.88
I4	0.07	0.25	0.67	0.71

**Table 5.12: Drying shrinkage mix I**

Drying Shrinkage %				
Mix	7 DAYS	14 DAYS	21 DAYS	28 DAYS
H1	0.08	0.32	0.39	0.43
H2	0.05	0.21	0.38	0.42
H3	0.05	0.17	0.36	0.4
H4	0.04	0.12	0.31	0.38

**Table 5.13: Drying shrinkage mix H**

Mix	Drying Shrinkage %			
	7 DAYS	14 DAYS	21 DAYS	28 DAYS
J1	0.06	0.1	0.23	0.29
J2	0.04	0.09	0.2	0.26
J3	0.04	0.06	0.2	0.25
J4	0.03	0.07	0.19	0.23

**Table 5.14: Drying shrinkage mix J**

### **5.3.2 Discussion of the drying shrinkage test results**

#### **Soil-lime**

The major source of shrinkage in soils (and concrete [56]) is drying shrinkage where water from the voids evaporates into the atmosphere. The reduction in shrinkage with the addition of increasing amounts of lime is graphically represented in figure 4.5. A substantial change is achieved with a 55% reduction in shrinkage with the addition of 4% lime. 7% Lime yields a reduction of 67% and 9% lime reduces the shrinkage of the sludge by 68%. The reduction in shrinkage is attributed to the ion exchange reactions, which take place between the clay and the calcium ions in the lime, where the clay's affinity for water is reduced.

#### **Soil-lime-cement**

The addition of cement to the soil-lime combination results in a further reduction in shrinkage. This is as a result of the formation of a cementitious matrix in the soil which resists volume changes. [64]

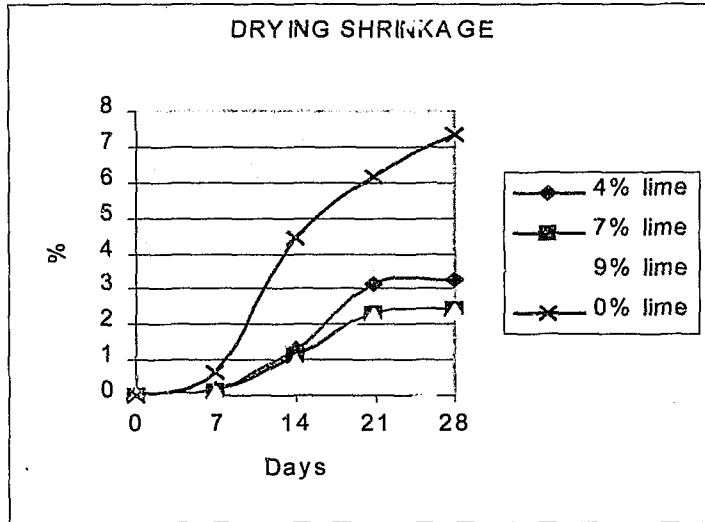


Figure 5.5: Shrinkage reduction achieved by lime stabilisation

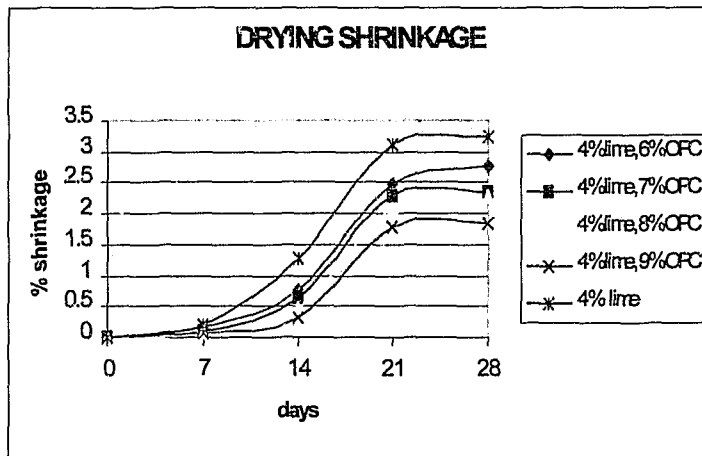
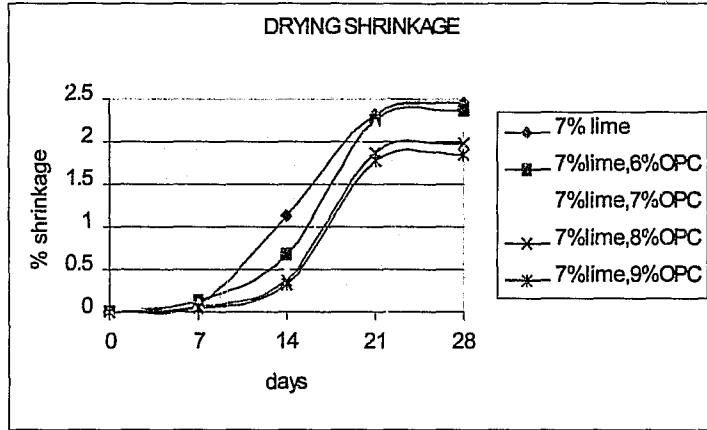


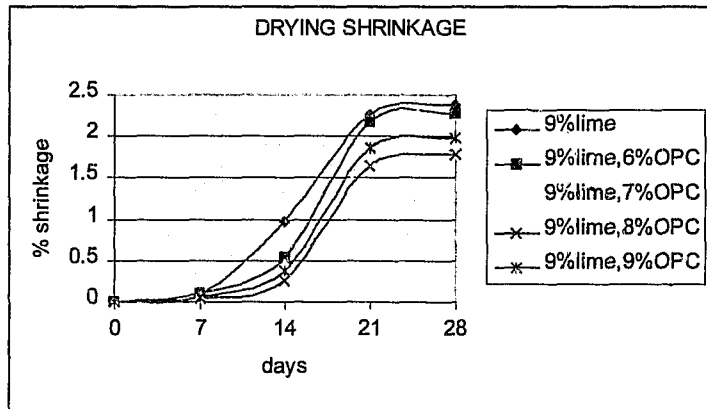
Figure 5.6: Reduction in shrinkage achieved by lime-cement stabilisation

The addition of 6% cement further reduces the amount of shrinkage achieved by the addition of 4% lime from 3.26% to 2.78%. The reduction in shrinkage achieved by an addition of 6% OPC to the 4% lime stabilised sludge is 15%, further additions of cement yield only small changes to the amount of reduction. This is probably as a result of more bonds forming increasing the strength of the cementitious matrix.

The reduction in shrinkage achieved with the addition of cement to the 7 and 9% lime-stabilised combinations are shown in figure 5.7 & 5.8.



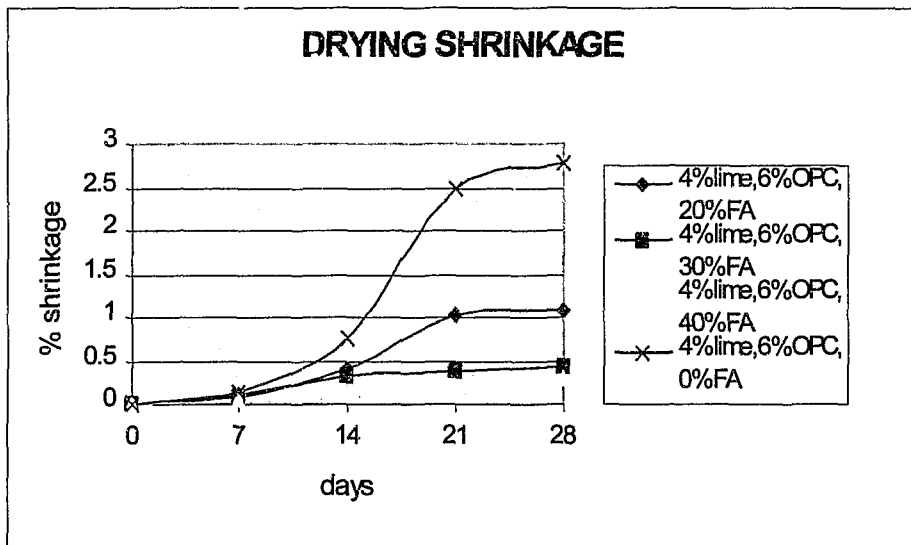
**Figure 5.7: Drying shrinkage of lime and lime-cement stabilised blocks (7% lime, variable cement)**



**Figure 5.8: Drying shrinkage of lime and lime-cement stabilised blocks (9% lime, variable cement)**

## Soil-lime-cement-fly ash

Figure 5.9 shows the affect that fly ash has on the reduction of shrinkage when compared to the similar combinations without fly ash. The addition of fly ash to the soil-lime-cement combinations further reduces the shrinkage. This affect can be attributed to two factors. The first is by adding fly ash, the total amount of the clay fraction in the mix is reduced, thereby reducing the combinations potential to shrink. The second is that the number of cementitious bonds may also be increased by the pozzolanic reaction between lime and the fly ash.



**Figure 5.9: Drying shrinkage of lime-cement and lime-cement-fly ash stabilised blocks ( constant lime and cement contents with variable fly ash)**

When comparing the K1 combination to the I1 combination the shrinkage reduces from 2.78% to 1.09% with the addition of 20% fly ash. If the fly ash content in the combination is increased to 30%(H1) the shrinkage is reduced from 1.09 percent to 0.43 percent, a 40% fly ash content (J1), reduces the shrinkage from 0.43 percent to 0.29 percent.

If the fly ash content in a combination is kept constant and the cement content increased, the shrinkage is also reduced. The increase in cement however has a smaller influence on the reduction in shrinkage than an increase in the fly ash content.

Although drying shrinkage is not generally used to measure the success of stabilisation. The appearance of shrinkage cracks associated with the loss of moisture will often be an important indicator of whether a soil has been stabilised sufficiently. Cracks often lead to a reduction in compressive strength and durability of stabilised soil blocks.

The aim therefore of this test was to investigate to what extent the addition of the various stabilisers affected the drying shrinkage. A general observation made by the author was that above approximately 2-3% shrinkage the shrinkage cracks seem to be more abundant. Based on this observation the author believes that for this particular material ideally drying shrinkage should be kept below 3%.

## **5.4 THE WATER ABSORPTION TEST**

### **Scope**

This method describes the procedure for determining the rate at which a stabilised soil block absorbs water under a constant head.

### **Apparatus**

- (i) A suitable location for curing the blocks. The area should consist of a hard clean surface, which is not exposed to direct light or dramatic temperature fluctuations.
- (ii) A suitable bath of water with sufficient capacity so that all specimens can be fully immersed simultaneously.
- (iii) A balance to weigh up to 20kg and accurate to 0,5g.
- (iv) An absorbent cloth.

### **Method**

#### **Preparation of specimens**

The specimens are standard Hydraform blocks manufactured using the designed mixes and at the moisture contents required to produce suitable blocks.

#### **Curing of specimens**

Cure the specimens for fourteen days under plastic in a suitable location. The blocks are then removed from the plastic and allowed to cure for a further fourteen days in the laboratory.

### **Method**

After the 28-day curing period the mass of the block is determined and recorded (initial mass). The block is then immersed in water for 24 hours. After the 24 hour period the block is removed from the water, the surface dried with an absorbent cloth. The mass of the 'wet' block is then determined and recorded.

### Determining the absorption

After 24 hours the amount of water absorbed can be expressed as a percentage of the initial mass of the block.

### Calculations

The amount of water absorbed  $W$  is calculated as follows:

$$W = \left( \frac{M - N}{M} \right) \times 100$$

Where:

$W$  = the mass of water absorbed during the time interval, expressed as a percentage of the initial mass of the block.

$N$  = the initial mass of the block in grams

$M$  = the mass of the block after 24 hours immersion in grams

### Notes

1. Blocks with similar initial dimensions should be chosen for this test.
2. Blocks, which have obvious surface cracks, should be avoided.

#### 5.4.1 Results of the water absorption test

Results for the L and M combinations could not be obtained as the blocks disintegrated when placed in water.

