STABILISATION OF CLAY SUBGRADE SOILS USING GROUND GRANULATED BLASTFURNACE SLAG

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The candidate confirms that the work submitted is his own and that appropriate credit has been given where reference has been made to the work of others

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ABSTRACT

Roads constructed on expansive clays may be adversely affected by the behaviour of the clay. Expansive clays suffer volume change due to changes in moisture content which causes heaving, cracking and the break up of the road pavement. Stabilisation of these types of soil is necessary to suppress swelling and increase the strength of the soil and thus partially decrease the thickness of road pavement layers.

The use of by-product materials for stabilisation has environmental and economic benefits. Ground granulated blastfurnace slag (GGBS), a by-product material in Egypt, and lime are used in the current work to stabilise samples of a clay soil similar to a typical Egyptian clay soil. This test soil comprises 80% River Aire soil and 20% calcium montmorillonite. The main objectives of this research were to investigate the effect of GGBS, with and without lime, on the engineering behaviour (plasticity characteristics, compaction, unconfined compressive strength (UCS) and swelling potential) of the test soil and to identify the reaction products of the stabilised materials to determine the mechanisms by which changes in engineering properties are obtained.

In order to achieve these objectives, extensive laboratory investigations were carried out. Various mixes (up to 10% GGBS by dry weight of the test soil and up to 30% replacement by hydrated lime) were prepared and cured under two representative conditions {20°C with 90-100% relative humidity (CC1) and 35° C with 50-60% relative humidity (CC2)} for up to 12 months. Compaction and plasticity were measured soon after mixing, the swelling potential and UCS were measured after longer curing periods.

Four analytical techniques {X ray diffraction, scanning electron microscopy, differential thermal analysis and nuclear magnetic resonance (NMR)} were used to identify the reaction products of the clay fraction of the test soil mixed with various amount of GGBS and lime. This pure clay test soil was used to ease identification of the reaction products.

The investigations showed that generally the engineering properties (UCS, swelling, plasticity) improved with the addition of GGBS and with increasing curing period and temperature. The addition of lime resulted in a dramatic improvement within the test ranges covered in the programme. The maximum dry density, MDD, decreased and the optimum moisture content, OMC, increased with increasing GGBS and lime content.

The major changes in the UCS and swelling behaviour are due to the formation of new cementitious materials. The analytical investigation confirmed two major reactions when GGBS and lime were added to the pure clay soil, hydration of GGBS activated by lime to produce calcium aluminosilicate hydrate gel (C-A-S-H) and hydrotalcite type phase, and the clay–lime reaction to produce calcium silicate hydrate (C-S-H), (C-A-H) and (C-A-S-H). The NMR test results revealed that the aluminosilicate chain length ($\overline{CL}$), the aluminium: silicate (Al/Si) ratio and the amount of Si in the formed C-S-H significantly increased with an increase in the curing temperature and period, which indicates a more stable and well crystalline C-S-H.

The results indicate that the use of GGBS alone, or preferably with lime, could have a significant effect on the behaviour of potentially swelling clays.

Recommendations for further studies include a study of the effect of cyclic loading on the test soil. Also, site trials should be carried out to assess the suitability of using these materials in the field

LIST OF ABBREVIATIONS AND SYMBOLS

Å	Angstrom (10^{-10} m)
°C	degree Celsius
$\overline{CL}$	Aluminosilicate Chain Length of C-S-H
c	Centi (10^{-2} m)
d	Lattice spacing
n	Order of diffraction
DTA	Differential Thermal Analysis
g	Gramme
GGBS	Ground Granulated Blastfurnace Slag
E_{40}	Secant modulus
kg	Kilogram
k	Kilo (10^{3})
l	Litre
m	Milli (10^{-3})
mm	Millimetre
µ	Micro (10^{-6})
µm	Micron or micrometer (10^{-6})
N	Newton
PPM	Parts per million
t	Time in seconds
AASHO	American Association of State Highway Officials
ASTM	American Society of Testing And Materials
BB1	Berk Bond Number 1
CBR	California bearing ratio
CC1	Curing Condition (20° C and 90-100 % relative humidity)
CC2	Curing Condition (35° C and 50-60 % relative humidity)
C-A-H	Calcium aluminate hydrate
C-A-S-H	Calcium aluminosilicate hydrate
C-S-H	Calcium silicate hydrate
I p	Inner Product
TGA	Thermogravimetry Analysis
LL	Liquid limit
L.O.I	Loss on Ignition
MAS	Magic Angle Spinning
MDD	Maximum Dry Density
NMR	Nuclear Magnetic Resonance
Op	Outer Product
OPC	Ordinary Portland Cement
OMC	Optimum Moisture Content
PL	Plastic Limit
PI	Plasticity Index
pH	Log $_{10}$ (H^{+} concentration)
SEM	Scanning Electron Microscopy

T	Temperature
UCS	Unconfined Compressive Strength
XRD	X ray Diffraction
XRF	X ray Fluorescence
φ	Angle of internal friction
θ	Angle of Diffraction
λ	(Lambda) wavelength

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CHAPTER ONE

INTRODUCTION

1.1 GENERAL INTRODUCTION

Economic development of any country is controlled to a great extent by the highway and airport networks. This is becoming particularly apparent in the developing countries, where tremendous lengths of roads need to be constructed in order to facilitate the development of agriculture, commerce and industry. The cost of any road pavement project includes initial costs and subsequent maintenance costs. The initial costs include many items such as land, accommodation works, bridges and subways, drainage, pavement construction etc. The type and the thickness of the pavement construction determines, a large percentage of the initial cost of any road project. Therefore, the development and use of methods to decrease the cost of pavement construction is very beneficial. It is essential to take into consideration the conditions of the subgrade soil before designing the type and the thickness of the pavement, as the subgrade carries the traffic loads as well as the pavement loads (Bari, 1995).

The major function of the pavement is to reduce stresses in the subgrade so that there is little or no deformation in the subgrade. Therefore, the more the subgrade is resistant to deformation the thinner the pavement will be, thus reducing the construction cost of the road.

Good quality subgrade soils are preferable for durable roads but are not always available for highway construction. The highway engineer designing a road pavement may be faced by weak or unsuitable subgrade. In this case the following methods to overcome this problem can be considered. Firstly, improve the in-situ materials by normal compaction methods and design for the modified properties. Secondly, import suitable materials from the nearest convenient source and replace the site materials. Thirdly, improve the properties of the existing materials by incorporating some other materials; this process is known as "soil stabilisation" (Ingles and Metcalf, 1972).

The most appropriate method will usually be determined by economic considerations, for example it may be cheaper to stabilise a soil using relatively expensive additives rather than excavate and dispose of unsuitable materials and import and place suitable fill, as well as the properties of the subgrade.

1.2 SOIL STABILISATION

Soil stabilisation, in its general meaning, considers every physical, physico-chemical and chemical method employed to make a soil suitable for its required engineering purpose (Abdelkader, 1981). In its specific meaning in road engineering, soil stabilisation is a regulated process to improve the soil by using additives in order to use it as base or sub base courses and carry the expected traffic and pavement loads.

There are several methods by which soils can be stabilised. The three basic techniques that have been successfully adopted in road construction are physical, physico-chemical (bituminous), and chemical stabilisation. Physical stabilisation is primarily concerned with the application of external energy. In some cases it is called mechanical stabilisation. Physical stabilisation with the incorporation of another material to give a well-graded mixture may result in materials suitable for use as pavement material. Physico-chemical stabilisation, which in the literature generally means the addition of bitumen, is a process whereby an additive is incorporated into the system. Bitumen acts as a cohesive agent in granular soil and in cohesive soil it acts as a waterproofing agent. Chemical stabilisation involves the addition of additives to the original soil to form new cementitious materials which result in the soil having better engineering properties (less swelling, better plasticity and workability and better strength). This process is primarily employed with fine grained soils such as silt and clay (Abdelkader, 1981; Ahmed, 1988).

Many chemical substances have been used to stabilise soils, e. g. lime, cement, calcium chloride, sodium chloride, various silicate compounds, and, recently, ground granulated blast furnace slag (GGBS) in South Africa and in the United Kingdom (Higgins *et al.*, 1998).

A particular problem in many areas of the world (e.g. U.S.A, India, and Egypt), is where expansive clays form the subgrade. Expansive clays are those which suffer volume and behaviour changes with changes of water content which results in the break-up of road pavements and damage to light structures. Replacement of such clays by other materials is generally expensive due to the high costs of excavation and disposal of unsuitable materials and the import and placing suitable fill. This is a particular problem in developing countries where construction costs are critical.

The typical cross section of an Egyptian road pavement is 200 to 300 mm of granular base courses made of compacted well graded natural pit-run gravel or crushed stone, covered by one or more layer of bituminous concrete courses (Abdelkader, 1981). This pavement system is not generally suitable for subgrade of expansive clays due to the need to import granular materials this compounding the problem of construction on expansive soils.

The volume change of expansive clays, due to a change in their moisture content, causes upward movement which is difficult to predict, resulting in heaving, cracking and the break up of the road pavement which are founded on such soils (Mowafy *et al.*, 1990). Furthermore, most types of clay soils require a greater thickness of base layer compared to those built on suitable and strong subgrade (sand and gravel) that result in a very high increase in the initial and total expenditures of such projects. To suppress swelling and reduce the volume change, to increase the strength of the expansive clay soils and thus decrease the thickness of the sub-base or base layer, stabilisation of these types of soil is necessary.

Lime and cement are the two common additives which have been employed in stabilisation of Egyptian clayey subgrades to produce a base or sub base layer instead of importing granular base course materials. Lime stabilisation is preferred to cement stabilisation because lime is often cheaper than cement as it is produced locally and lime can also improve the workability of clay. Lime stabilisation requires adequate clay content and a relatively high curing temperature so it is also more suitable than cement in tropical and sup-tropical countries. Furthermore, cement hydration may be retarded by montmorillonite which is the predominant clay mineral in expansive clays.

Although lime is widely used, cement is still very useful as a soil stabiliser when the clay content is low and/or the temperature is not high enough. However, due to the gradual depletion of conventional construction materials and shortages in cement production, the need to develop an alternate binder to cement for use in soil stabilisation has become necessary. Researchers have considered the use of some by-product materials as alternative materials to cement and also to lime. The use of by-products has environmental and economic benefits. These materials should satisfy the engineering requirements (suppress swelling potential, improve plasticity characteristics and enhance stress-strain behaviour) and be affordable.

Granulated blast furnace slag (GBS), which is produced as a by-product in the manufacture of pig-iron, has been suggested as a binder (Higgins, 1998 a). This material is produced in huge amounts by the Egyptian iron and steel company and it is comparatively cheap. It is mainly used, after being ground to fine ground granulated blastfurnace slag (GGBS), to produce blended cement. However, it has not been used as a soil stabiliser agent in Egypt although it has been used in the U. K and South Africa. GGBS on its own has only mild cementitious properties and it is generally used in combination with Portland cement or hydrated lime (calcium hydroxide) which provides the necessary alkali for activation (Richardson and Groves, 1992; Higgins, 1998 a).