<u>Mix</u>	<u>ABSORPTIVITY (%)</u>
K1	9.9
K2	9.7
K3	9.5
K4	9.2

Table 5.15: Water absorption of mix K

<u>Mix</u>	<u>ABSORPTIVITY (%)</u>
N1	8.8
N2	8.6
N3	8.5
N4	8.2

Table 5.16: Water absorption of mix N

<u>Mix</u>	<u>ABSORPTIVITY (%)</u>
O1	8.0
O2	7.8
O3	7.5
O4	7.4

Table 5.17: Water absorption of mix O

<u>Mix</u>	<u>ABSORPTIVITY (%)</u>
I1	7.2
I2	6.9
I3	6.5
I4	6.2

**Table 5.18: Water absorption of mix I**

<u>Mix</u>	<u>ABSORPTIVITY (%)</u>
H1	6.1
H2	5.6
H3	5.3
H4	5.0

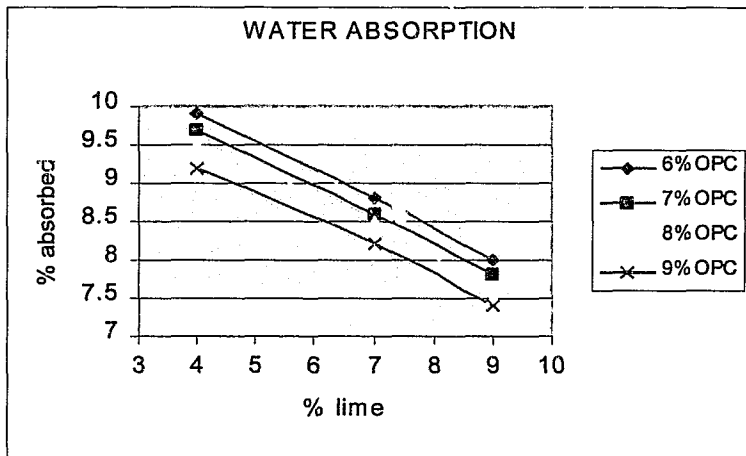
**Table 5.19: Water absorption of mix H**

<u>Mix</u>	<u>ABSORPTIVITY (%)</u>
J1	4.8
J2	4.4
J3	4.0
J4	3.7

**Table 5.20: Water absorption of mix J**

### 5.4.2 Discussion of the water absorption test results

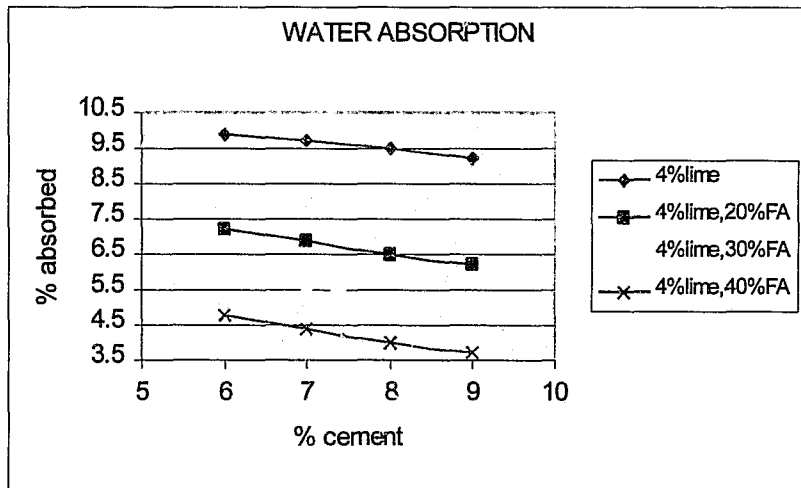
The lime-stabilised blocks could not be evaluated as they disintegrated when placed in water. Comparisons however have been made between the blocks which were stabilised with lime-cement combinations and those stabilised with lime-cement-fly ash combinations. The results of the test are indicated graphically in figure 5.10 – 5.12.



**Figure 5.10: Water absorption results for lime-cement stabilisation.**

The increase in the amount of cement in a combination reduces the amount of water absorbed by the test blocks, this may be as a result of the hydration products filling up the voids in the soil thereby reducing its porosity. The increase in the cement content does not have as great affect on the water absorption properties of the combinations as an increase in the lime content. The affect of the lime is a reduction of the clays' affinity for water, the reason for the increase in lime being more significant than an equal increase in cement is that the lime is the agent responsible for the reduction in water affinity and the amount of lime produced during hydration of the cement is equal to approximately 25% [10] of

the amount of cement added and so there is less lime available to reduce the clays affinity for water than if lime is added. For example, 6% cement added to the soil will have less of an effect on the water absorption than if 6% lime is added.



**Figure 5.11: Water absorption results for lime-cement-fly ash combination**

The fly ash addition has the greatest influence on the absorption properties when the results of lime-cement combinations and lime-cement-fly ash combinations are compared. The water absorbing properties of the blocks were reduced by inclusion of fly ash into the soil-lime-cement combinations. The lime-cement combinations flocculate the clay giving the soil a more open structure [14] thereby increasing the soils void content and therefore, porosity. The addition of fly ash is postulated to fill these voids and therefore reduce the water absorbing properties. It may therefore act as a filler between flocculated particles. By increasing the fly ash content in a soil-lime-cement combination where the lime and cement content is kept constant, the soil component and therefore the clay fraction is also reduced, thereby reducing the potential to absorb water.

The affects of the various stabilisation combinations are illustrated in figure 5.12.

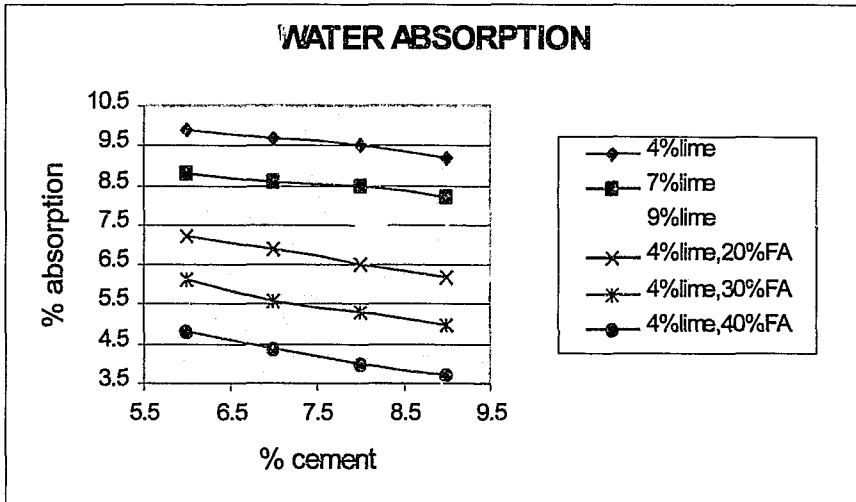


Figure 5.12: Cumulative affect of various stabilisation combinations on water absorptivity.

## 5.5 THE ACCELERATED EROSION TEST

### Scope

This method covers the procedure for determining the rate of erosion of a soil or stabilised soil block exposed to a water jet sprayed onto a fixed target area.

### Objective

This test is essentially a test of durability of the stabilised soil blocks when exposed to the action of driven water. It is conducted to give an idea of how well the material would withstand driven rain when no protective coating has been applied to the block's surface.

### Apparatus

- (i) A suitable location for curing the blocks. The area should consist of a hard clean surface, which is not exposed to direct light or dramatic temperature fluctuations.
- (ii) An accelerated erosion device consisting of a pump capable of delivering a constant pressure of 70kPa of water through a perforated nozzle 50mm in diameter set at a distance of 470mm from the test specimen.
- (iii) A flexible waterproof material which is capable of adhering to the block to mask the 100mm diameter portion the of block.
- (iv) A measuring device which consists of a 300mm long straight edge and a pin, graduated in millimeters and 100mm long, which moves at right angles to the straight edge.
- (v) A 100mm diameter stencil and marking pen.
- (vi) A time keeping device.

## **Method**

For the purpose of this test an apparatus and procedure developed by the National Building Technology Centre in Australia was used. The apparatus directs a spray of water onto a shielded target area. The test is run for one hour or until the block is excessively eroded, and interrupted every fifteen minutes to examine the condition of the block. Test results are recorded on the basis of the average depth of penetration of the water jet after each fifteen-minute interval.

### **Preparation of specimens**

The specimens are standard Hydraform blocks manufactured using the designed mixes and at the moisture contents required to produce suitable blocks.

### **Curing of specimens**

Cure the specimens for 14 days under plastic sheeting. The blocks are then removed from the plastic and placed so that air is allowed to circulate freely around the blocks allowing them to dry for a further fourteen days.

### **Eroding the specimens**

The eroding device should be calibrated by starting the pump and directing the spray into the target area. The pressure should be set at 70 kPa.

A 100mm-diameter circle is marked centrally on the face of the block to be tested. The flexible waterproof material should be adhered around the outside of the perimeter of the circle, to the specimen, to form a continuous seal. The specimen is then mounted in front of the test rig. The pump is then started. After 15 minutes the test should be stopped, the block removed and examined for signs of erosion. A description of deterioration to the surface should be noted and the average depth of erosion measured using the measuring device. The specimen is then replaced and the procedure repeated three more times. If the specimen becomes saturated before one hour, the test should be stopped.

### **Determination of the rate of erosion**

After being exposed to the water jet for a total of one hour the specimen is removed and the final depth of erosion measured. From the data collected a rate of erosion in millimeters per minute can be determined and a descriptive table set up.

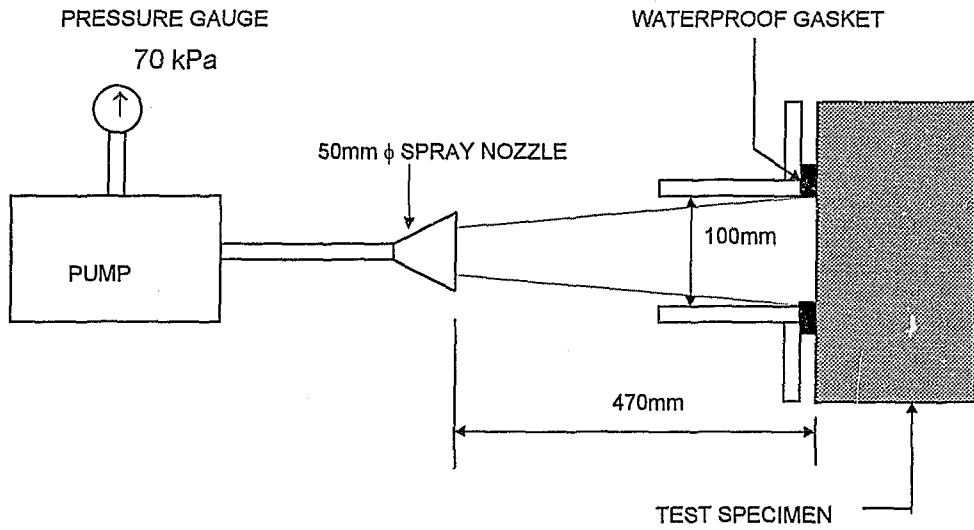
### **Calculations**

The rate of erosion can be calculated using the following formula:

$$R = \frac{D}{T}$$

Where:

- R = the rate of erosion in mm/minute
- D = the depth of erosion at time T in minutes
- T = the time since commencement of the test in minutes.



**Figure 5.13 : Diagram of the Accelerated Erosion test rig**

### 5.5.1 Results of the accelerated water erosion test

		TIME ELAPSED			
		15 min	30 min	45 min	60 min
DEGREE OF STABILISATION	4%	20mm deep pockets of erosion	Stopped test block eroded totally	-	-
	7%	14mm deep pockets of erosion	22mm deep pockets	28mm deep pockets	30mm deep pockets
	9%	11mm deep pockets of erosion	19mm pockets	24mm pockets	28mm pockets

**Table 5.21: Results of the accelerated erosion test for lime-stabilised blocks**

		TIME ELAPSED			
		Mix	15 min	30 min	45 min
DEGREE OF STABILISATION	K1	9mm deep pockets	12mm deep pockets	13mm deep pockets	15mm deep pockets
	K2	10mm deep pockets	12mm deep pockets	13mm deep pockets	No further erosion
	K3	7mm deep pockets	9mm deep pockets	10mm deep pockets	No further erosion
	K4	9mm deep pockets	No further erosion	No further erosion	No further erosion

**Table 5.22: Results of the accelerated erosion test for lime-cement stabilised blocks (4% lime, variable cement)**

		TIME ELAPSED			
	Mix	15 min	30 min	45 min	60 min
DEGREE OF STABILISATION	N1	9mm deep pockets	No further erosion	No further erosion	No further erosion
	N2	6mm deep pockets	8mm deep pockets	No further erosion	No further erosion
	N3	6mm deep pockets	No further erosion	No further erosion	No further erosion
	N4	6mm deep pockets	No further erosion	No further erosion	No further erosion

**Table 5.23: Results of the accelerated water erosion test for the lime-cement stabilised blocks (7% lime, variable cement)**

		TIME ELAPSED			
	Mix	15 min	30 min	45 min	60 min
DEGREE OF STABILISATION	O1	8mm deep pockets	10mm deep pockets	No further erosion	No further erosion
	O2	7mm deep pockets	8mm deep pockets	No further erosion	No further erosion
	O3	8mm deep pockets	No further erosion	No further erosion	No further erosion
	O4	7mm deep pockets	No further erosion	No further erosion	No further erosion

**Table 5.24: Results of the accelerated water erosion test for lime-cement stabilised blocks (9% lime, variable cement)**

		TIME ELAPSED			
	Mix	15 min	30 min	45 min	60 min
DEGREE OF STABILISATION	I1	4mm deep pockets	No further erosion	No further erosion	No further erosion
	I2	3mm deep pockets	No further erosion	No further erosion	No further erosion
	I3	3mm deep pockets	No further erosion	No further erosion	No further erosion
	I4	3mm deep pockets	No further erosion	No further erosion	No further erosion

**Table 5.25: Results of the accelerated water erosion test for the lime-cement-fly ash stabilised blocks (4% lime, 20% fly ash, variable cement)**

		TIME ELAPSED			
	Mix	15 min	30 min	45 min	60 min
DEGREE OF STABILISATION	H1	Very slight roughening of the surface which could not be accurately measured			
	H2	Very slight roughening of the surface which could not be accurately measured			
	H3	Very slight roughening of the surface which could not be accurately measured			
	H4	Very slight roughening of the surface which could not be accurately measured			

**Table 5.26: Results of the accelerated water erosion test for the lime-cement-fly ash stabilised blocks (4% lime, 30% fly ash, variable cement)**

### 5.5.2 Discussion of accelerated water erosion test results

The accelerated water erosion test is mainly a qualitative test, although semi-quantitative results can also be extracted. The results of the test will indicate the depth of erosion occurring over the test period and this can be used to compare the results of the various combinations of stabilizers. A recommendation has also been made that the stabilisation should reduce the depth of the erosion to less than 15mm. [39]

Blocks from each of the mixes were subjected to the erosion test. The only blocks which showed significant erosion, were the blocks which were stabilised with only lime. The degree to which the erosion took place also decreased with an increase in lime content.

The durability of stabilised soil blocks is dependent to a certain degree on the porosity of the block and the bonds between soil particles. The soil agglomerates, which are formed as a result of flocculation by the clay, increase the porosity. When the water which is sprayed against the block, the water forces its way through the voids in the compressed block and breaks the bonds between the soil particles resulting in erosion. The cementitious bonds formed by the reaction between the clay and the lime are not strong enough to 'hold' the soil particles together.

The addition of cement to the lime-soil mix increased the durability of the test specimen markedly, with only very small pockets of erosion being visible. An increase of cement also reduced the number and depth of the eroded pockets. The blocks which included fly ash, did however perform better than the test specimens without. With the degree of erosion decreasing with an increase in fly ash, the blocks which included forty percent fly ash showed no erosion whatsoever. The inclusion of fly ash in a mix also ensured that the surface, which was exposed to the test, was smoother than the surface of blocks which did not

include fly ash. Therefore, the cement has the largest influence in the increase in durability of the block due obviously to its cementing action. The fly ash seems to reduce the porosity of the blocks and therefore increases the blocks' resistance to erosion. The addition of lime to the clay soil results in flocculation of the clay particles, which reduces the density of the block and increases the porosity. This may lead to increased erosion when compared to a mix which includes fly ash, as the fly ash will then act as filler.

## 5.6 EXPANSION DUE TO ABSORPTION OF WATER

### Scope

This method describes the procedure used to determine the amount of expansion, which occurs if a stabilised soil block is soaked in water.

### Apparatus

- (i) A suitable location for curing the blocks. The area should consist of a hard clean surface, which is not exposed to direct light or dramatic temperature fluctuations.
- (ii) A suitable bath of water with sufficient capacity so that all specimens can be fully immersed simultaneously.
- (iii) A balance to weigh up to 20kg and accurate to 0,5g.
- (iv) Two targets per block, consisting of 2mm thick aluminium rectangular plates (15mm x 35mm).
- (v) An epoxy resin based adhesive capable of adhering aluminium to block surfaces, mixing cups and mixing rods.
- (vi) A digital vernier caliper.

### Method

#### Preparation of specimens

The specimens are standard Hydraform blocks manufactured using the designed mixes and at the optimum moisture contents required to produce suitable blocks. Targets are attached to the block using the same procedure described in the drying shrinkage test.

#### Curing of specimens

Cure the specimens for fourteen days under plastic in a suitable location. The blocks are then removed from the plastic and allowed to cure for a further fourteen days in the laboratory.

The aluminium targets are attached as for the linear shrinkage test. A zero reading is taken using the vernier caliper. The blocks are immersed in the tank of water for 24 hours. After the 24 hour period the blocks are removed and the reading across the aluminum targets taken using the vernier caliper. The blocks are then left to air dry in the laboratory for a period of seven days. The reading across the targets is then taken to determine the shrinkage.

### Determining the expansion

After the block has been soaked for 24 hours the reading across the targets is taken and the amount of expansion determined.

### Calculations

The amount of expansion is calculated as follows:

$$E = \left( \frac{L_1 - L_0}{L_0} \right) \times 100$$

Where:

- E = The percentage expansion expressed as a percentage of the zero reading.
- L<sub>0</sub> = zero reading across the targets in mm.
- L<sub>1</sub> = the reading taken after the block has been immersed in water for 24 hours measured in mm.

### **5.6.1 Results of the test for expansion due to the absorption of water.**

The expansion of blocks produced with mix L and M could not be determined as the blocks disintegrated when placed in water.

<b><u>Mix</u></b>	<b><u>% Expansion</u></b>
K1	0.81
K2	0.76
K3	0.74
K4	0.72

**Table 5.27: Expansion of lime-cement stabilised blocks (4% lime, variable cement)**

<b><u>Mix</u></b>	<b><u>% Expansion</u></b>
N1	0.70
N2	0.68
N3	0.64
N4	0.61

**Table 5.28: Expansion of lime-cement stabilised blocks (7% lime, variable cement)**

<b><u>Mix</u></b>	<b><u>% Expansion</u></b>
O1	0.58
O2	0.56
O3	0.53
O4	0.50

**Table 5.29: Expansion of lime-cement stabilised blocks (9% lime, variable cement)**

<u>Mix</u>	<u>% Expansion</u>
I1	0.43
I2	0.37
I3	0.34
I4	0.33

**Table 5.30: Expansion of lime-cement stabilised blocks (4% lime, 20% fly ash, variable cement)**

<u>Mix</u>	<u>% Expansion</u>
H1	0.30
H2	0.26
H3	0.22
H4	0.20

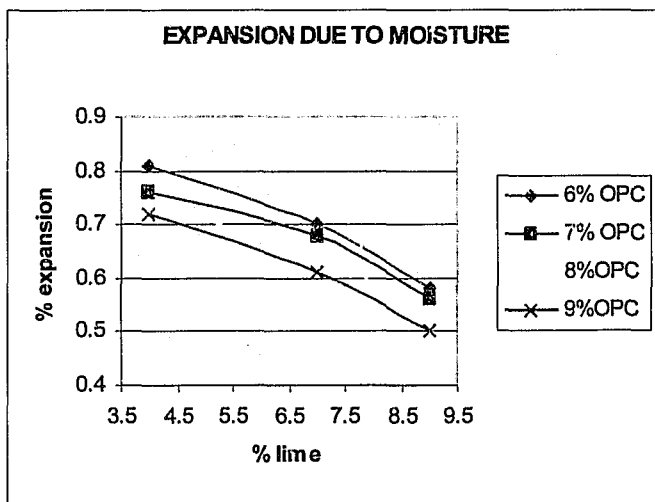
**Table 5.31: Expansion of lime-cement stabilised blocks (4% lime, 30% fly ash, variable cement)**

<u>Mix</u>	<u>% Expansion</u>
J1	0.17
J2	0.14
J3	0.10
J4	0.08

**Table 5.32: Expansion of lime-cement stabilised blocks (4% lime, 40% fly ash, and variable cement)**

### 5.5.2 Discussion of test results

The lime-stabilised blocks could not be tested for expansion as they disintegrated in water. The expansion of the blocks stabilised with lime-cement show a similar trend to the results of the water absorption test, where an increase in cement at a constant lime content reduces the amount of expansion. The increase in cement will result in a higher degree of cementation between soil particles therefore providing the block with an increased volume stability. The increase in lime content as discussed before reduces the clay's affinity for water and therefore reduces the potential of the block to swell as a result of water absorption.



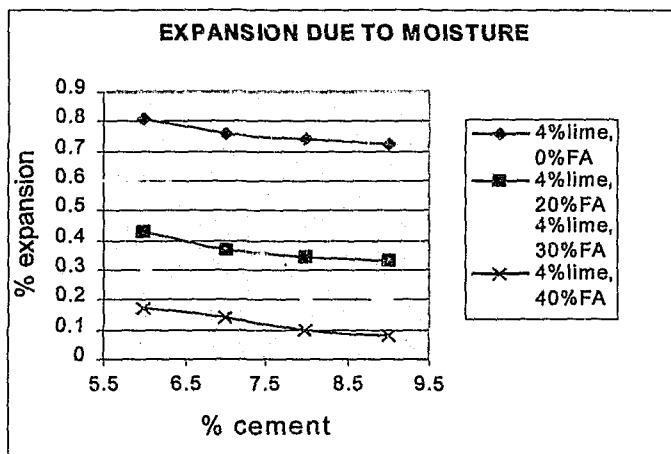
**Figure 5.14: Effect of lime and cement content on the expansion due to absorption of water**

When comparisons are made between combinations which have the same lime and cement content with increasing fly ash contents e.g. K1, I1, H1, and J1 (see figure 5.15) the initial addition of 20% fly ash results in a reduction of 59.4%, by increasing the fly ash content to 30% a further reduction of 20.3% is achieved

and when the fly ash content is raised to 40% another 20.3% reduction in the expansion is achieved. A similar trend is observed at higher cement contents.

The fly ash as is the case in the water absorption test, reduces the clay fraction in the soil-lime-cement combination, thereby reducing the expansion. The cementitious bonds are also increased as the pozzolanic reaction occurs between the fly ash and lime produced during the hydration of the cement.

The results of the expansion test correlate with the results of the water absorption test, where the mix with the highest water absorption result also has the highest amount of expansion. As the amount of water absorbed decreases, the expansion also decreases.



**Figure 5.15: Effect of fly ash content on expansion due to absorption of moisture**

The expansion which took place in the specimens tested is low, for example over the average length of the blocks i.e. 130mm the expansion ranged between 1.05mm and 0.1mm. The author believes that all the results of the expansion test are acceptable, especially as the chance of a block becoming totally saturated in a wall as in the test is unlikely and therefore the blocks will perform even better in field conditions.

## **5.7 MINERALOGICAL ANALYSIS OF CHANGES DUE TO STABILISATION**

The physical tests conducted on the blocks produced from the stabilised and unstabilised soil provides evidence of the improved performance of the Panfontein soil as a result of the stabilisation process.

A study using x-ray diffraction analysis was undertaken to determine the mineralogical changes which occur in the Panfontein soil as a result of the addition of the stabilising agents used in the production of the compressed earth blocks. The objective of the study was to provide evidence of the theoretical reactions, which take place in the compressed earth blocks and result in the improved performance of the sludge, as indicated by the physical tests described earlier in this chapter.

This improved performance is attributed to two reactions. The first is the modification of the clay mineral in the soil by the addition of lime and secondly, the formation of a cementitious matrix in the soil as a result of the addition of cement-fly ash. A x-ray diffraction analysis was undertaken to provide evidence of the two reactions having taken place in the soil.

The reduction in the height of a peak in an XRD diffraction pattern, associated with a clay mineral is considered to be evidence of the modification of the clay mineral by the addition of lime. [18] The changes in the physical properties and the mineralogy of the soil with the addition of lime was investigated and reported in chapter 3. The results are included here to enable comparisons with samples which included lime, cement and fly ash.

To provide evidence of the formation of a cementitious matrix it was necessary to identify the presence of hydration products of cement in the stabilised soil. The cementitious products formed are identified by peaks formed at  $6.32^{\circ}2\theta$  -  $7.37^{\circ}2\theta$  (12 - 14A) and  $29.79^{\circ}2\theta$  -  $29.08^{\circ}2\theta$  (3.0 - 3.07A), these are the peaks

associated with calcium silicate hydrate. The peaks associated with hydrated calcium aluminates can be found at  $10.78^{\circ}2\theta$  (8.2A),  $36.68^{\circ}2\theta$  (2.45A) and  $39.7^{\circ}2\theta$  (2.27 A). [10] These are the peaks that would be associated with the improved performance of the soil as a result of the formation of a cementitious matrix.