This dissertation describes an investigates of the effect of GGBS alone, and GGBS activated by lime as an alkali activator, on the engineering properties of a test soil which represents similar Egyptian clayey soil in order to access its suitability for use in Egypt. The predominant clay minerals in this test soil are montmorillonite, kaolinite and small amounts of illite, chlorite and mica. The selection of these materials and design of the test soil are described later in this dissertation.

1.3 STRUCTURE OF THE DISSERTATION

The structure and mineralogy of soils in general is described in chapter 2, with emphasis on montmorillonite and kaolinite and clay-lime reactions. Slag stabilisation and the effects of GGBS on the engineering properties of soils are outlined in chapter 3, while in chapter 4 the scope, aims and objectives of this investigation are presented. In chapter 5 the properties of materials used in the research are discussed while chapter 6 contains a description of the test equipment, instrumentation and test procedures. Chapters 7 and 8 report the results of the engineering and analytical tests respectively, and these are discussed further in chapter 9. Chapter 10 contains the conclusions drawn from the investigation, together with recommendations for further research.

The starting point of the current work will be a review of the mineralogy and general properties of clay soils, the problems of expansive soils, stabilisation of clay soils using lime and the effect of the addition of lime on the engineering properties of clay soils.

CHAPTER TWO

LITERATURE REVIEW

2.1 INTRODUCTION

The engineering properties of sub-grade soils including plasticity characteristics, compaction properties, volume stability and strength may be enhanced by adding materials such as lime, cement, sodium chloride and GGBS. The changes in properties of the soils primarily depend upon the type and amount of binder, curing conditions and time, organic matter content and the percentage of clay.

This chapter includes a literature review of the structure of soils and clay minerals, especially montmorillonite and kaolinite, as they are the predominant clay minerals in Egypt. The problems of expansive clay are also discussed together with soil stabilisation in general. Special consideration is given to lime stabilisation, including the general soil-lime reactions, effect of lime on compaction characteristics, plasticity, volume stability and strength. The use of slag in the stabilisation of soils is considered in chapter 3.

The changes which occur in clay soil when lime is added can be divided into two categories, modification and stabilisation. During modification calcium ions are adsorbed by clay particles in cation exchange reactions. This process starts immediately and it changes the plasticity of the clay without the formation of any new cementitious materials. In the stabilisation process calcium ions attack the clay minerals due to chemical reactions between the clay minerals and calcium hydroxide, and new materials are formed, mainly calcium silicate hydrate, calcium aluminate hydrate and calcium aluminate silicate hydrate (Bell, 1996). The total lime content required for modification (change in plasticity) is in the range 1-3% by dry weight of soil, while that required for both modification and stabilisation is in the range 3-8% by dry weight depending primarily on the clay fraction of soil under investigation and also on the type of the clay minerals (Ingles and Metcalf, 1972; Bell, 1988 a; Diamond and Kinter, 1964). These aspects are considered in detail below.

The structure of soils and clay minerals is critical in an understanding of the process of soil stabilisation generally and clay-lime reaction in particular and this is considered below.

2.2 THE COMPOSITION OF SOILS AND CLAY MINERALS

2.2.1 SOIL COMPOSITION

Soils consist of solids, water and air, and the forces of interaction between these constituents and their spatial arrangements govern their behaviour to a large extent. The most important phase of the structure is the solid phase because it governs the plasticity characteristics, volume change due to moisture content changes and many of the other engineering properties of soils. The solid phase may be composed of coarse-grained particles and/or fine-grained particles. The civil engineer divides the materials at the earth's crust into two categories: (1) rock and (2) soils (Grim, 1968). Terzaghi and Peck (1948) defined soil as "a natural aggregate of mineral grains that can be separated by such gentle means as agitation in water". The engineer considers any natural loose material at the earth's crust regardless of particle size distribution, composition or organic matter as a soil.

2.2.2 CLAY MINERALS

2.2.2.1 Introduction

Clay minerals play an important role in industry and therefore, a tremendous amount of research into clay systems has been carried out. Clays are used in many industrial products and processes e.g. in the ceramics flooring industry and for building blocks

(Mateos, 1964; Van Olphen, 1963). Moreover, they are of special interest to the highway engineer and the civil engineer in general. The type of clay mineral present in a soil is important as it governs many properties such as reactivity, plasticity and volume stability. Other non-clay mineral materials may be present in clay soils. These include quartz, calcite, dolomite, mica and feldspar. The presence of these materials may be detected by particle size distribution analysis, as the non clay minerals tend to occur as particles coarser than 2 μm (Grim, 1968).

Organic materials sometimes occur in clay soils and can be of many different forms. They may be present as leaf matter or discrete particles of wood, as organic molecules adsorbed on the surface of the clay mineral particles or adsorbed between the silicate layers. Wood particles range from large chunks to particles of colloidal size. These tiny-sized particles of wood may give a dark-grey or black colour to the material. A very small amount of organic material present may have a very large pigmenting effect, and organic matters can have a major effect on the behaviour of soils (Kinuthia, 1997).

2.2.2.2 Definitions

Clays can be defined as "natural earthy, fine-grained materials which develop plasticity when mixed with water". Plasticity is a key characteristic of clays. Generally, clays are composed of silica, alumina and water with small quantities of iron and alkalies. There are, however, some materials called clays which do not satisfy all the clay specifications. Thus so-called "flint clays" have no plasticity when mixed with water. They have the other characteristics of clay (Grim, 1953). In civil engineering the maximum size of clay particles is defined as 2 μm. The fundamental reason for placing the upper limit of the clay size fraction at 2 μm is that the lower size limit of non-clay minerals is generally 2 μm (Grim, 1953). The clay fraction is the percentage of particles under 2 μm diameter related to the whole amount of soil.

2.2.2.3 Structure of clay minerals

There are two main units which make up the atomic lattices of most clay minerals; these are the tetrahedral silica unit and the octahedral unit of aluminium or magnesium (Grim, 1968). The tetrahedral silicon unit consists of a silicon atom equidistant from four oxygen atoms or hydroxyl ions if required to balance the electrical charge on the structure. Silicate tetrahedral groups are normally arranged to form a hexagonal network (Brown, 1984), which is repeated to form a sheet with the typical composition $Si_4O_6(OH)_4$, figure 2.1. The tetrahedra are arranged so that all the points are aligned and their bases are in the same plane.

The other unit takes the form of an octahedral crystal in which an aluminium, magnesium or iron atom occupies the centre of the structure enclosed by six hydroxyls, figure 2.2. When aluminium is present only two thirds of the possible cationic positions are filled to electro-charge balance the structure, and this form is called the gibbsite with the formula $Al_2\,(OH)_6$. When only magnesium is present all the possible positions are filled. This gives the brucite structure and this clay mineral may be described by the formula $Mg_3\,(OH)_6$ (Grim, 1968; Berman, 1963).

Most of the common clay minerals are composed of these two structural layers stacked in various forms. The order in which these layers are stacked greatly influences the physical and chemical characteristics of a clay mineral. Some clay minerals are fibrous and consist of different structural units from those mentioned above (Van Olphen, 1964 and Grim, 1953). This basic structural unit is composed of silica tetrahedrons arranged in a double chain as shown in figure 2.3. The structure is similar to that of the silica tetrahedrons sheet except that it is continuous in one direction while in the other direction it is restricted to a width of about 11.5 Å.

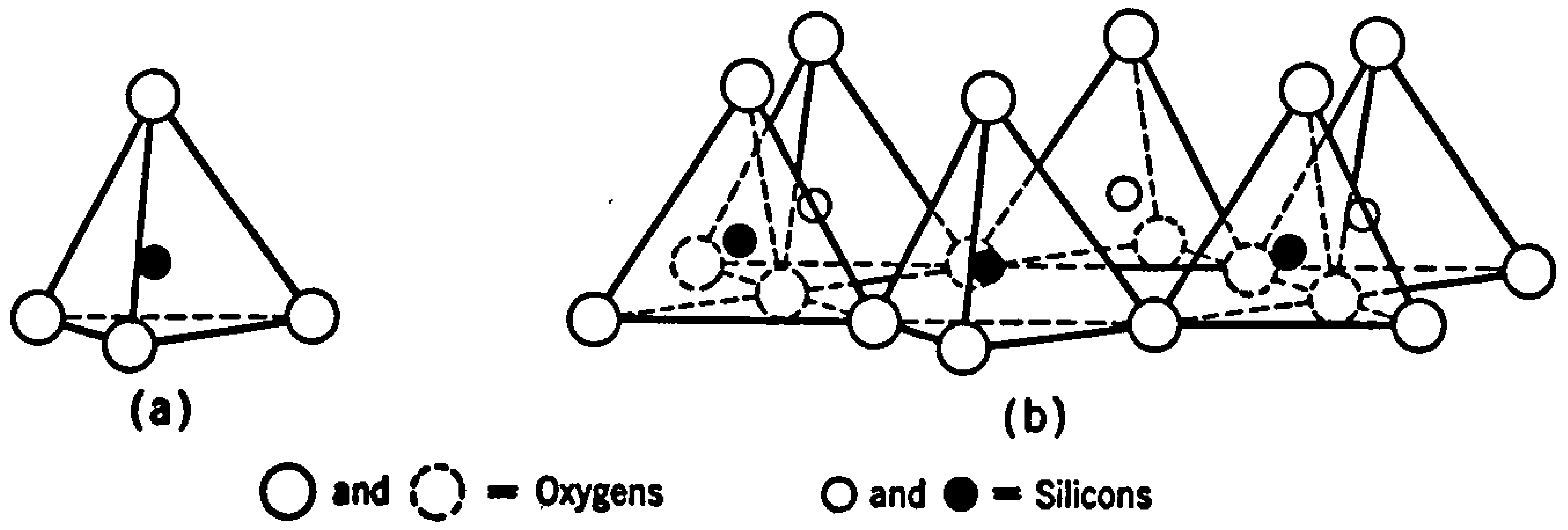

Figure 2.1 Diagrammatic sketch showing (a) single silica tetrahedron unit and (b) the sheet structure of the tetrahedrons arranged in a hexagonal network, Grim, 1962.