Therefore the presence of the peaks associated with the hydration products of cement and a reduction in the peaks associated with the clay minerals would be considered to be evidence of the reactions assumed to be responsible for the improved performance of the soil when stabilised.

#### **Mix design for X-ray diffraction analysis.**

To determine the mineralogical changes, samples prepared using the design parameters shown in table 5.33 were used. Samples of the soil stabilised with lime, lime-fly ash, lime-fly ash-cement and lime cement combinations were investigated. Details of the stabiliser combinations are given in table 5.33.

The aim of the design mixes used in the XRD study was to investigate how the addition of the different stabilisers would affect the mineralogy of the sludge especially the clay fraction. It was hoped that by showing a reduction of the peaks, which are associated with/indicative of the clay minerals, it would be possible to explain the changes in physical properties of the sludge. This was done by first looking at the effect the addition of lime on its own would have on the sludge in dosages of 4% and 7% by dry mass of the sludge, this was discussed in chapter 3. The next investigation was to determine what effect fly ash would have on a sludge-lime and fly ash mix. This was investigated using the parameters detailed in the C1 and C2 series in table 5.33. The next step was to determine the changes, which take place as a result of the addition of ordinary Portland cement. The effects of the addition of OPC to the sludge-lime-fly ash mix were investigated using the parameters detailed in the D1 to D4 series in

table 5.32. A study was also undertaken to try to identify the changes, which take place with only the addition of lime and cement to the sludge. The lime content in the sludge-lime-cement mix was varied. The parameters used are detailed by samples K1, K4, N1, N4, O1 and O4.

In order for a change in the clay mineral to be detected, it was necessary to obtain an XRD spectrum of an unstabilised sample of the sludge. This would indicate the type or types of clay mineral/s present in the sludge. This spectrum could then be compared to the spectra of samples where lime, cement and fly ash had been added to the sludge to analyse the affect of each stabilising agent on the sludge.

#### **Preparation of samples.**

The samples, which were analysed by XRD were produced using the same process as that used to manufacture the blocks for the physical tests. Once the blocks were produced, they were individually sealed in plastic bags and left to cure. The XRD was conducted on two samples from each mix design. The first sample was analysed after seven days of curing and the second sample after twenty-eight days curing. This was done to monitor the changes over the curing period used in blocks produced for the physical tests. After the specific curing period the samples were taken to the x-ray diffraction laboratories at The Council for Geoscience for analysis.

## Mix design parameters

Sample	Sludge	Lime	Fly Ash	Cement
A0	100%	0%	0%	0%
B1	96%	4%	0%	0%
B2	93%	7%	0%	0%
C1	76%	4%	20%	0%
C2	56%	4%	40%	0%
D1	70%	4%	20%	6%
D2	67%	4%	20%	9%
D3	50%	4%	40%	6%
D4	47%	4%	40%	9%
K1	90%	4%	0%	6%
K4	87%	4%	0%	9%
N1	87%	7%	0%	6%
N4	84%	7%	0%	9%
O1	85%	9%	0%	6%
O4	82%	9%	0%	9%

**Table 5.33: Mix design parameters used in x-ray diffraction analysis.**

### **5.7.1 Results of the XRD analysis**

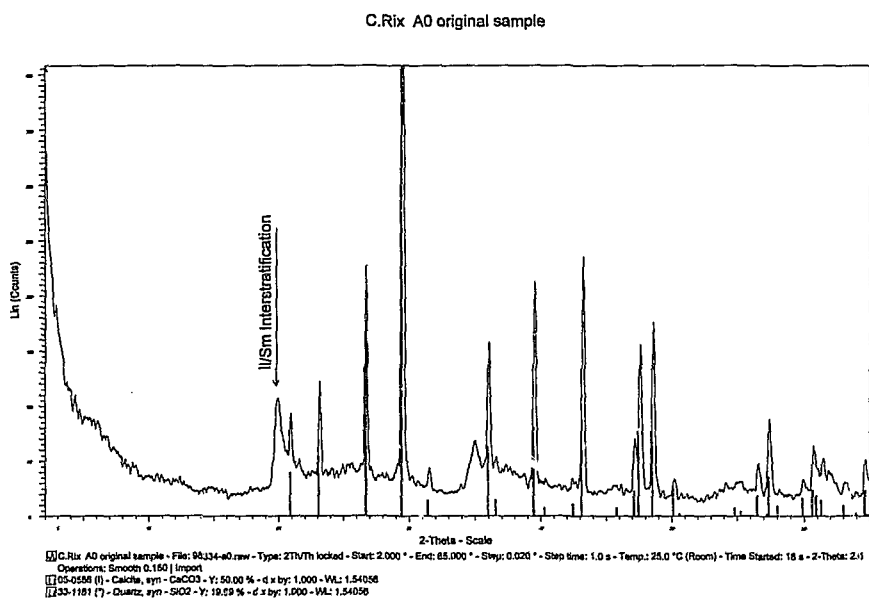
The main objective of the analysis was to provide evidence of the changes afforded by the addition of the stabilising agents. The results are presented in the form of spectra to show qualitative changes in the peak intensity of the clay mineral and the formation of peaks indicating the hydration products of the cement to provide evidence of mineralogical changes to correlate with the improvements in performance of the blocks in the physical tests. A quantitative analysis was also done in order to facilitate explanation of the results. The quantitative results are reported in table 5.34.

Sample	Calcite		Portlandite		Mullite		Quartz		Illite/Smeectite Interstratification	
	7 days	28 days	7 days	28 days	7 days	28 days	7 days	28 days	7 days	28 days
A0	70	70	-	-	-	-	10	10	20	20
B1	71	72	-	-	-	-	10	10	19	18
B2	68	70	3	2	-	-	10	10	19	18
C1	63	67	-	-	9	6	10	10	18	17
C2	54	57	-	-	20	18	10	10	16	15
D1	59	61	5	3	10	11	10	10	16	15
D2	58	61	6	5	8	9	10	10	18	15
D3	53	55	4	3	18	19	10	10	15	13
D4	46	54	11	4	19	19	10	10	14	13
K1	70	71	-	-	-	-	10	10	20	19
K4	69	71	3	-	-	-	10	10	18	19
N1	66	65	5	7	-	-	10	10	19	18
N4	67	66	5	8	-	-	10	10	18	16
O1	67	64	6	9	-	-	10	10	17	17
O4	63	64	9	10	-	-	10	10	18	16

Table 5.34: Results of XRD quantitative analysis

## 5.7.2 Discussion of the XRD results.

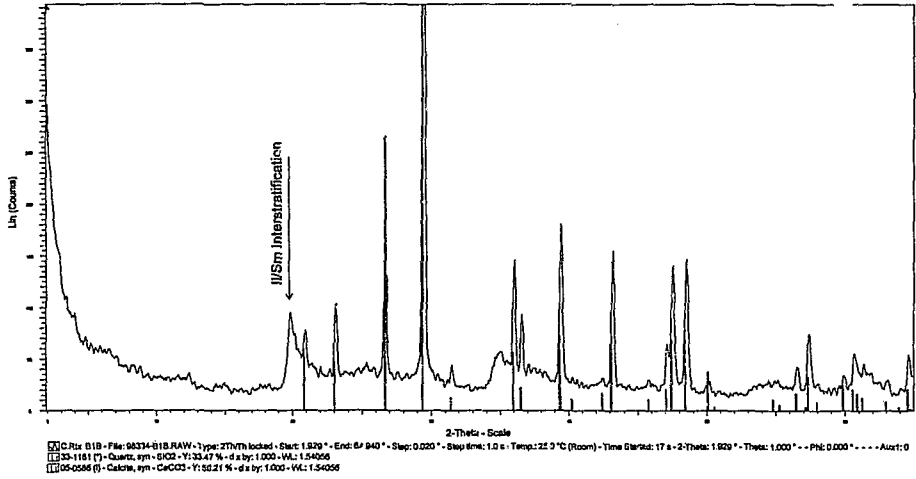
The unstabilised sample of sludge is composed of 10% quartz, 70 % calcite and 20% Illite/smectite interstratification. The calcite is calcium carbonate which is presumed to be as result of carbonation of the calcium hydroxide introduced during the water purification process. The illite/smectite clay mineral fraction is of an irregular stacking arrangement as determined by Böhman.



**Figure 5.16: XRD spectrum of the unstabilised Panfontein soil.**

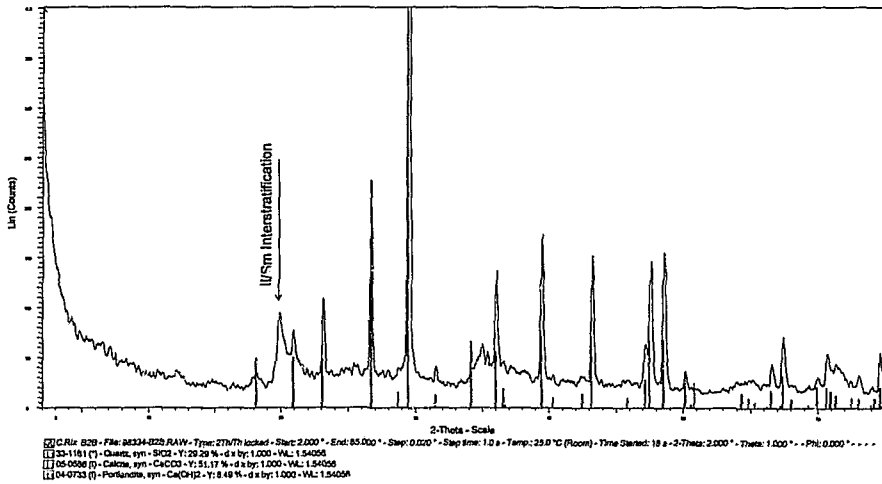
The mixes B1 and B2 show the addition of lime to the sludge in dosages of 4% and 7% by dry mass of the soil respectively. The results of B2 show the presence of 3% Portlandite (calcium hydroxide) which seems to indicate that the sludge has reacted with some of the lime added, namely 4% of the 7% added. The fact that there is no Portlandite present in B1, which only contained 4% lime originally also reinforces the findings of the Initial consumption of lime test, which indicated that the sludge had a lime fixation point of 4%.

C.Rix B1B



**Figure 5.17: XRD spectrum of the Panfontein soil stabilised with 4% lime.  
(mix B1)**

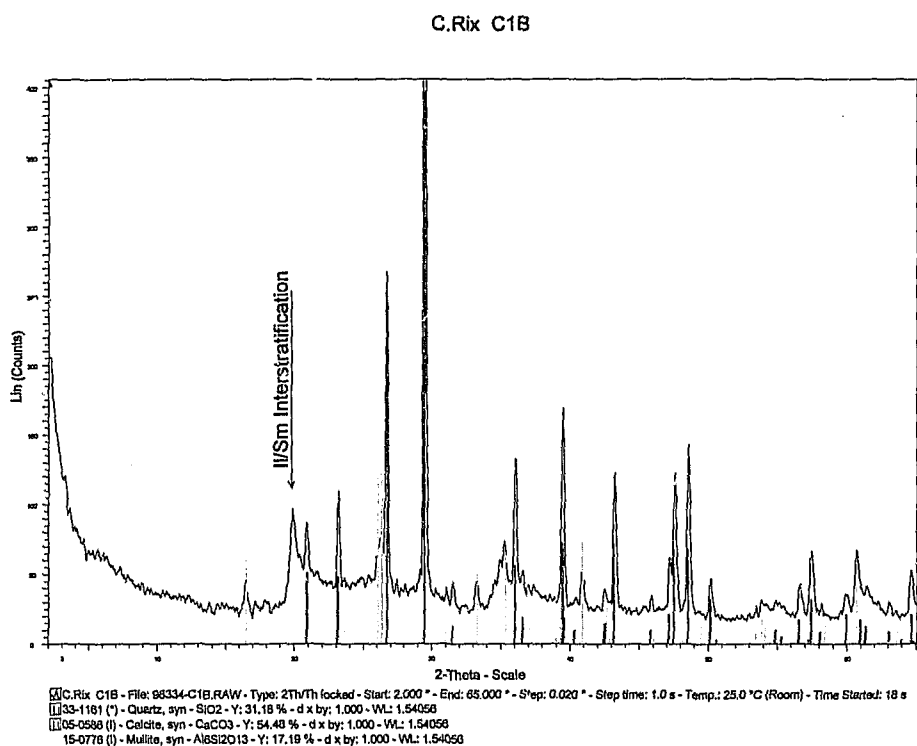
C.Rix B2B



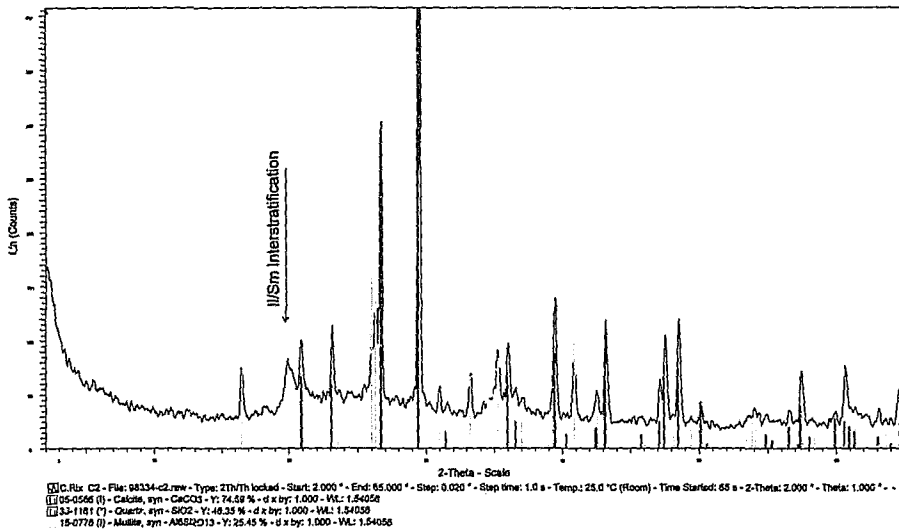
**Figure 5.18: The XRD spectrum for the soil stabilised with 7% lime.  
(mix B2)**

## Lime-fly ash combinations

The spectra shown in figure 5.19 and 5.20 are the spectra of the soil stabilised with 4% lime and fly ash. The change in spectra compared to B1 and B2 is the formation of peaks indicative of Mullite. Mullite is the mineralogical name given to a high temperature phase of aluminium silicate found in fly ash. The actual decrease in the peak height of the clay mineral was very small.