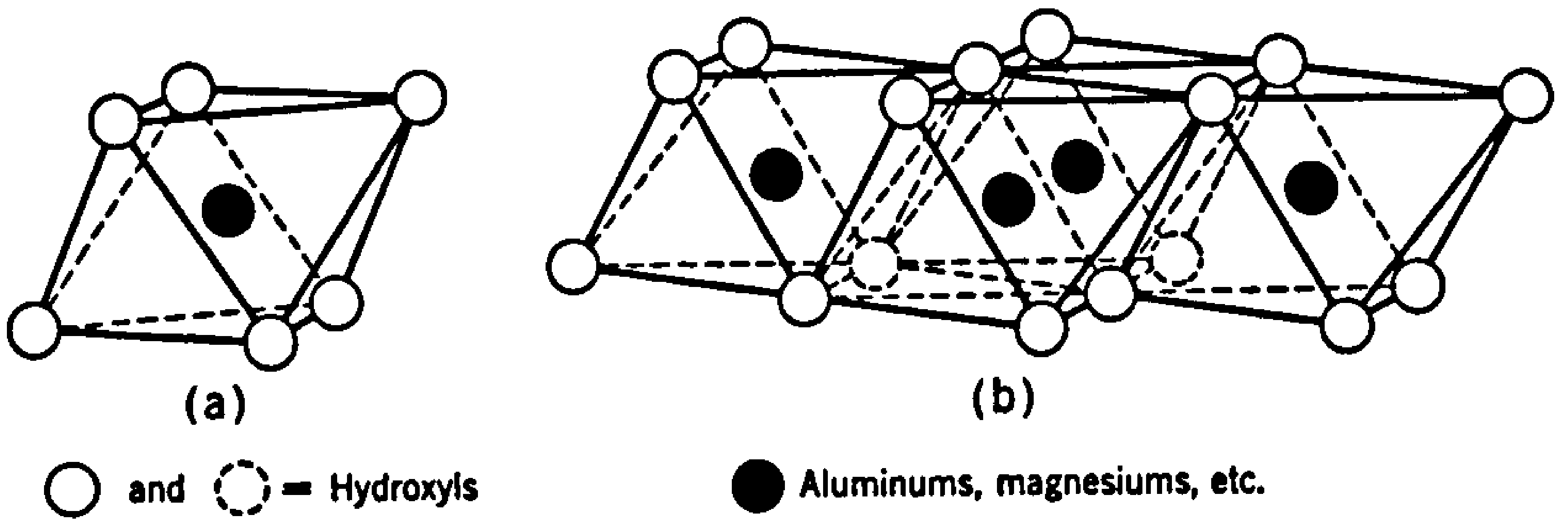

Figure 2.2 Diagrammatic sketch showing (a) single octahedral unit and (b) the sheet structure of the octahedral units, Grim, 1962.

The common clay minerals encountered in engineering practice are montmorillonite, kaolinite and illite (Grim, 1962). Also, the predominant clay types in Egypt are montmorillonite and kaolinite with minor amount of illite, most of the literature is related to these three types (Fayed, 1970; Fayed and Hassan, 1970; Wahdan and Abdel-Aal, 1977; Naga *et al.,* 1981).

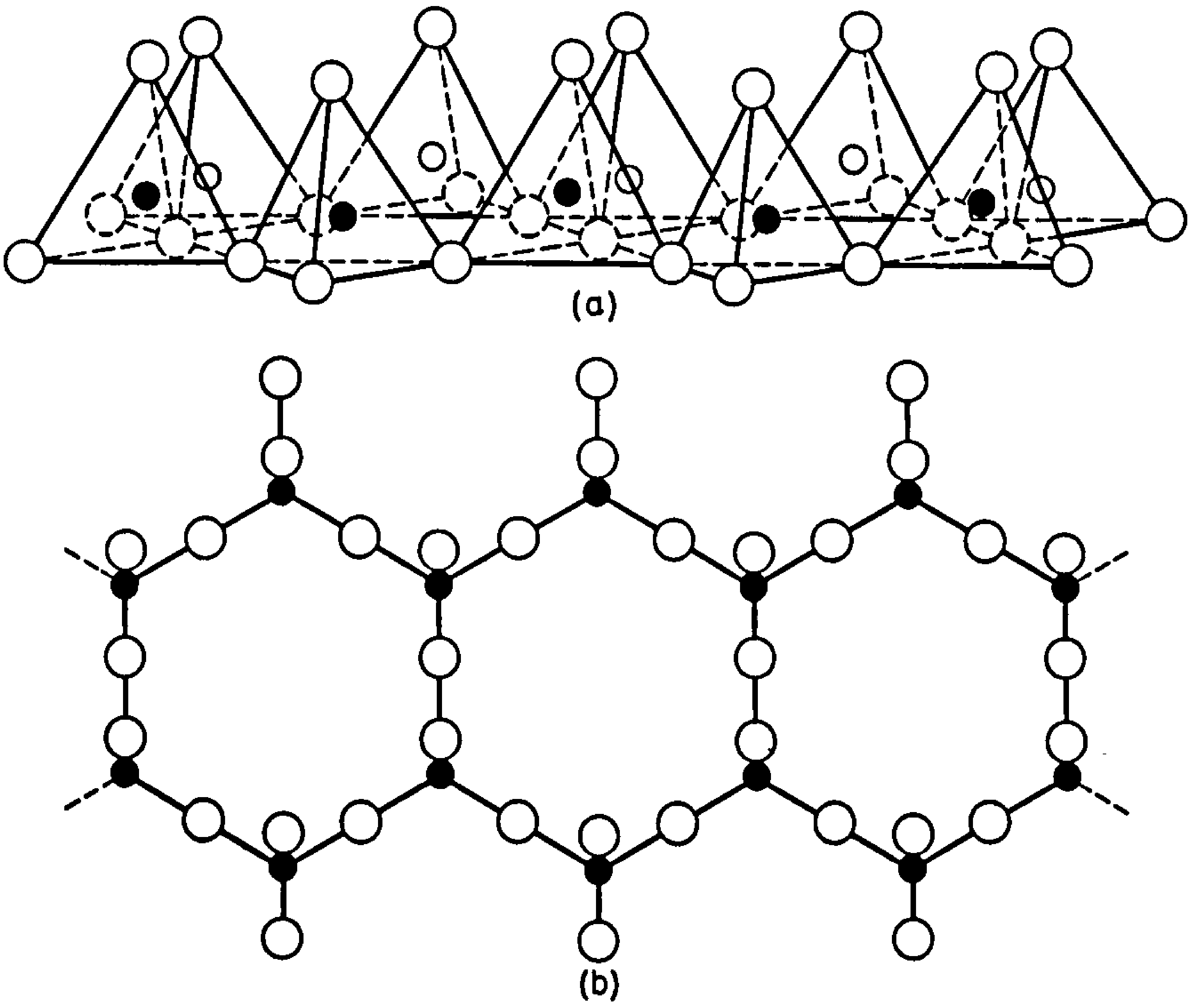

Figure 2.3 Diagrammatic sketch of double chains of silica tetrahedrons, as in the amphibole structural type of clay minerals: (a) in perspective, (b) projected on the plane of the base of the tetrahedrons, Grim, 1962.

A) Montomorillonite

Montmorillonites are formed from the weathering of volcanic ash under poor drainage conditions. They have a high shrinkage and swelling potential which can be several times their dry volume. Montomorillonite has a high liquid limit and high activity (Young and Warkentin, 1966). It is a three-layer mineral; its basic structural unit is a gibbsite sheet $Al_2(OH)_6$ sandwiched between two silicate sheets, figure 2.4.

These units are stacked one above the other. The bonds between successive crystal units are comparatively weak. The strength of the bonds is dependent upon the exchangeable cations involved. Water molecules and other cations can enter between the sheets causing them to move apart and the particles to expand and the mineral may be split up into its unit layers. The particle size of montomorillonite is small. It has a specific surface area of 800 m^2/g and because of the considerable area of the charged surfaces, montmorillonite exhibits high plasticity, cohesion, swelling and shrinkage characteristics depending upon the nature of the exchangeable cations present.

The montmorillonite formula as listed by Grim (1968), is $(OH)_4Si_8Al_4O_{20}.nH_2O$, and the composition is approximately 66.7 % SiO_2, 28.3 % Al_2O_3, 5% H_2O. In the silicate tetrahedral sheet aluminium can partly replace the silicon, and magnesium can replace aluminium. Iron, zinc, lithium, and other atoms can also replace aluminium. This replacement is often referred to as isomorphous substitution which is considered to be a prime factor which influences the "Cation Exchange Capacity", see section 2.5 (Berman, 1963). Soils containing large proportions of montmorillonite are poor foundation materials, because they have the tendency to absorb large amounts of water and show a large volume change between the wet and dry seasons (Mitchell, 1976).

B) Kaolinite

The structure of kaolinite is a single silicate tetrahedral sheet and a single alumina octahedral sheet combined in a unit, figure 2.5. The kaolinite mineral is a stacking of such layers with hydrogen bonding between the hydroxyls of the alumina sheet and the oxygens of the silicate sheet (Grim, 1968). Due to this strong bond, the kaolinite crystal typically consists of about 100 individual kaolinite layers stacked together and these are difficult to dissociate (Elsekelly, 1987).

Kaolinite has the largest crystals of all the clay minerals and the smallest specific area, ($15m^2/g$). Penetration of water molecules and ions between the layers is difficult because of the strong hydrogen bonding. Therefore the lattice is considered non-expanding, and because of this the surface area to which the water molecules can be attracted is restricted to the outer face. Therefore, the plasticity of kaolinite is very low compared to other type of silicate clays.

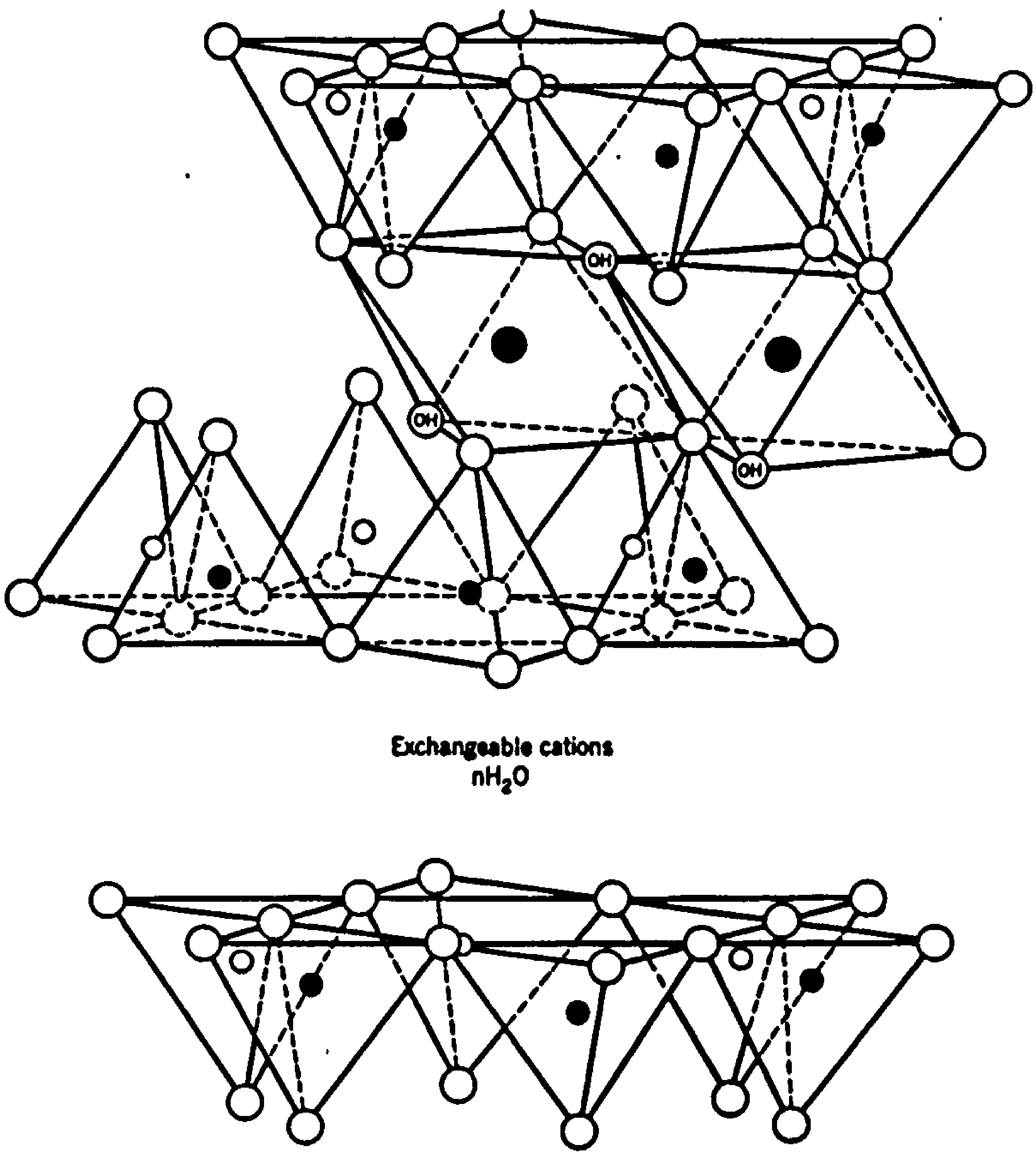

Figure 2.4 Diagrammatic sketch of the structure of montmorillonite, Grim 1953

The structural formula of kaolinite is $(OH)_8Si_4Al_4O_{10}$. This mineral is often referred to as having a 1:1 lattice. The theoretical composition of kaolinite, is approximately 46.54% Si_2, 39.50% Al_2O_3 and 13.96 % H_2O, The interlayer distance of each unit cell is 7.2Å. Compared to other clay minerals, the degree of perfection of the crystal is high and the amount of isomorphous substitution is low. In general kaolinite may be considered to be a well- crystallised clay mineral with relatively little physico-chemical activity (Ross and Kerr, 1932; Grim, 1962).

Kaolinite minerals are characterised by their relatively low liquid limit and activity (Young and Warkentin, 1966; Dennon and More, 1986). The edges of the kaolinite plates are positively charged in a low pH environment, and change to being negatively charged in a high pH environment. The superposition of the oxygen and hydroxyl planes in adjacent units causes the units to be held together by hydrogen bonding between the layers (Grim, 1962). This bond is strong and prevents water absorption between the layers and consequently they are considered very stable from an engineering point of view. Therefore, kaolinite does not have the same degree of swelling, plasticity, cohesion, or shrinkage as montmorillonite.

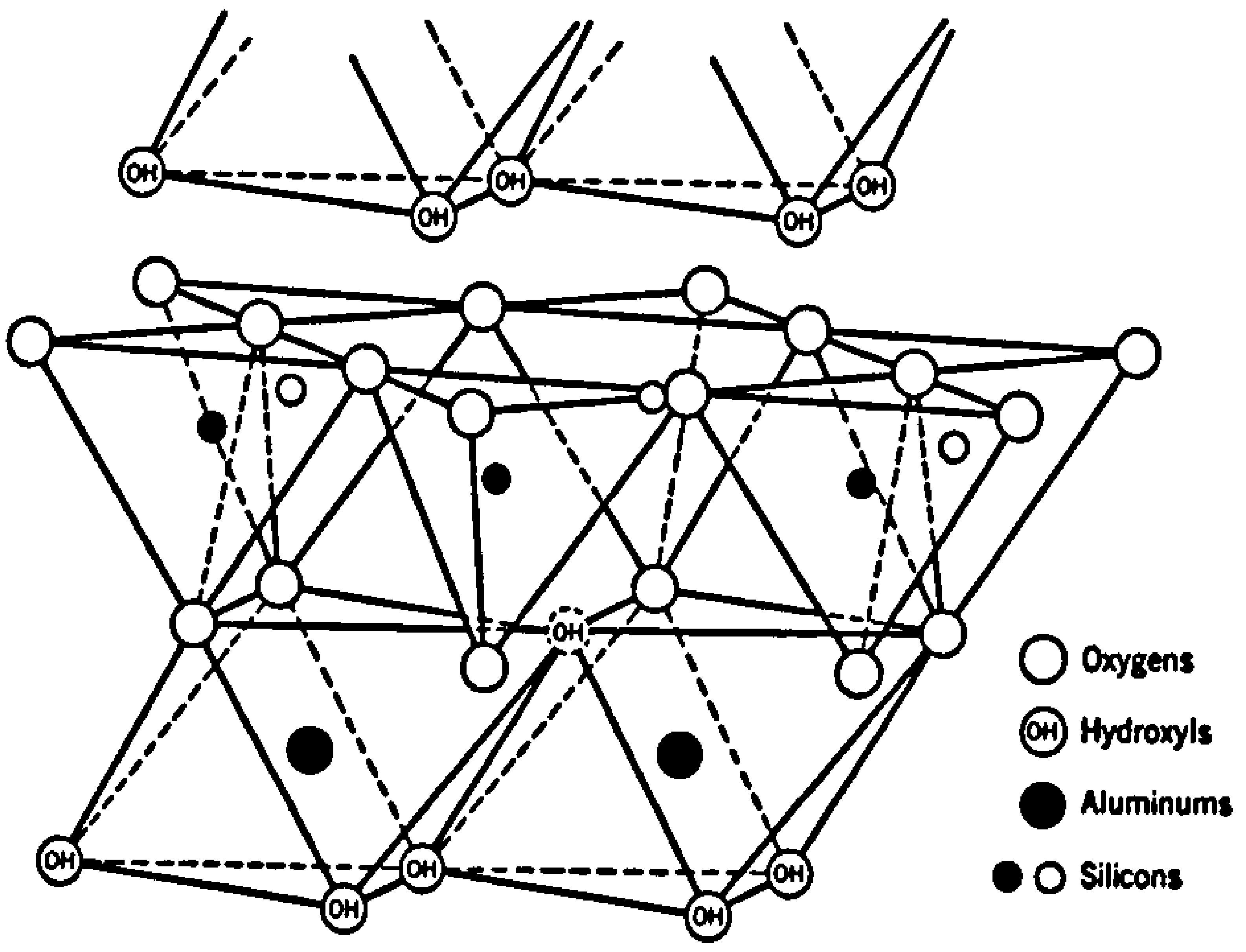

Figure 2.5 Diagrammatic sketch of the structure of kaolinite, Gruner, 1932, after Grim, 1962.

C) Illite

The structure of illite is very similar to that of montmorillonite. The distinct difference is that the individual layers are joined together by potassium ions, figure 2.6. This is a particularly stable system since the potassium ions are just small enough to fit perfectly within the hexagonal space formed by the oxygen ions on the surfaces of the silicate sheets. The specific surface, and consequently the surface activity, are smaller than for montmorillonite. The activity of illite can be considered intermediate between of kaolinite and montmorillonite (Grim, 1962).

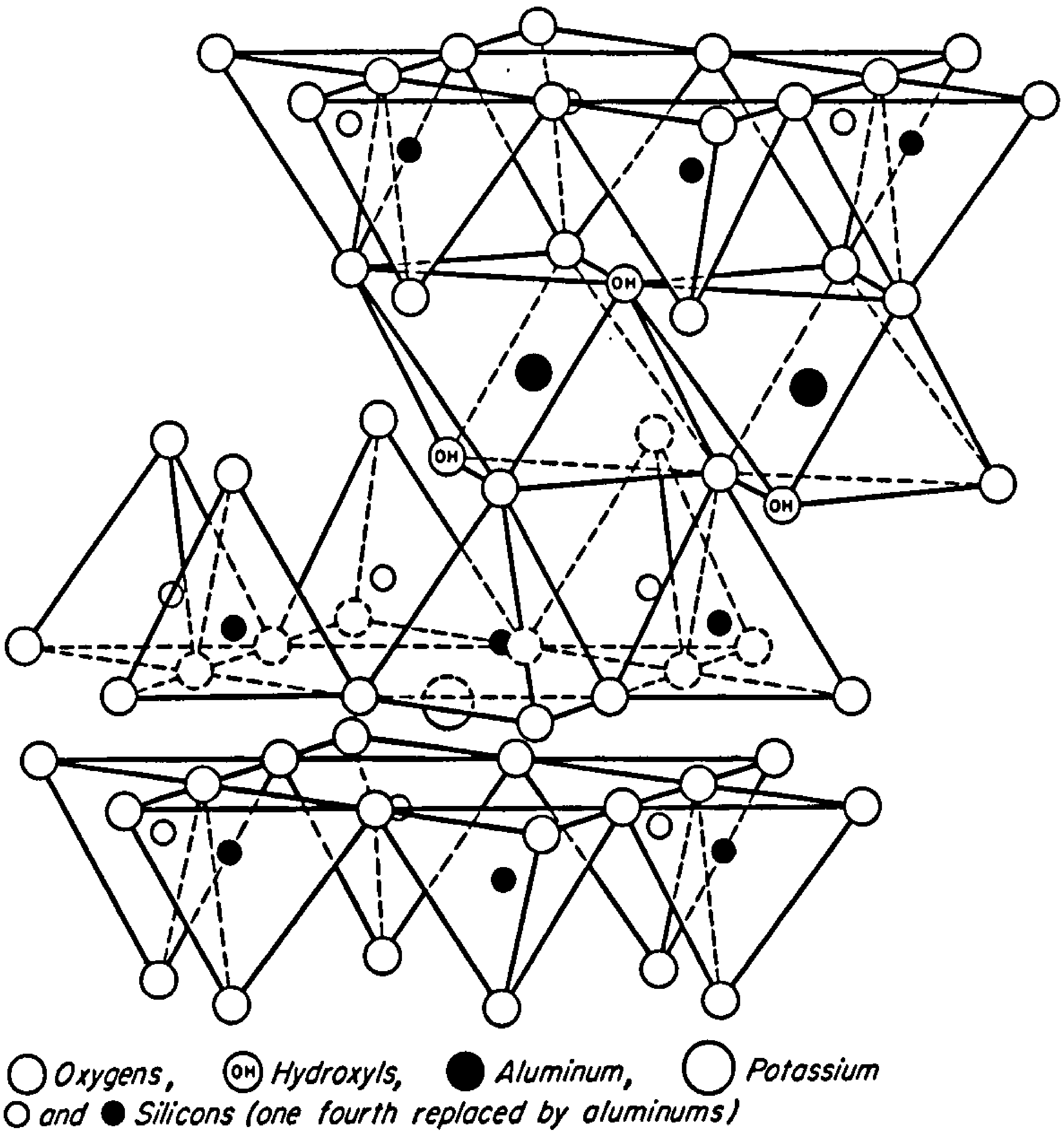

Figure 2.6 Diagrammatic sketch of the structure of illite, Jackson and West 1930, after Grim, 1962.