**Figure 5.19: The XRD spectrum for the soil stabilised with 4% lime and 20% fly ash. (mix C1)**



**Figure 5.20: The XRD spectrum for the soil stabilised with 4% lime and 40% fly ash. (mix C2)**

### Lime-fly ash-cement combinations

In the results of the D1 and D2-series Portlandite is picked up in the mixes, which were tested after seven days of curing. Since it was shown in the B1 and C1-series that no Portlandite is present after seven days the Portlandite which appears can only be assumed to be a result of the hydration of the cement added. The increase in the percentage of Portlandite identified in D1 and D2 as well as in D3 and D4 is as a result of there being more cement in D2 than D1 originally (9% and 6% respectively), the same applies to D4 and D3. The reduction in the percentage of clay mineral is the most significant in this series. This is probably as a result of the substitution effect, with the addition of fly ash replacing some of the clay in the mix, this can also be seen by the reduction in the percentage of calcite, which reduces with an increase in the percentage fly ash.

C.Rix D1B

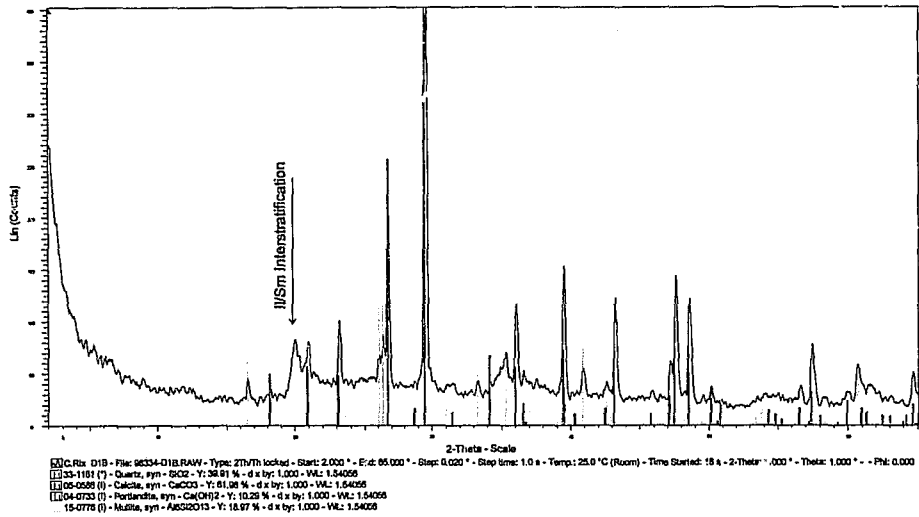


Figure 5.21: XRD spectrum of soil satabilised with 4% lime, 20% fly ash and 6% cement. (mix D1)

C.Rix D2B

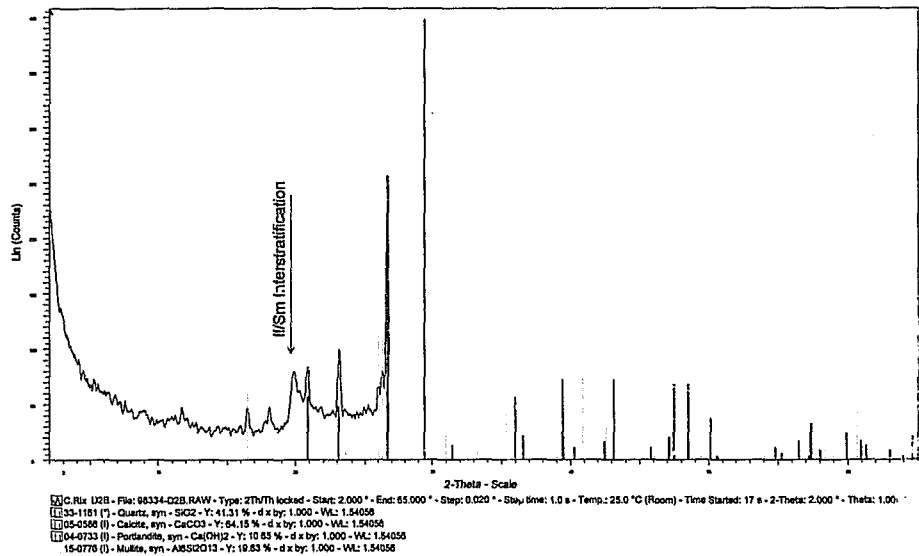


Figure 5.22: XRD spectrum of soil stabilised with 4% lime, 20% fly ash and 9% cement. (mix D2)

C.Rix D3B

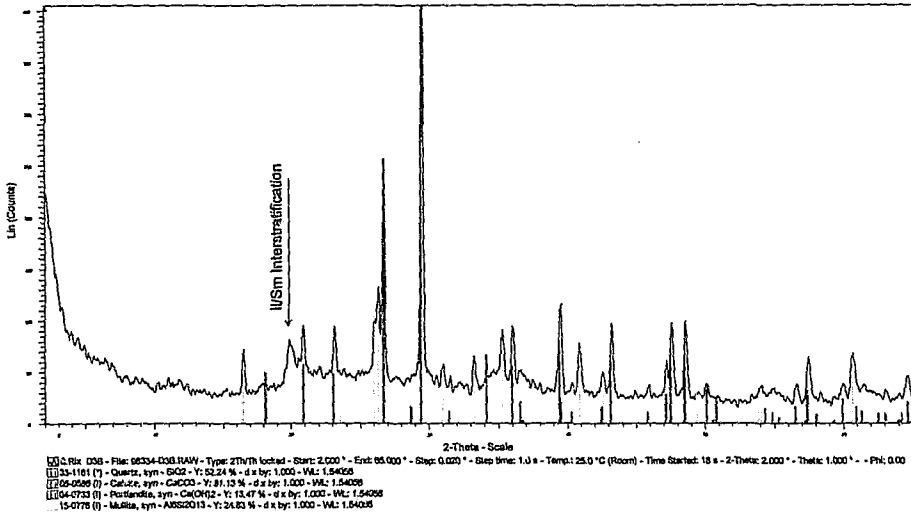


Figure 5.23 : XRD spectrum of soil satabilised with 4% lime, 40% fly ash and 6% cement. (mix D3)

C.Rix D4B

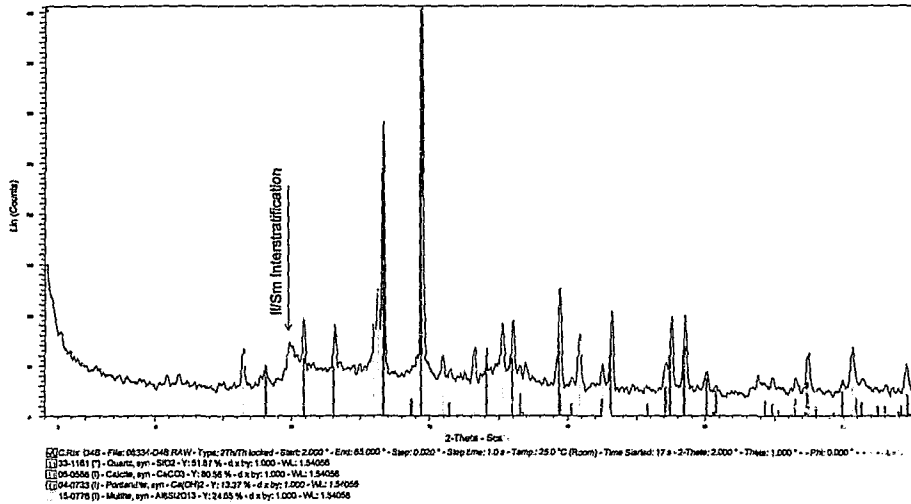
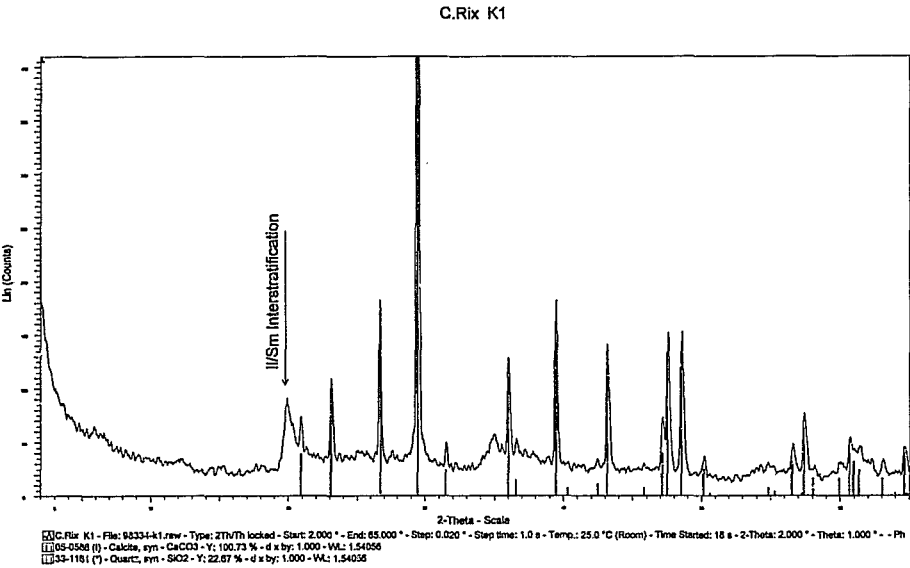


Figure 5.24: XRD spectrum of soil satabilised with 4% lime, 40% fly ash and 9% cement. (mix D4)

The effect of combinations of only cement and lime on the sludge was also investigated. The author decided to see what an effect an increasing percentage of lime with a constant cement content would have on the sludge. The difference in Portlandite levels when comparing K1, N1 and O1 which have the same cement content, namely 6% but differ in lime content (4%, 7% and 9% respectively) is assumed to be as a result of the combination of the calcium hydroxide which is not consumed by the clay during the modification process and the calcium hydroxide produced during the hydration of the cement.

The same trend is evident in the K4, N4 and O4 samples, which have a 9% cement content instead of a 6% content, except the Portlandite levels are higher as a result of there being more cement in the mix which will produce more calcium hydroxide than the mixes with 6% cement.



**Figure 5.25: XRD spectrum of soil stabilised with 4% lime and 6% cement. (mix K1)**

C.Rix N1B

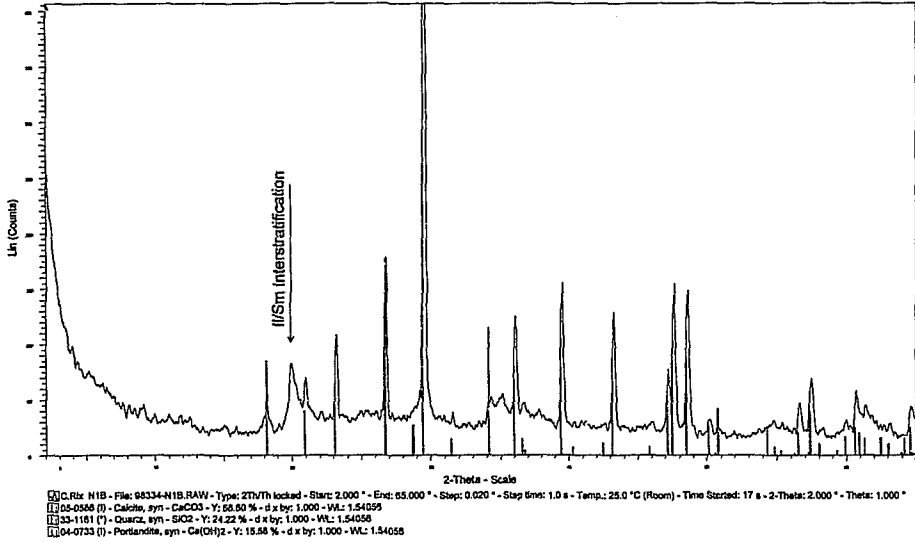


Figure 5.26: XRD spectrum of soil stabilised with 7% lime and 6% cement.  
(mix N1)

C.Rix O1

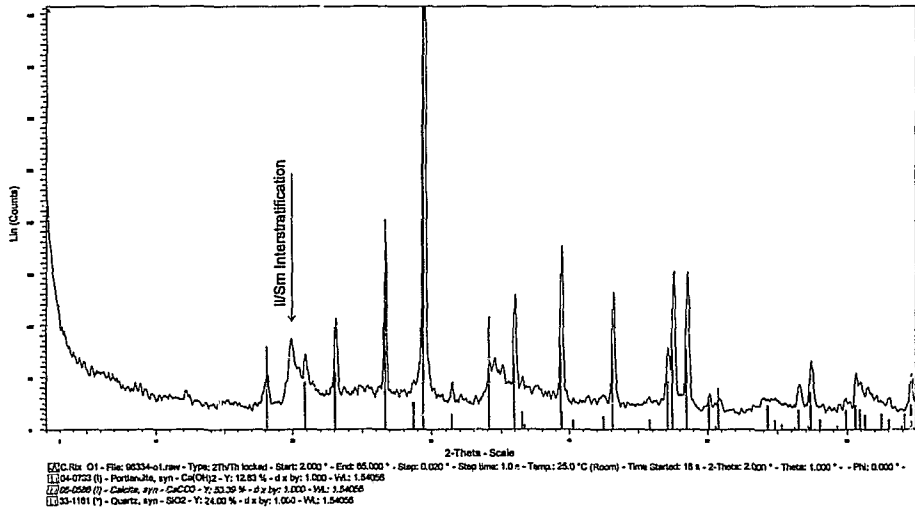


Figure 5.27: XRD spectrum of soil stabilised with 9% lime and 6% cement.  
(mix O1)

C.Rix K4B

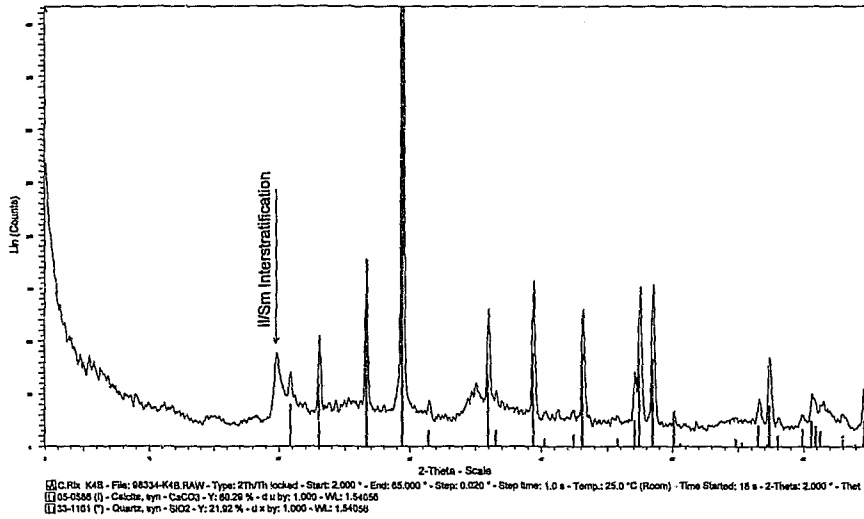


Figure 5.28: XRD spectrum of soil stabilised with 4% lime and 9% cement.  
(mix K4)

C.Rix N4B

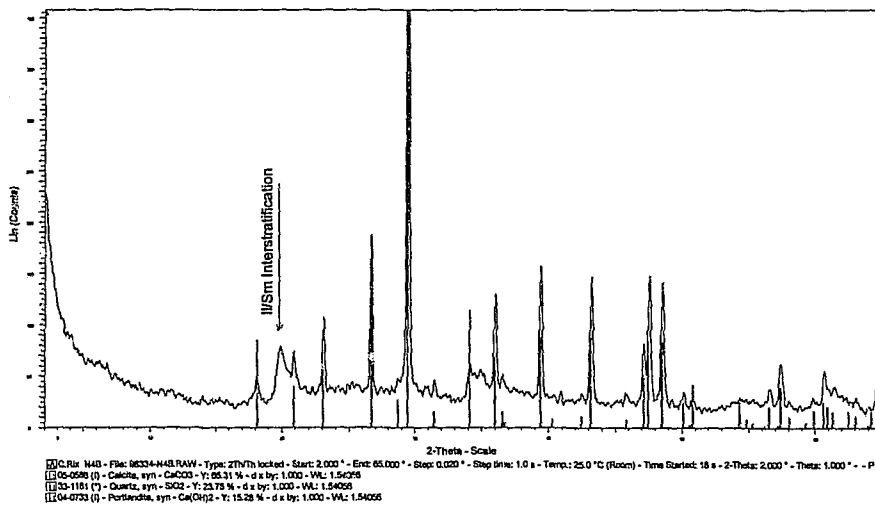
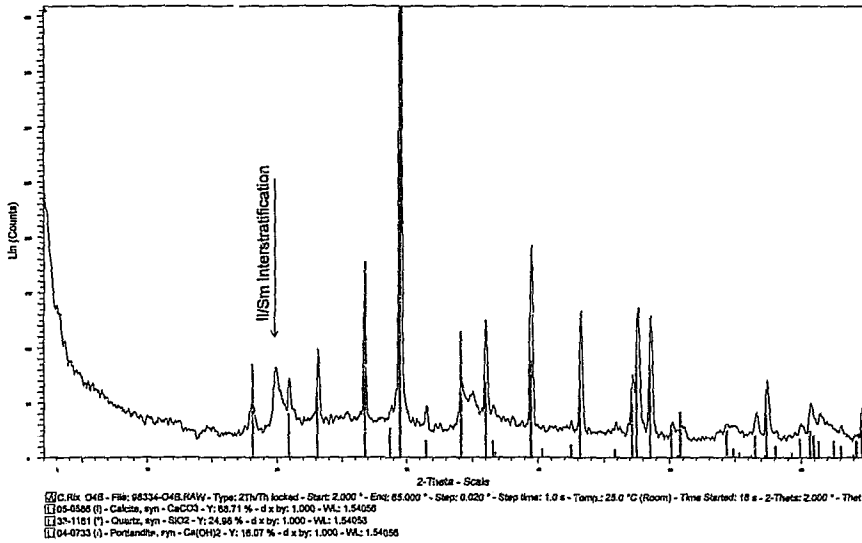


Figure 5.29: XRD spectrum of soil stabilised with 7% lime and 9% cement.  
(mix N4)

C.Rix O4B



**Figure 5.30: XRD spectrum of soil stabilised with 9% lime and 9% cement.  
(Mix O4)**

## Conclusions

The results of the analysis which were hoped for was a decrease in the intensity of the clay mineral peak to indicate modification by the reaction between lime and the clay fraction. The presence of peaks related to the hydration products of cement namely calcium silicates would correlate with the improved strength and durability characteristics of the stabilised soil blocks. The results however did not provide proof of either of these changes having taken place as was suggested by the results of the physical tests.

The only evidence of the hydration of the cement in the XRD results is the presence of Portlandite. The x-ray traces for the samples which contained only lime and cement showed a higher percentage Portlandite than the other mixes subjected to x-ray diffraction. This is assumed to be as a result of the calcium hydroxide being produced during the hydration of the cement. The percentage is higher than those in the mixes which contained fly ash, because the excess calcium hydroxide is not used up during the pozzolanic reaction with the fly ash.

The author believes that the clay mineral in addition to having an interstratified stacking arrangement (see section 1.4), has a poorly crystallised structure. This poorly crystallised structure does not give good reflections in the spectra and therefore the peaks required to make a positive identification of the changes in the clay mineral are not present.

The lack of evidence of the hydration products in the spectra may be accredited to the fact that the products may be of an amorphous nature. Work done at the Swedish Cement and Concrete Research Institute [26] has shown that carbonation of calcium hydroxide results in the formation of calcite, which covers the silicate minerals. The high percentage of calcite (calcium carbonate) therefore may also cover the silicate minerals.

# CHAPTER SIX

## 6.0 GENERAL CONCLUSIONS

The objective of this research was to determine the properties and composition of the 'sludge' from Rand Water and to explore ways in which the highly plastic clayey soil could be stabilised to produce building blocks of an acceptable standard. The standard used is discussed in section 2.5. Wet and dry compressive strength, water absorption and durability in terms of water erosion were used as the principal criteria to determine what combination of stabilisers provided sufficient improvement in the performance of the blocks for them to meet the required standards.

The results of the physical tests have shown that the addition of lime to the soil does not provide the blocks with sufficient dry and wet strength after 28 days of curing to be considered a suitable stabiliser used on its own. This however was expected as the lime was added for its clay modification properties and not its cementing properties. The reduction in shrinkage when comparing the unstabilised block and the block stabilised with lime is evidence of this modification. Results for tests, which involved soaking, could not be obtained as the blocks disintegrated when placed in water. The lack of durability of the lime-stabilised blocks indicated by the accelerated water erosion test is also evidence of the failure of lime as the sole stabilising agent for this soil.

The addition of cement permitted the full range of evaluation to be carried out and yield results for all the tests conducted. By using lime-cement combinations the physical properties of the blocks were improved significantly. The addition of 6% ordinary Portland cement with the 4% lime improved the dry compressive strength from 2.84 MPa for the 4% lime-stabilised blocks, to 5.57 MPa for the lime-cement stabilised blocks. The cement addition also provided the block with a wet compressive strength of 2.44 MPa, where the lime-stabilised blocks disintegrated in the water and are therefore assumed to have no wet strength. A further increase (6-9%) in the cement content increased both the wet and dry

compressive strength. The drying shrinkage was improved by the addition of 6% cement, although not substantially (3.26% to 2.78%). A further increase in the cement content reduced the shrinkage still further. The improvement in the performance of the blocks as a result of the inclusion of cement in terms of the water absorption and expansion tests could not be gauged as comparative results for these tests of the lime-stabilised blocks could not be determined. However, the fact that results were obtained for these tests is evidence of the improved performance.

Although the properties of the lime-cement stabilised blocks exceed the standards set, the flocculation of the clay minerals by the lime, give the surface of the blocks a porous appearance, which the author believes is not ideal as this is potentially a weakness in the durability of the block.

The addition of fly ash decreases the porosity of the block, resulting in the surface of the block having a smoother finish. The results of the tests improved with increases in the addition of fly ash, except for the dry compressive strength which is probably as a result of the replacement of a percentage of the clay by the fly ash, where the clay in the dry state has higher strength than the fly ash. The effect of the fly ash is seen in the results of the wet compressive strength test. The wet strength of the combinations with fly ash is higher than the combinations which are identical (i.e. same percentage lime and cement) but without fly ash.

The stabilisation reactions which take place as a result of the addition of lime and cement as indicated by the results of the physical tests were not apparent in the mineralogical analysis conducted. The changes in the mineralogy were not shown in the XRD spectra of samples tested. This is probably as a result of The clay mineral having a poorly crystallised structure. The only evidence of the hydration products of the portland cement is the calcium hydroxide which is found in samples stabilised with lime and cement, as the cement addition

increases so the amount of calcium hydroxide also increases. The addition of the fly ash reduces the total amount of clay in a mix and probably reacts with the excess calcium hydroxide to form cementitious products similar to those produced during the hydration of the cement.

The results of the physical tests are therefore a more reliable indicator of the reactions related to lime and cement stabilisation having taken place.