2.3 WATER ADSORPTION AT CLAY SURFACES

Clay particles in soils are almost always hydrated, i.e. surrounded by layers of water molecules adsorbed onto the clay particles. When the behaviour of clay soils is considered, these water molecules are considered as a part of the clay surface. This water layer affects all soil properties including plasticity, compaction, strength and water movement in soil (Yong and Warkentin, 1975; Gillot,1987; Abdi, 1992).

Clay soils may suffer volume changes due to moisture content changes which results in swelling and shrinkage (Bell, 1983). The ability of clay to imbibe water leads to swelling and when it becomes dry it shrinks. Gillot (1987) stated that this phenomenon is influenced by many clay properties including specific surface area, cation exchange capacity, degree of consolidation, organic matter content and the cementitious agents present. Cementitious agents can bond minerals together and this leads to suppression of the swelling by a reduction in the surface area exposed to moisture, and by increasing the strength of the materials which places an internal constraint upon expansion.

2.4 CLAY PLASTICITY AND ACTIVITY

The plastic properties of soil materials are expressed in terms of plastic limit (PL), liquid limit (LL) and plasticity index (PI) as proposed by Atterberg, (1911). Allen (1942) defined these terms as follows: "Liquid limit is the moisture content expressed as a percentage by weight of the oven dried soil at which the soil will just begin to flow when jarred slightly. Plastic limit is the lowest moisture content expressed as a percentage by weight of the oven dried soil at which the soil can be rolled into threads 3 mm in diameter without breaking into pieces. Plasticity index is the difference between the liquid and the plastic limits. It is the range of moisture content in which a soil is plastic".

The activity of a soil is the ratio of the plasticity index to the percentage clay fraction (i.e. % less than 2 μm). Activity is a very useful value indicating the plasticity index of the clay-size fraction of the soil. It also indicates the ability of clay soils to react with chemical agents. The activity of montmorillonite ranges from about 0.5 to 7 and of kaolinite from 0.01 to 0.41, depending on the clay fraction (Grim, 1962).

2.5 ION EXCHANGE

Ion exchange is the replacement of one ion adsorbed on the clay lattice surface by another. The physical properties of clays are dependent on the exchangeable ions. Ion exchange is of great importance in the applied sciences where clay materials are used. The plastic properties of the clay are very different depending on the type of the exchangeable cation present.

Grim (1962, 1968) showed that clay surface is usually negatively charged and this is the cause for cation attraction to the particle surface. He found that the main sources of the negative charge on the clay surface are:

1) Broken bonds around the edges of the silicate-aluminate units leaving unbalanced charges which are balanced by adsorbed cations. The number of broken bonds per unit mass and hence the exchange capacity, increases as the particle size decreases.
2) Substitution within the lattice structure of trivalent aluminium Al^{3+} for quadrivalent silicon Si^{4+} in the tetrahedral sheet and of lower valence ions Mg^{2+} for trivalent aluminium Al^{3+} in the octahedral sheet result in unbalanced charges within the clay structure of some of the clay minerals. This charge imbalance may be balanced either by other lattices (i.e. OH^-) for O^{2-} adsorption of positive cations. Thus, clay particles have negatively charged surfaces and attract positively charged cations (Abdi, 1992).

Exchangeable cations are positively charged ions from salts in the pore water which are attached to the surface of clay particles to balance the excess negative charge. Cation exchange occurs because one cation can be replaced by another of similar valence, or by two of one- half the valence of the original one and so on (Yong and Warkentin, 1975). For example, if clay containing sodium as the exchangeable cations is washed with a solution of calcium chloride, each calcium ion will replace two sodium ions and the sodium can be expelled in the solution. The reaction can be represented as:

$$2\ Na\ clay + Ca\ Cl_2 \Leftrightarrow Ca\ clay + 2\ Na\ Cl$$

Cations can be arranged in a series on the basis of their replacing power. The general order of replacement of the adsorbed cation is : $Li^+ < Na^+ < H^+ < K^+ < NH << Mg^{2+} < Ca^{2+} << Al^{3+}$. At equal concentration any cation will tend to replace those to its left in the series (Grim, 1968).

The number of cations that are exchangeable is defined as the cation exchange capacity and is usually expressed in milliequivalents of cations per 100 grams of oven dry soil (meq/100g). The milliequivalent may be defined as one milligram of hydrogen ions (H^+) or the amount of any other cation that will replace it on the clay mineral surface. The cation exchange capacity should be measured at pH 7. At higher pH more cations are adsorbed, perhaps because of increasing dissociation of weakly bonded $Si\text{-}OH^-$ groups on exposed clay crystal edges. Below pH 5 the cation exchange capacity is constant (Grim, 1962). Table 2.1 gives the cation exchange capacity for the three common clay minerals.

It can be seen from table 2.1 that the large net negative charge carried by the montmorillonite particles and its large specific surface area means that the cation exchange capacity of montmorillonite is very high compared to that of kaolinite and illite.

Table 2.1Values of cation exchange capacities, (Wu, 1976)

Clay Mineral	Exchange Capacities (meq/100g)
Kaolinite	3- 15
Illite	10- 40
Montmorillonite	80- 150

The characteristics of clay which are discussed above affect to different degrees the expansive behaviour of clay soils. These characteristics, including the size and shape of crystal particles, will depend on the expandability of the crystal lattice. The degree of crystalinity governs the swelling behaviour of the clay (Grim, 1962) and hence the "expandability". The next section discusses the swelling behaviour of the "expansive" clay in some detail.

2.6 EXPANSIVE SOILS

2.6.1 INTRODUCTION AND DEFINITION

The volume change of some clayey soils due to change in their water content represent one of the most serious problems in the field of foundation engineering. Volume change may cause unpredictable movement of structures that are built on such soils. Expansive soil can be defined as "a clay soil capable of undergoing a large volume change, (shrinkage and swelling) when subject to variations in moisture content. When the predominant clay mineral of the soil is of the swelling lattice type, e.g. montmorillonite, the soil can be classified as an expansive soil (Xidakis, 1979). Some types of illite are considered to be expansive while kaolinite is considered an inexpansive lattice. Swelling potential refers to both the swelling percent and the swelling pressure. Mowafy *et al.,* (1990) defined the swelling pressure as the external pressure required to consolidate a preswelled sample to its initial void ratio. While Chen (1975) defined the swelling pressure for undisturbed soil as " the pressure required to keep a volume of a soil constant at its natural dry density"; and for remoulded soils as "the pressure required to keep the volume of a soil constant at maximum proctor density".

The problem of expansive soil was not recognised by soil engineers until about 1940. Prior to 1940 the damages caused to various structures were attributed to poor construction and /or the settlement of the foundation soils. In 1938 the scientists of the U. S. Bureau of Reclamation first realised the role of expansive soils in damage to buildings (Chen, 1975). Since then much research has been published on the problems of expansive clay. However, in spite of the valuable effort and work of the scientist and engineers, the processes involved the swelling and shrinkage of soils still needs much clarification (Xidakis, 1979).

2.6.2 PROPERTIES OF EXPANSIVE SOILS

a) Morphological and mineralogical properties

The colours of expansive soils vary; most of them are dark or black in colour. However, others colours such as grey, light green or red have been observed in some deposits; thus the colour is not a distinct characteristic of these soils (Lyon Assoc., 1971).

The mineralogical compositions of the clay fraction of expansive soils include predominantly montmorillonite. Kaolinite and illite may be present in minor quantities (Fadl, 1971). Only expansive alluvial soils contain illite and only expansive soils over volcanic rocks contain halloysite. Calcium and magnesium are the principal exchangeable cations, generally with minor amounts of potassium and sodium. Sodium may be the principal cation in some exceptional cases. Cation exchange capacities are usually high (30 to 64.9 m eq /100 g clay), clay content is also high (>20%) with a very low percent of coarse sand and gravel. Clays saturated with sodium cations display a higher volume change than clays with calcium cations, and they display a higher swelling pressure (Lyon Assoc., 1971). The pH of these soils ranges from 6.30 to 9.20.

Some clays, such as the montmorillonites, swell when wet and shrink when dry. After drying, soils high in montmorillonite are criss-crossed by wide, deep cracks that allow rain to penetrate rapidly, see figure 2.7. Later, because of swelling, montmorillonitic soil close up and become much more impervious than kaolinitic or chloritic soils. Some swelling is due to the penetration of water between crystal layers, resulting in expansion of individual particles. However, most of the swelling results from water attracted to the colloids and to ions adsorbed by them. This property is responsible for the development and stability of soil structures. Soils rich in montmorillonite cover large areas, more than 350,000 km^2, around the world (Gradusov, 1974).

Figure 2.7 A field scene showing the cracks that result when a soil high in montmorillonitic clay dries out, Brady, 1990.

b) Engineering properties

Most of the literature on expansive clays comes from India and the United States; with little from Africa and Australia. The engineering properties of expansive soils according to Chen 1975 and the Lyon Association, 1971, are summarized in table 2.2.

From table 2.2, wide ranges of variability are observed in the engineering properties of expansive soils reflecting the fact that these soils are collected from different sources and areas with different characteristics and environmental conditions.

Table 2.2 Engineering properties of the expansive clays, Lyon association, 1971; Chen 1975

Liquid limits	22% - 124%
Plasticity Indices	11% -74%
Shrinkage limits	10% -30%
Classification using AASHTO	A-7-5 to A-7-6
Organic matter content %	0.5 – 2.5
Specific gravity	2.65- 2.75
CBR reported after 4 days soaking period	< 2
Effective Cohesion	2.5- 3.0 kN/m^2
Effective Angle of internal friction	15° -25°
Volume expansion	40% -150%
Free swelling	100% -400%
Swelling pressure	1000-3000 kN/m^2

2.6.3 DAMAGE CAUSED BY EXPANSIVE SOILS

Many problems associated with foundations on expansive soils have been reported from all over the world. These problems include the heaving, cracking and break up of pavements, building foundations and channel and reservoirs linings. The foundations of light structures supported on the ground (e.g. highways) are more affected by expansive soil problems than heavy or deep buried structures (Xidakis, 1979). The annual cost of structural damages in the U.S.A alone is about $2.3 billion, more than twice the damage from earthquakes, hurricanes and floods (Bruer, 1973).

Many researchers have investigated the effect of initial water content on the amount of swelling and swelling pressure. They noticed that each particular expansive soil has a certain initial water content at which no swelling phenomenon will occur, and this is dependent to a certain degree on the salt concentration of the pore water. They found that the percentage of swelling was inversely proportional to the salt concentration in the pore solution.

The area covered by expansive clays in the U. S. A is about 25% of its whole area. Furthermore, expansive clays cover wide areas in many other countries outside the U.S.A, for example in India, Africa and Egypt.

Two examples of the effects of differential settlement caused by expansive soil on structures are given in figures 2.8 and 2.9. These structures which are located in the north-eastern United States, are constructed on deep deposit of expansive soils (Hunt, 1986).