As discussed in 5.1.1 the results of the compressive strength test, water absorption and accelerated water erosion test would be compared to limits for these tests which have been suggested by various organisations worldwide. These limits would be used to determine which combination of the stabilisers would produce a block which could be used in the construction of single-storey buildings. Based on the results achieved in the above tests and taking into account the variance in performance between laboratory conditions and field conditions the author believes that the mix which consists of 4% lime, 7% cement and 20% fly ash (i.e I2) produces a block which performs well in the tests.

Although a number of the other mixes also produced blocks which have sufficient compressive strength to qualify, the blocks general appearance, especially the smoothness of the faces also influenced the final decision. It is suggested that the more smooth the surface generally the more durable a block will be. The mixes which did not contain fly ash had rougher surfaces than those which did. The author believes that this will potentially be a weakness in the durability of the block and the addition of even 20% fly ash provides the block with a smooth surface capable of resisting erosion by driven water. As the percentage fly ash increases so does the smoothness of the face of the block. The addition of 20% fly ash is sufficient to produce a durable block.

Although the compressive strength of the mix with 6% OPC (I1) is above that required, the author feels that with the difference in the laboratory and field

production conditions 7% OPC will assure that the standards set will be achieved.

The results indicate that this clayey soil can be used to produce building blocks using lime, cement and fly ash as stabilising agents. The author believes that a combination of 4% lime, 7% cement and 20% fly ash added to the soil compacted at an optimum moisture content of 23.4% produces a block which can be used for construction of single storey buildings.

# **APPENDICES**

## **A.1. Descriptions of the testing procedures used in the laboratory tests to determine the physical properties of the soil.**

### **A.1.1 Atterberg Limits**

#### **The determination of the liquid limit of the soil**

##### **Definition**

The liquid limit of a soil is the moisture content, expressed as a percentage of the mass of the oven-dried soil, at the boundary between the liquid and plastic states. The moisture content at this boundary is arbitrarily defined as the liquid limit and is the moisture content at which the two halves of a soil cake will flow together for a distance of about 10 mm along the bottom of the groove separating the two halves, when the cup of the liquid limit device is dropped 25 times through a distance of 10mm at a rate of two drops per second.

##### **Scope**

This method covers the determination of the liquid limit of soils as defined above, using the liquid limit device, based on the results of at least three trials.

##### **Apparatus**

- i) A liquid limit device (figure 3.1). This device consists of a brass bowl 27mm deep, cut from a hollow sphere of 108mm internal diameter and 2mm thick. This bowl is hinged at one end and a cam device enables it to be

tapped by being dropped exactly 10mm onto the base at a rate of 2 taps per second.

- ii) A grooving tool (figure 3.2) used in conjunction with the bowl consists of a handle to which is attached one end of a blade in the form of a quadrant of a machined ring, 44mm internal diameter, with a equilateral triangular section having 13,5mm faces, one flat being on the inside of the ring , the outer edge having been machined to produce a flat surface 2mm wide.

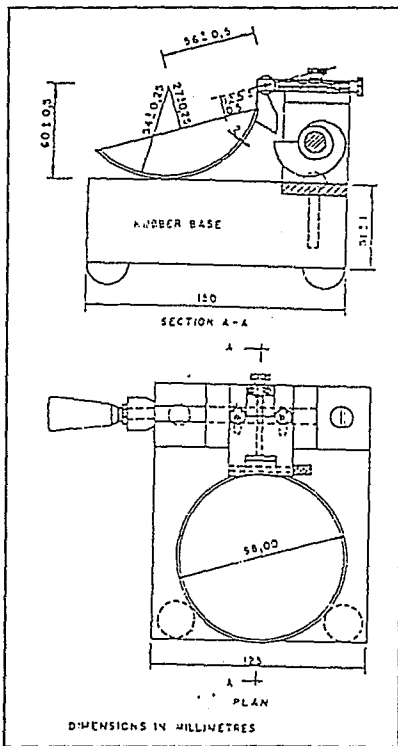


Figure 3.1 : Liquid Limit device

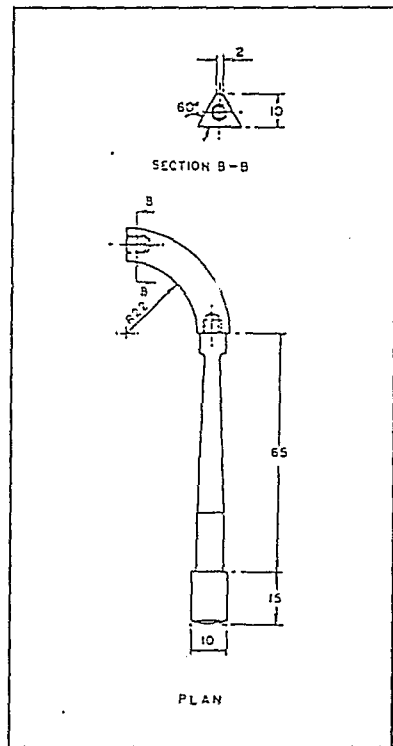


Figure 3.2 : Grooving tool

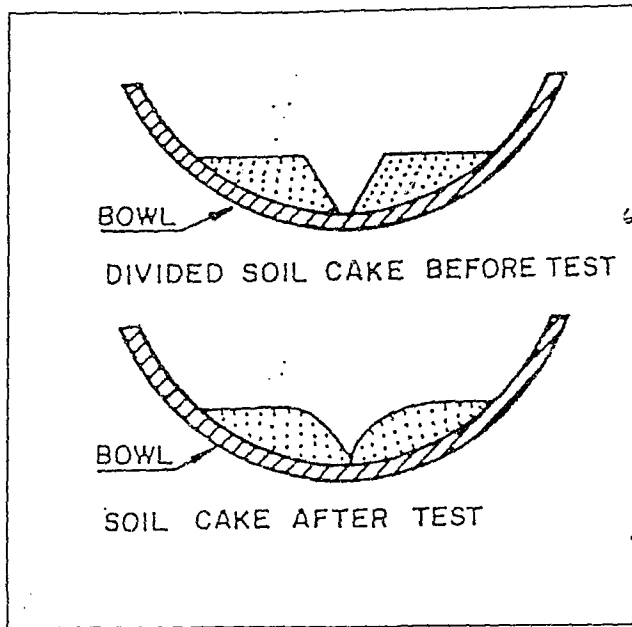
In addition, the apparatus includes:

- iii) A calibrating plate with a thickness of 10 mm.
- iv) Distilled water in a squeeze bottle.
- v) A porcelain evaporating dish with a diameter of about 100mm.
- vi) A spatula with a slightly flexible blade about 100mm long and 20mm wide.
- vii) Suitable containers, such as weighing bottle with capacities of 30ml to 45ml.
- viii) A balance to measure up to 100g, sensitive to 0,01g.
- ix) A drying oven, thermostatically controlled, and capable of maintaining a temperature of 105 to 110°C.

#### Method

Test procedure:

48g of the thoroughly mixed soil fines ( i.e. the material passing the 0,425mm sieve), is measured out and transferred to a porcelain dish. Distilled water is added and the moist material is then thoroughly mixed for 10 minutes with sufficient water to form a stiff consistency, approximately three-quarters of the wet soil is transferred to the brass bowl of the liquid limit device, mixed slightly and flattened out in the front portion of the bowl with the spatula. The material is then divided into two equal portions with one cut of the grooving tool. The groove should fall along the centerline drawn through the cam-follower attached to the bowl. The device is then operated at a speed which results in two taps per second being applied to the soil, until the lower parts of the faces of the two soil portions have flowed together and made contact across a distance of about 10mm.



The number of taps required to close the groove across the distance is recorded and a sample of approximately 2 to 3g, representative of the total thickness of the layer, is transferred to a tared weighing bottle for the determination of its moisture content.

The procedure is repeated for additional determinations on the soil in the bowl of the liquid limit device. However, before each test 2 to 3g of material from the porcelain dish is added to the material in the bowl, to ensure that the quantity of material in the bowl is approximately the same for all determinations. Sufficient distilled water is added each time in order to obtain three samples of varying consistencies, so that at least one determination will be made in preferably each of the following range of taps: 28 to 35, 22 to 28, and 15 to 22. The test should preferably proceed from the drier to the wetter condition of the soil.

The moist material, which has been left over in the porcelain dish, is set aside for the determination of the plastic limit and the linear shrinkage. The masses of the containers with the soil samples are measured, after which the samples are oven dried (105 - 110°C) to a constant mass. As a rule the material is dried overnight.

When the containers are removed from the oven, the lids are replaced to prevent the absorption of moisture, and after allowing them to cool, their masses are obtained again. The loss in mass is the mass of water, which is expressed as a percentage of the oven-dried mass of the soil.

#### Calculations

##### Moisture Content

The moisture content of the soil is expressed as a percentage of the mass of the oven-dried soil and is calculated as follows:

$$\text{Moisture content (\%)} = \frac{\text{mass of wet soil} - \text{mass of dried soil}}{\text{mass of dry soil}} \times 100$$

The calculations should be carried out to the first decimal place.

##### Liquid Limit:

A "Flow curve" representing the relationship between moisture content and corresponding number of taps is then drawn by plotting the moisture contents as ordinates on a linear scale and the number of taps as abscissa on a logarithmic scale. The flow curve should be a straight line drawn as nearly as possible through the three plotted points (see example in figure 3.3).

The moisture content corresponding to 25 taps is then read off the curve and is taken as the liquid limit of the soil and reported to the nearest whole number.

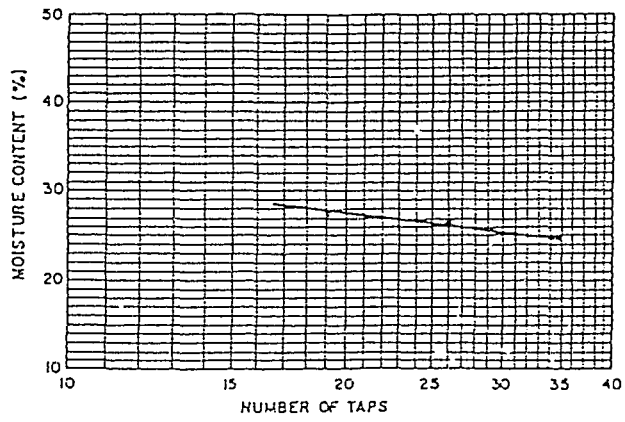


Figure 3.3 : Example of a flow curve ( TMH; 1986; pg 14 & 15 )

## **A1.2 The determination of the plastic limit and the plasticity index of the soil.**

### **Definition:**

The plastic limit of the soil is the moisture content, expressed as a percentage of the mass of the oven - dried soil, at the boundary between the plastic and semi – solid states.

The plasticity index is the numerical difference between the liquid limit and the plastic limit and indicates the magnitude of the range of the moisture contents over which the soil is in a plastic condition.

### **Scope**

This method covers the determination of the plastic limit of a soil, as defined above by measuring the lowest moisture content at which a soil can be rolled into threads 3mm in diameter without the threads crumbling. It also covers the calculation of the plasticity index from the liquid limit and the plastic limit.

### **Apparatus:**

- (i) A ground - glass plate measuring some 150mm x 220mm.
- (ii) Containers, such as weighing bottle with capacities of 30mm - 45mm.
- (iii) A balance to measure up to 100g, sensitive to 0,01g.
- (iv) A drying oven, thermostatically controlled, and capable of maintaining a temperature of 105 to 110°C.

### **Method:**

Approximately 2-3 g of the moist soil set aside from the liquid limit determination is kneaded with the fingers to an ellipsoidal shape. It is then rolled into a thread of uniform diameter throughout its length. The rolling is done on the ground - glass plate using either the fingers or the palm of the hand. When the diameter of the thread has been reduced to about three mm, the thread is rolled up into a

ball and again kneaded to an ellipsoidal shape and rolled out. This process is continued until the crumbling caused by drying out of the soil prevents the formation of a thread 3mm in diameter. This means that the crumbling occurs when the thread had a diameter slightly greater than 3 mm, but this is regarded as a satisfactory end point, provided that just prior to that, the soil had been rolled into a thread 3mm in diameter.

The operator should ensure that the thread is not broken up or crumbled by applying excessive pressure, but crumbles because of a lack of elasticity. The crumbled soil thread is transferred to a tared weighing bottle for the determination of its moisture content. A duplicate determination should then be carried out.

The masses of the containers with the soil samples are measured, after which the samples are oven dried to a constant mass. As a rule the material is dried overnight. When the containers are removed from the oven, the lids are replaced to prevent the absorption of moisture, and after allowing them to cool, their masses are obtained again. The loss in mass is the mass of water which is expressed as a percentage of the oven-dried mass of the soil. Duplicate determinations should not vary by more than 1.5 units.

Calculations:

Plastic Limit

The plastic limit is expressed as the percentage by mass of moisture contained in the oven dried soil and is calculated as follows:

$$\text{Plastic limit} = \frac{\text{Mass of wet soil} - \text{Mass of dried soil}}{\text{Mass of dried soil}} \times 100$$

The calculation should be carried out to the first decimal place, and the average of the two determinations is taken as the plastic limit.

### Plasticity index

The plasticity index is obtained by subtracting the plastic limit from the liquid limit.

### **A1.3 The determination of the linear shrinkage of the soil.**

#### Definition

The linear shrinkage of the soil for the moisture content equivalent to the liquid limit, is the decrease in one dimension, expressed as a percentage of the original dimension of the soil mass, when the moisture content is reduced from the liquid limit to oven - dried.

#### Apparatus

- (i) A shrinkage trough with inside dimensions of 150mm long x 10mm x 10mm and made of 16 gauge tinned copper or stainless steel.
- (ii) Mould grease.
- (iii) A spatula with a slightly flexible blade about a 100mm long and 20mm wide.
- (iv) A drying oven thermostatically controlled and capable of maintaining a temperature of 105 to 110°C.
- ( v ) A vernier caliper.

#### Method

##### Waxing the trough

It is generally recommended that the inside of the trough be waxed with a layer of paraffin wax before being filled in order to prevent adhesion between the soil and the trough. In practice this procedure is usually replaced by coating the inside of the trough with a layer of shutter oil or a thin layer of Vaseline.

### Filling the trough

The test should be run immediately after the liquid limit test so that the moist material left over can be used for filling the trough without further mixing. If for some reason it is required to carry out only the shrinkage test, about 40g of soil fines would first have to be mixed with water and brought to the liquid limit.

Fill one half of the wax trough with the soil at its liquid limit by taking small pieces of soil on the spatula and pressing the soil down against the one end of the trough and working along the trough until the whole side is filled and the soil forms a diagonal surface from the top of one side to the bottom of the opposite side.

The trough is now turned round and the other portion is filled in the same manner. The hollow along the top of the soil in the trough is now filled so that the soil is raised slightly above the sides of the trough. The excess material is removed by drawing the blade of the spatula longitudinally once only from one end of the trough to the other while exerting sufficient pressure with the forefinger on the blade of the spatula to ensure that the blade moves along the sides of the trough leaving it exactly full of the soil. During this operation the wet soil may fall away from the end of the trough, in which case it should be pushed back gently with the spatula.

### Drying the wet material

The trough with wet material is placed in a drying oven and dried at a temperature of between 105 and 110°C until all shrinkage has stopped. As a rule the material is dried out over night - though 3 hours should be sufficient time in the oven. The trough with material is taken out and allowed to cool in the air.

### Measuring the shrinkage

It will be found that the ends of the dried soil bar have a slight lip. These lips must be removed by abrading with a sharp narrow spatula, so that the end of the soil bar is parallel to the end of the trough. If the soil bar is curved, it should be pressed back into the trough with the fingertips so as to make the top surface as level as possible. The loose dust and sand removed from the ends, as well as loose material between cracks, should be emptied out of the trough by carefully inverting the trough whilst the material is being held in position with the fingers. The soil bar is then pressed tightly against the end of the trough. It will be noticed that the soil bar fits better at the one end than at the other end. The bar should be pressed tightly against the end at which there is a better fit. The distance between the other end of the soil bar and the respective end of the trough is measured by means of a vernier caliper.

### Calculation

The shrinkage is expressed as a percentage of the original wet length of 150mm that is calculated as follows:

$$\text{Linear shrinkage} = \frac{150 - \text{length of dry soil (mm)}}{150} \times 100$$

#### **A1.4 The determination of the grain size distribution by means of a sieve analysis.**

##### **Scope**

This method describes the determination of the particle sizes of a soil by means of shaking the soil through a nest of sieves. The nest consists of eleven sieves, each sieve with a mesh with smaller opening sizes than that of the preceding sieve.

##### **Apparatus**

- (i) A set of nesting sieves with square openings of the following sizes in mm:
  - 13.2
  - 9.5
  - 6.7
  - 4.750
  - 2.0
  - 0.850
  - 0.425
  - 0.250
  - 0.150
  - 0.075A pan and a lid
- (ii) Brushes for cleaning the sieves
- (iii) An electronic balance sensitive to 1g.
- (iv) A thermostatically controlled oven capable of maintaining a temperature of 100°C.
- (v) A galvanized pan 500mm in diameter.
- (vi) A mortar and rubber tipped pestle.
- (vii) A mechanical sieve shaker.

## Method

### Washing the soil

Make sure the sieves to be used are clean. The mass of each sieve should then be determined to 0.1g. Select approximately 500g of soil, oven dry the soil over night and determine its dry mass. Wash the soil, a little at a time through the 0.075 mm sieve being careful not to lose any of the soil retained on the sieve. Collect the washed material in the pan, then oven dry the soil retained on the sieves overnight. Determine the mass of the dry material to 0.1g.

### Sieving the soil

Break down the dry material into its individual particles using the rubber tipped pestle and mortar. Sieve the material through the nest of sieves by hand shaking, using a motion of horizontal rotations for 10 minutes or use a mechanical shaker for approximately half that time. Weigh each sieve with the material retained on it to an accuracy of 0.1g. The percentage passing is then calculated.

### Calculations

- (i) The mass of soil retained on each sieve is calculated as follows:

$$m = M - S$$

Where:

m = mass of soil retained in grams

M = mass of sieve and soil retained in grams

S = mass of sieve in grams

- (ii) The percentage passing is calculated using the following formula:

$$P = \frac{A - \sum m}{A} \times 100$$

Where:

P = the percentage passing or the percentage finer by mass

A = the mass of the pan and dry material before washing in grams

$\Sigma m$  = the cumulative sum of the mass of soil retained on the sieves in grams, starting from the sieve with the largest opening and working downwards.

The results can then be reported graphically with the particle size in mm on a logarithmic x - axis and the 'percentage finer' by mass on a linear y - axis.

**A1.4.1 The determination of the grain size distribution in the soil by means of a hydrometer.**

**Scope**

The distribution of particles larger than 0,075 is determined by sieving, whilst the distribution of particle sizes smaller than 0,075mm is determined by a sedimentation process, based on Stokes' Law defined hereunder, using a specially calibrated hydrometer. This method does not give absolute results, but gives data, which are comparable and consistent, if the method is followed in detail.

Stokes' Law:

Stokes' Law states that: Maximum grain diameter

$$d = \sqrt{\frac{300nL}{980(G - G1)T}}$$

where:

- n = viscosity of the suspending medium in Pascal - seconds.  
The viscosity of distilled water at 20°C is 0,00105 Pa.s.
- L = the distance in centimeters through which the grains settle in a period of time, T.
- T = time in minutes, period of sedimentation.
- G = relative density of soil particles
- G1 = relative density of suspending medium (which is 0,99823 for water at 20°C)

## Apparatus

- (i) A balance to weigh up to 200g, accurate to 0,1g.
- (ii) A canning jar, wide mouth, about 1000 ml capacity.
- (iii) A Bouyoucos cylinder graduated at 1130 and 1205 ml.
- (iv) A Bouyoucos hydrometer
- (v) A dispersing apparatus with paddle. The paddle is attached to a 120 to 150mm shaft, which is rotated (by an electric motor), at a rate of 1450 revolutions per minute.  
If such a disperser is not available an egg whisk with 4 vanes may be used.
- (vi) A tablespoon.
- (vii) A stop-watch.
- (viii) A wash-bottle  
A dish or small basin approximately 150mm diameter.  
A thermometer, measuring from 0 to 50°C, graduated in 0,2° C.
- (ix) A pipette with a 10 ml capacity.
- (x) Calgon solution.

## Method

### Preparation of sample

100g, accurate to 0.1g (50g for samples containing a reasonably high percentage of the silt and clay fractions) of the soil fines (finer than 0,425mm) is weighed out. The weighed sample is transferred to a canning jar, and about 400ml of distilled water and 10ml of Calgon solution is added. The soil-water mixture is stirred well with a glass rod and left to stand for at least two hours but preferably overnight.

After the mixture has been allowed to stand, it is dispersed for 15 minutes with the standard dispersing paddle or for five minutes with the egg whisk. The paddle

is washed clean with distilled water, allowing the wash water to run into the container with the suspension.

#### Filling the cylinder and correcting the temperature

The suspension is poured into the Bouyoucos cylinder and the canning jar is rinsed with distilled water from the wash bottle. All the suspension is transferred quantitatively to the cylinder. The cylinder is then filled with distilled water to the 1000 ml mark.

#### Hydrometer readings

The cylinder is inverted a few times, using the palm of one hand as a stopper over the mouth of the cylinder, until a homogenous suspension is obtained. It is placed on a tabletop and the stopwatch started. At about 10 seconds the hydrometer is inserted and a reading is taken at 18 seconds to give a percentage of the sub 0.075mm particles (the fine sand, silt and clay fraction). A reading at 40 seconds gives the percentage of sub 0.05mm material (silt and clay fraction). For the clay only fraction (sub 0.002 mm) and colloid fraction (sub 0.001 mm) readings after 7 hours and 24 hours respectively may be taken. The hydrometer is removed from the cylinder after each reading and kept in a cylinder of distilled water. Temperature readings of the water solution should be taken after every hydrometer reading.

#### Calculations

The calculation to determine the particle size is fairly lengthy but programs do exist which can be used to calculate the particle sizes. Alternative descriptions of the calculations can be found. [26]

## **A.2. LABORATORY TESTS CONDUCTED TO DETERMINE CHANGES IN THE PHYSICAL PROPERTIES AND MINERALOGY OF THE PANFONTEIN SOIL AS A RESULT OF THE ADDITION OF LIME**

### **A.2.1 THE INITIAL CONSUMPTION OF LIME TEST**

#### **Objective**

The objective of the ICL test is the control of the pH in lime and cement stabilised soil in order to allow the possible formation of cement minerals, calcium silicate hydrate and for proper modification to take place.

#### **Equipment**

1. Balance accurate to 0.1g.
2. pH meter accurate to 0.02 units
3. Six plastic beakers (150ml, tall)
4. Spatula or pallet knife
5. Soft tissues
6. Glass beaker (200ml)
7. Distilled water.
8. Lime to be used: This lime should be the poorest quality lime meeting SABS 824. If a high quality of lime is used in the test and a low quality lime is used in the field the results which were achieved in the laboratory will not be achieved in the field. Whereas if a low quality lime is used in the laboratory, the use of a high quality lime in the field will not yield negative results.
9. Jet bottle for cleaning electrode
10. Drying oven capable of maintaining a temperature between 105 - 110°C.
11. Thermometer accurate to 0.5°C
12. Sample riffler or splitter (25mm openings approximately are sufficient).

### **Preparation of soil samples**

- a) The test is carried out on 2 kg of material finer than 19mm. Any oversize material is crushed to pass 19mm.
- b) Oven dry sample at 105 - 110°C until constant mass is reached.
- c) The sample is further reduced to 300g quantities by means of sample splitters and placed into 150 ml (or larger) plastic containers.

### **Testing procedure**

- a) Since most materials require between 2 and 5 percent lime [2], it is advisable to set up six beakers with lime percentages of 1; 2; 3; 4; 5 and 6 of the dry soil mass, i.e. 200g soil + 2; 4; 6; 8; 10 and 12g of  $\text{Ca}(\text{OH})_2$ . This will ensure, in most cases, that the percentage of lime required can be determined in one hour. Measure out the lime to the nearest 0.1g and add it to the soil. Mix soil and dry lime.
- b) Slightly over-saturate samples with distilled water. The material shall be judged over-saturated when the pores of the material are water filled and free water can be observed on the surface of the mix. The surface particles need not be submerged completely.
- c) Mix for 30 seconds every 10 minutes.
- d) After one hour, measure the pH by inserting the pH electrode gently into a hole made in the material with the spatula to a depth of about 20 – 30 mm and gently covering this part of the electrode with the material. Tap the beaker gently for contact between electrode and material. The pH meter must be equipped with an electrode standardised with a buffer solution having a pH of 12.00.
- e) Record the pH of each of the soil/lime/water mixtures. The lowest percentage lime which causes the pH to remain constant, is the saturation lime content of this particular gravel. It should be noted that as pH is temperature dependent the actual saturation pH value will not be constant but at normal temperatures, it will be of the order of 12.4.

### **Recording of results**

The results are recorded to the nearest 0.5 percent lime required to produce the maximum pH in the soil/lime/water mixtures as indicated by the pH of the mix. Since the pH depends on the temperature, this is also recorded. This percentage of lime is read as the ICL of the soil.

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