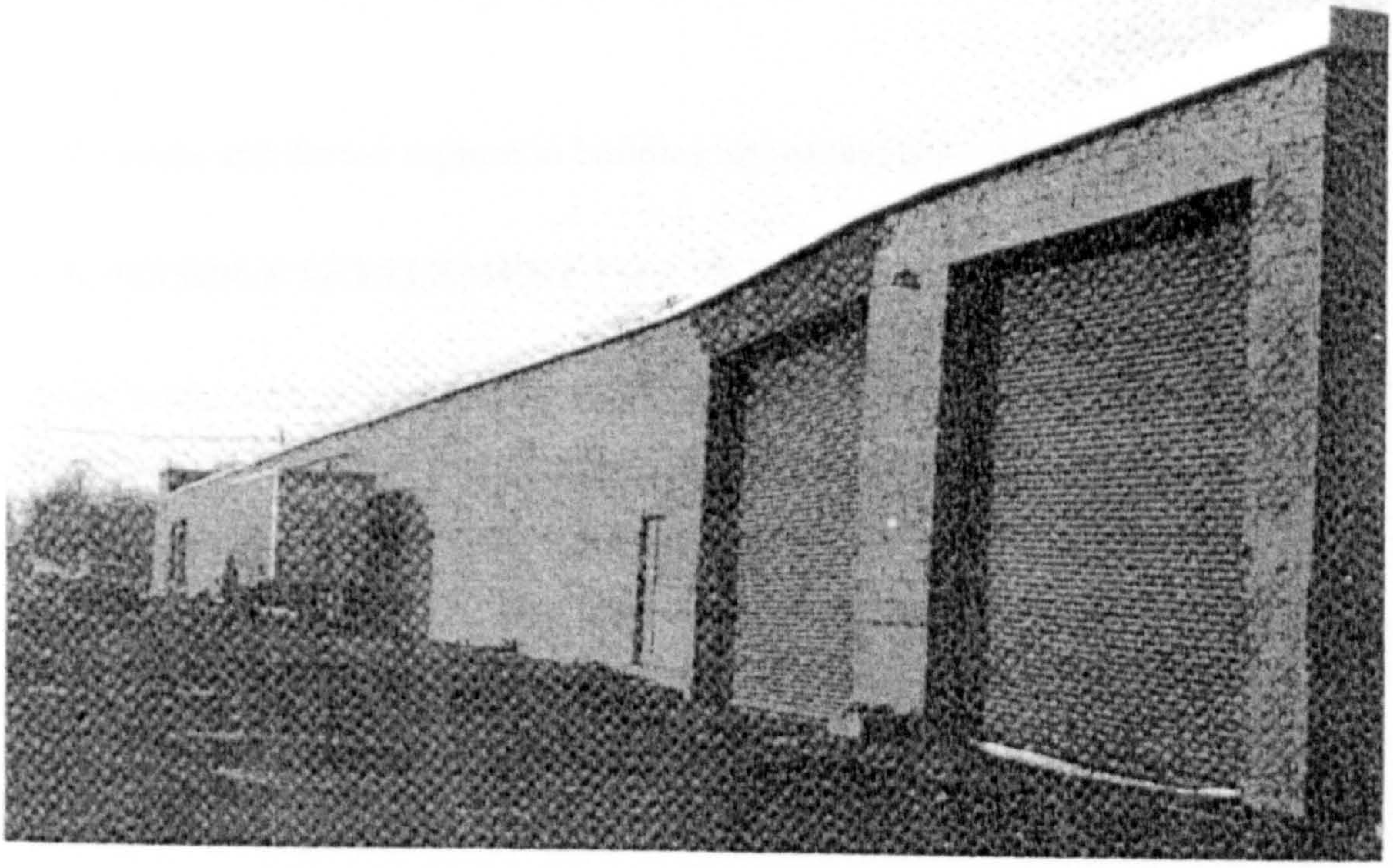

Figure 2.8 Differential settlement apparent along wall of warehouse, Queens, New York, 1960, (Hunt, 1986)

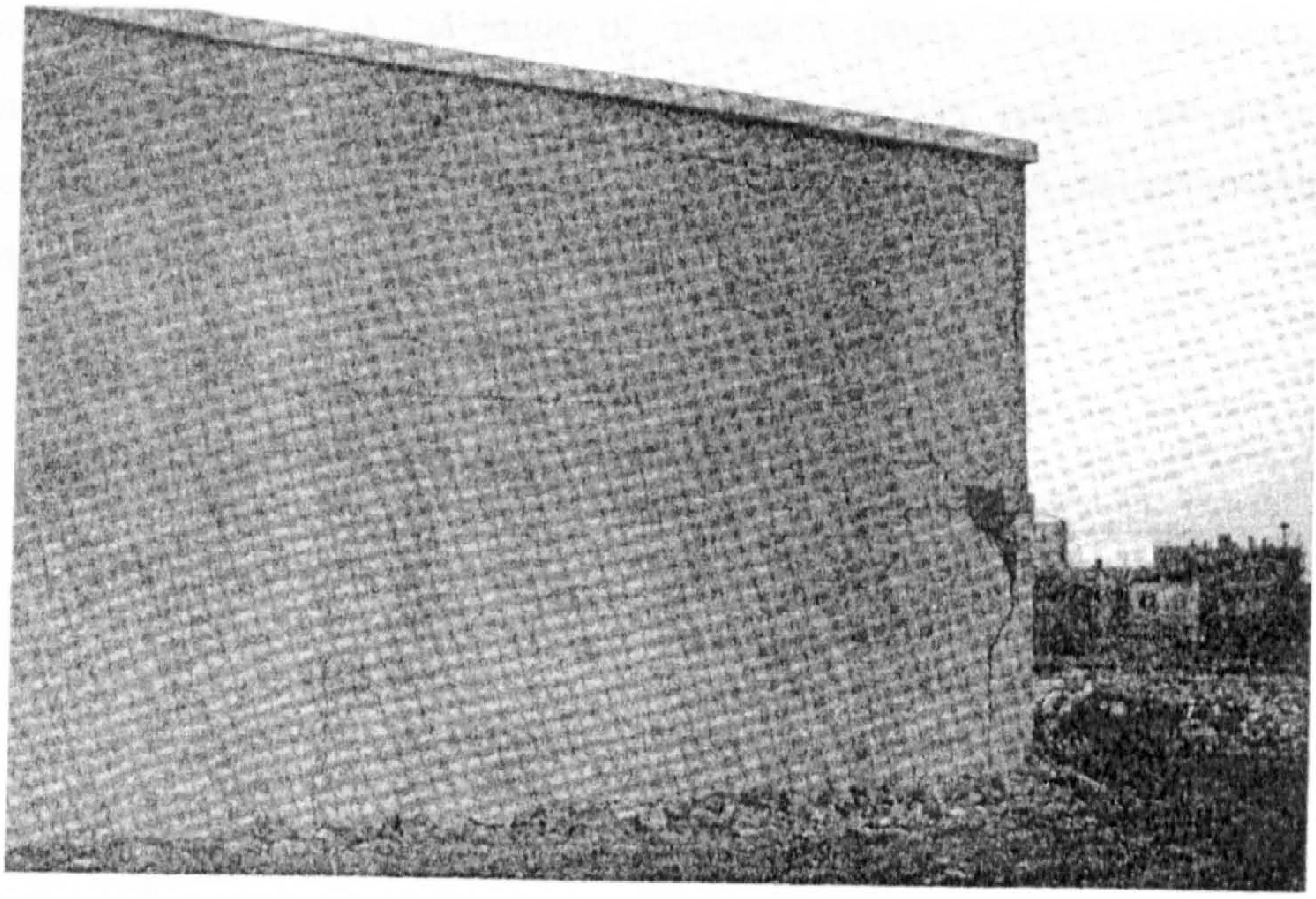

Figure 2.9 Cracks and corner rupture in building shown in figure 2.8, (Hunt, 1986)

2.6.4 PERMISSIBLE SETTLEMENT FOR BUILDINGS

Differential heave and /or contraction of foundation soils are the main reasons for structural damage to buildings or highways founded on expansive soils. Differential heave or settlement of the soil surface is a function of many parameters such as the thickness and mineralogy of the clay layer and the variation in moisture content underneath the structure, etc. Variations in the water content of the soil under and around the structures are due to changes in the environmental conditions e.g. the depth and the frequency of rainfall, the rate of evaporation, mineralogy and depth of ground (Xidakis, 1979). Changes in local conditions, such as breakage of water pipes, leakage of sewer lines and poor drainage of surface water, also change the water content around the structure (Gromko, 1974).

Differential settlement is the controlling factor in structural performance. Bjerrum (1963) published the limiting angles of distortion for various conditions as given in figure 2.10. It was suggested that cracking of panels, in frame building structures, and columns and beams structures are likely to occur if $\delta > 1/300$ and $\delta > 1/150$

respectively, where δ is the angle of distortion (Hunt, 1986). Therefore, it is recommended either to stabilise the expansive clay to reduce the differential settlement or to design the structures built on such soils to sustain the additional stresses caused by any extra settlement.

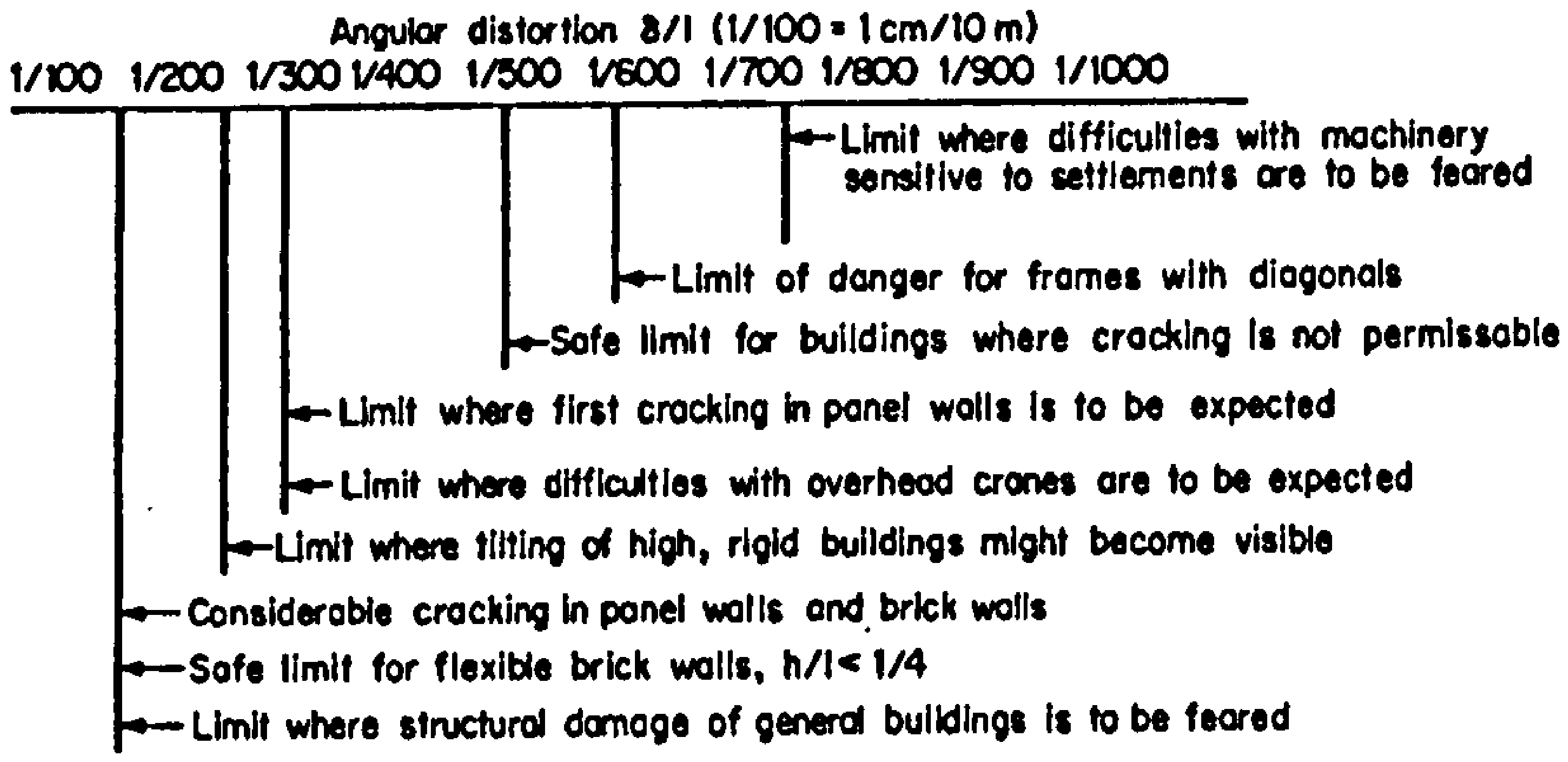

Figure 2.10 Limiting angular distortion for structures, Bjerrum, 1963

2.6.5 IN-SITU HEAVE OF EXPANSIVE SOIL

Soil heave due to soil swelling, or settlement due to shrinkage, are equally common in the field. However, heave is more dangerous to structures and roads than contraction because brittle structures are more susceptible to damage from the doming associated with heave than to dishing. Doming causes tensile stresses in the upper parts of the structure. On the other hand, shrinkage may cause dishing, resulting in tensile stresses in the foundations, which can better resist such stresses (Xidakis, 1979).

Several techniques and laboratory tests have been developed to determine in-situ heave, but it is difficult to obtain duplication of the field conditions in the laboratory. The field behaviour of a soil and structure is affected by factors such as:

There is often a general upward movement beginning shortly after the construction and generally finishing after about 5-6 years. This heave is mainly due to an increase in moisture content underneath the covered area; it is a very slow process and it does not depend on the environmental conditions.

Local heaving which results from breakage of water and/or sewer pipes, watering of gardens, causes severe damage in some cases because it is unpredictable. Cyclic expansion-contraction phenomena are related to seasonal fluctuations of the soil water content, around areas of the building or pavements (Kassiff *et al.,* 1969; Chen, 1975).

Soil swelling is generally assumed to occur normal to the surface because lateral swelling is generally inhibited by adjacent soil. However, because expansive soils can suffer severe cracking and fissuring on drying, lateral swelling may be considerable and may be greater than the swelling normal to the surface (Parcher and Lu, 1965).

Gromko (1974) listed the main factors which might affect the in-situ movement of an expansive soil. One of the important factors is time. Since highly expansive clays tend to exhibit very low permeability, sufficient time is necessary for the swelling process to be completed and the full swelling potential of the soil may not be achieved during the design life of the building (30-40 years). Expansive soils with lower swelling potentials but higher permeabilities may exhibit more in-situ swelling during a single weather season than highly expansive clays.

2.6.6 TREATMENT OF EXPANSIVE SOILS

Treatment of expansive soils is not always easy and/or economic. Many researchers have developed practical methods for construction on expansive soils and these can be summarised as follows:

1. Realignment of the project, (e. g. a highway) to avoid the expansive soil deposits, or excavating and backfilling (highways, building) wherever possible.
2. Minimise the water content change in the expansive clays after construction. One of the most common ways is to construct suitable drainage systems and control the vegetation coverage.
3. Using an appropriate design of the structure on the expansive soils based on the estimated average vertical heave for the environmental conditions.

4. Using one of the various methods of soil stabilisation. There are many ways to control heave potential including control of compaction, i.e. compaction at lower density and higher moisture content, and chemical stabilisation with lime and cement. Lime stabilisation is recommended in the case of expansive clays in preference to cement stabilisation because montmorillonite may retard cement hydration and cement does not improve the workability of the clay. Chemical stabilisation may not be possible if the clay contains a high percentage of organic matter (Chen, 1975; Kassiff *et al.*, 1969; Gromko, 1974; Lyon Assoc., 1971).

One of the most valuable pieces of research carried out on Egyptian swelling soils is the work of Mowafy *et al.*, (1990). They developed three successful techniques for the treatment of Egyptian expansive soils. They found a decrease in the magnitude of swelling and swelling pressure with an increase in the initial water content. They suggested compacting these soils in the field at high moisture content to suppress the swelling potential. They also found that mixing coarse fractions of granular material caused a substantial decrease in the swelling potential of the swelling soils, due to the reduction in the clay fraction in soil mixtures. A higher proportion of sand content, and corresponding lower clay content, results in larger capillary canals in the soil pores and the corresponding reduction in soil suction. They observed a substantial decrease in the swelling percent and swelling pressure with an increase in the concentration of sodium chloride in the pore fluid. More detailed examples are presented in section 2.9.3.

2.7 SOIL STABILISATION

2.7.1 INTRODUCTION

Many investigators have reported that soil properties can be altered by adding other materials (Grim, 1968; Ingles and Metcalf, 1972; Arabi and Wild, 1989; Higgins *et al.*, 1998). The properties that can be affected include plasticity, strength and volume change and the chemical agents used include lime, cement, sodium chloride, fly ash and alkali-activated blast furnace slag. The modification of properties depends to a great extent on the type of clay minerals, the percentage clay fraction in the soil, stabiliser type and percentage, temperature, moisture content, curing time and conditions, and the organic content (Mitchell and Hooper, 1961).

Soil stabilisation is used to improve the mechanical properties of inferior soils in the construction of civil engineering projects such as road pavements, sub-grades, sub-bases, runways and shallow foundations. Therefore, the type of stabilisation chosen in any project depends on both the properties of the soil involved and the nature of the project (Mitchell and Hooper 1961; Abdi, 1992).

2.7.2 LIME STABILISATION

The use of lime in soil stabilisation precedes the beginning of clearly recorded history. Probably the earliest work in modern times on the use of lime in road construction is in 1925, when short experimental lengths of dirt road in the American state of Missouri were treated with hydrated lime to reduce rutting during rain and snow (McDowell, 1966). In 1943, the U.S Corps of Engineers used hydrated lime to reduce the plasticity of a soil used in the construction of a Texas airfield (Johnson, 1948). The sections treated with lime have shown good durability in spite of heavy traffic; whereas sections constructed without lime failed and required extensive repairs. Since that time, much research has been carried out to determine the ideal method of using lime to stabilise soils, and the physical and chemical reactions which may occur (Johnson, 1948).

At the end of the Second World War, the American road-building programme was increased considerably. In Texas, in the U.S.A, the large-scale use of lime in pavement construction programmes began, and Texas is still one of the largest users of lime for this purpose (Dumbleton, 1962).

Johnson (1948) concluded that the addition of about 5% hydrated lime significantly reduced the plasticity of cohesive soils and increased the strength of both fine and coarse-grained soils. By 1951 the U. S. Army Corps of Engineers had prepared specifications, based upon a study begun in 1946, for the construction of road bases using soil stabilised with hydrated lime alone or in conjunction with cement. In

1951, Galloway and Buchanan suggested that the effect of lime on soils was due to an exchange of calcium ions for adsorbed cations on the clay particle surfaces. They found that the reactivity of a soil toward hydrated lime increased as the plasticity index and the cation exchange capacity of the soil increased.

Lime has been successfully employed in many countries outside the U.S.A, particularly in warm countries because it needs a relatively high temperature to react with the clay. In the United Kingdom, lime stabilisation was first used in the construction of the A 38 in Worcestershire in 1951 (Brook- Bradley, 1952). In this project, part of the sub-base was stabilised with hydrated lime.

Lime used in soil stabilisation may be in many forms such as quick lime CaO, hydrated lime $Ca(OH)_2$ and dolomitic lime. Quick lime is the direct product of the calcination of limestone and it seems to be a more effective stabiliser than hydrated lime as it has a high ability to absorb water. Lime stabilisation can be defined as the "reaction between silica and alumina within the clay structure with lime and water to form calcium silicate hydrate and calcium silicate aluminate hydrate gels which subsequently crystallise to bind the structure together" (Rogers *et al.,* 1997)

Generally between 1-3% by dry soil weight of hydrated lime is required to modify soil, while 2-8% by dry soil weight is required for cementation to take place. Bell (1988 a) suggested that 1% lime by dry weight of soil is required for stabilisation for each 10% clay (< 2μm). The exact amount of lime required should be determined by further tests (Bell, 1988 a).

In summary, lime stabilisation has a long history all over the world especially in the warm countries, as it needs a relatively high temperature to react with clay particles. Lime stabilisation using quicklime is more effective than hydrated lime. Generally, between 1- 3 % by dry soil weight of hydrated lime is adequate for modification of clay soil, while, 2- 8 % by dry soil weight of hydrated lime is required for full modification and stabilisation depending upon clay type, percentage of clay in the soil, curing periods and conditions.

2.7.3 LIME STABILISATION PROCESSES

Using lime in the stabilisation of clay soils not only increases the compressive strength of such soils, but also modifies other physical and chemical properties (Fessberg 1959; Brand and Schoenberg, 1959). When lime is added to a clayey soil, lime attacks the clay mineral fraction of the soil and the engineering properties of the soils are altered. These properties include the Atterberg limits of the soil (Wang *et. al.,* 1963; Jan and Walker, 1963; Andrews, 1966), the effective grain size distribution, the moisture content, dry unit weight relationship, and the swelling and shrinkage properties of the soil (Lund and Ramsey, 1959 ; Mitchell and Hooper, 1961), and the soil suction properties (Clare and Crutchley, 1957). This is discussed further in section 2.9.

2.8 CLAY-LIME REACTIONS

2.8.1 INTRODUCTION

Little (1996) reported that practical reasons for the addition of lime to the soil are to improve workability and compaction and reduce swelling and shrinking characteristics by saturating the clay with calcium ions. However, the chemical interaction of lime with clay must also be considered an important part of a permanent improvement due to the formation of cementitous materials which increases the strength of soil-lime mixtures. The reaction will be stronger in the case of high silicate content in the soil.

The addition of lime to a clay soil in the presence of water, results in several types of chemical reactions taking place simultaneously, which makes it difficult to separate and analyse them. However, the most important reactions can be divided into four groups; (a) cation exchange; (b) flocculation and agglomeration; (c) carbonation; and (d) pozzolanic reactions, (Bell and Coulthard, 1990; Bari, 1995; Thompson, 1966 a).

2.8.2 CATION EXCHANGE

In lime stabilisation cation exchange is a physico-chemical reaction whereby Ca^{2+} ions from the lime displace the sodium or magnesium ions naturally present in the soil. The addition of lime to a soil creates a concentration of free Ca^{2+} that will replace dissimilar adsorbed cations on the colloidal surface of the clay (see section 2.5).

2.8.3 FLOCCULATION AND AGGLOMERATION

Flocculation where clay particles clump together into larger sized aggregates takes place rapidly. This is thought to be caused by cation exchange. It has been suggested that cation exchange and the resulting modification in the electrical double layer alter the density of electrical charge around clay particles causing them to become electrically attracted to each other (Hilt and Davidson, 1960; Eades and Grim, 1960). Herzog and Mitchell (1961) suggested that flocculation was due to an increase in the electrolyte concentration in the pore water and ion exchange. Flocculation produces an apparent change in texture as a result of a larger sized aggregates and the soil becomes more friable

Modification of a clay soil by the addition of lime will depend substantially upon the dominant cation originally adsorbed in the double layer, and upon the type of clay. For example, a sodium-based montmorillonite has a comparatively high cation exchange capacity and will require a relatively high percentage addition of lime to achieve calcium saturation and full flocculation. Hilt and Davidson (1960) suggested the existence of a "lime fixation point". This point represents the maximum percentage of lime addition at which no further calcium cations may crowd onto the clay particles, and above which any lime excess can not make any further modification of the flocculation or the plasticity

Most researchers have reported that flocculation and agglomeration is largely responsible for the initial material property changes. The pozzolanic reactions are responsible for long term changes (Ingles and Metcalf, 1972; Wild *et al.*, 1988). Thompson (1966 a) found that flocculation and agglomeration are responsible for the change in plasticity, shrinkage, and workability characteristics of the mixtures but

Thompson (1968) found that cation exchange and flocculation and agglomeration are not the basic lime-soil reactions responsible for the marked strength increases noted for many lime- soil mixtures.

2.8.4 CARBONATION

Carbonation is the reaction of lime with carbon dioxide from the atmosphere to form calcium carbonate and/or magnesium carbonate, depending on the type of lime used (Davidson and Handy, 1960). Although this reaction was originally believed by some researchers to constitute the initial source of stability in soil-lime mixtures, it is now recognised that the reaction products are only weakly cementitious. Goldberg and Klein (1952), and Eades *et al.*, (1962) observed the formation of calcium carbonate when tested soil-lime mixtures cured in the open air. Also, Eades *et al.*, confirmed the formation of calcium carbonate in field lime stabilisation which consumed a considerable part of the available lime for pozzolanic reactions.

Carbonation consumes part of the lime and that affects the extent of the pozzolanic reactions which are the most important reactions resulting in products of cementitious agents (Eades and Grim, 1960; Thompson 1968). Therefore, it is desirable that carbonation should be minimised during construction because carbonation of free calcium reduces the free lime available for the pozzolanic reaction and cation exchange (Herzog and Mitchell 1961; Eades et. al., 1962; Little, 1996). This precaution is of particular relevance to laboratory studies of the remaining reaction mechanisms, implying that lime should be stored in an airtight container. However, carbonation will ultimately occur, but it is not serious if it occurs following a reasonable level of high–pH stabilisation (Little *et al.*, 1996). The elevated level of pH causes silica and alumina from soil to be dissolved and the principle cementitious products will be formed. Bagonza *et al.*, (1987) observed in a laboratory investigation that a carbon dioxide environment leads to carbonation in lime stabilisation, and completely penetrated samples in less than three days and prevented the principal cementious products (C-S-H, C-A-H and C-A-S-H) from being formed as carbonation consumed the available lime.

In contrast, Graves *et al.*, (1990) and Little *et al.*, (1994) demonstrated the structural benefits of carbonation cementation on limestone bases in Florida and Texas. They pointed out that although these bases had little clay content, they received structural benefits from the lime. The strength that came from carbonation was adequate and satisfied the project requirement.

In conclusions, carbonation is the reaction of lime with carbon dioxide from the atmosphere to form calcium carbonate. This reaction consumes a considerable amount of the available lime for the pozzolanic reactions and prevents formation of the principal cementitious products.

2.8.5 THE POZZOLANIC REACTION

The reaction between lime, water, and the various sources of silica and alumina in clay to form cementious materials is referred to as the soil-lime pozzolanic reaction. The cementing agents formed are generally regarded as the major sources of the strength increase in lime-soil mixtures. Possible sources of silica and alumina in typical soils include clay minerals, quartz, feldspars, mica, and other similar silicate or alumino-silicate minerals (Thompson, 1964; Eades, 1962).

The reaction products of clay-lime mixtures are very similar to those formed during cement hydration. The reaction between clay particles in a soil and lime increases bonding between clay particles and hence increases the strength of the mixtures. This increases with the length of the curing periods, and has been attributed to the progressive dissolution of SiO_2 and Al_2O_3, as the reaction continues (Croft, 1964).

In clay-lime reactions the main reaction products formed are amorphous or poorly crystalline calcium-silicate-hydrate (C-S-H), and calcium alumino-silicate hydrate (C-S-A-H) and crystalline calcium aluminate hydrate (C-A-H) phase (Sloane, 1965; Diamond and Kinter, 1966; Arabi and Wild, 1989).

Hilt and Davidson (1961) observed that the quantity of lime required to produce the maximum change in the Atterberg limits of a clay soil was also the minimum quantity that was required to be added to the soil before any strength changes occurred. This amount is the lime fixation point or the lime retention point.

Cabrera and Nwakanma (1979) studied the pozzolanic reactions and reaction mechanisms of a red tropical soil and lime system. They found that lime is consumed at a fast rate in the initial stages of the reaction, up to five to seven days. The rate of consumption of lime slows after that. It is interesting to note that the total consumption of lime between seven and twenty eight days represents about eight per cent of the lime consumed during the initial seven days. They suggested that the increase in strength of a soil– lime system beyond the first seven days cannot be explained in terms of the predicted pozzolanic reaction products as only a small amount of lime was consumed between seven and twenty eight days. It is suggested that the increase in strength, with an increase in the curing period, shown by these soils is mainly due to the changes in the structure of the cementitious products formed during the pozzolanic reaction, i.e. hydration and increase in crystallinity of the reaction products, without forming new products. It is unlikely that pozzolanic reactions finish after only 7 days. However, the pozzolanic reaction and formation of cementitious materials depend on many factors, curing conditions and periods, clay type and content and moisture content at the mixing time.

The simplified equations of a typical soil- lime reaction are as follows:

$$Ca(OH)_2 \rightarrow Ca^{2+} + 2\,(OH)^-$$

$$Ca^{2+} + 2\,(OH)^- + SiO_2\text{ (Clay Silica)} \rightarrow \text{C-S-H}$$

$$Ca^{2+} + 2\,(OH)^- + Al_2O_3\text{ (Clay Alumina)} \rightarrow \text{C-A-H}$$

Many authors have identified the products which may be formed in the soil-lime reaction process, in most instances by the use of X-ray diffraction (XRD) and differential thermal analysis (DTA). It has been established that the exact long-term cementious materials depend substantially upon the original clay mineral and upon the reaction conditions.

Eades and Grim (1960) studied, using XRD and differential thermal analysis (DTA), the reaction products of pure clay kaolinite and hydrated lime mixtures cured at 140°F (60°C). They noticed that as the lime content and the curing time increased kaolinite is attacked and crystalline calcium silicate hydrate forms. They suggested that many different intermediate components might be formed before silica and lime reach equilibrium. Eades and Grim (1960) also showed that the illite-lime reaction gives the same calcium silicate hydrate as kaolinite. Unlike kaolinite and illite in reaction, X-ray data for the montmorillonite reaction demonstrates that there is a destruction of the mineral structure with little formation of new materials. However, the compressive strength values for the treated montmorillonite seems to indicate that there is a strength increase due to the formation of reaction products. The major cementitious materials that are expected to form (C-S-H and C-S-A-H) are amorphous which are not detected by the XRD.

In contrast to Eades and Grim, other authors mention the forming of calcium silicate hydrate (C-S-H gel), together with crystalline calcium-aluminate hydrate phases (C_3AH_6 and C_4AH_{13}) and calcium alumino silicate hydrates (such as C_2ASH_8) (Croft, 1964; Sloane; 1965, Arabi and Wild, 1989; Abdi, 1992). Unlike kaolinite, the triple layer clay minerals (illite and montmorillonite) completely deteriorate without the formation of any new crystalline phase detected by X- ray diffraction. The data obtained from the strength tests of montmorillonite-clay mixtures, however, suggest the possible formation of non-crystalline calcium silicate hydrate gel that might not be detected by XRD (Eades and Grim, 1960).

Hilt and Davidson (1960) studied the long-term reaction products formed in a montmorillonite-lime mixture. They identified the formation of crystalline calcium aluminate hydrate and C-S-H, at normal temperature. Like Hilt and Davidson, Glenn and Handy (1963) identified the formation of calcium silicate hydrate and calcium aluminate hydrate, (C_4AH_{13}) together with (C_3AH_6), in the montmorillonite lime mixture. They also observed the formation of calcium silicate hydrate and calcium aluminate hydrate in the kaolinite lime mixtures.

Lees *et al.*, (1983) also studied the reaction products in soil-lime and soil-lime-sodium chloride mixtures by X- ray diffraction and scanning electron microscopy. They used two main clay minerals, kaolinite and montmorillonite, and a variable percentage of lime and sodium chloride. The formation of calcium aluminate hydrate C_4AH_{13} and poorly crystallised C-S-H was confirmed in the case of kaolinite–lime mixtures. The formation of calcium aluminate chloride hydrate and sodium calcium silicate hydrate has been found using sodium chloride in conjunction with the lime. For montmorillonite clay- lime treatment they demonstrated the formation of poorly crystallised C-S-H and the calcium aluminate hydrates C_4AH_{13} or CAH_{10}. Treatment with lime and sodium chloride resulted in the formation of sodium calcium chloride silicate hydrate and calcium aluminate chloride hydrate.

Croft (1964) studied the mineralogical changes in pure clay-lime pastes cured at 40°C in compacted mixtures. From X-ray diffraction, differential thermal analysis, and the chemical analysis of the clay-lime reaction, it was found that the action of hydrated lime on various clays in pastes showed considerable attack on the mineral structures. He found that the action of lime on kaolinite mixtures produced partially hydrated calcium aluminate and calcium silicate hydrate in all samples.

Eades and Grim (1963) found that a considerable reaction occurred in quartz and mica bearing soils with lime when examined under a petrographic microscope. The quartz and mica particles developed fuzzy outlines and visible cementing gel. X-ray diffraction showed the presence of calcium silicate hydrate as well as calcium carbonate. These authors did not refer to the possibility that calcium aluminate hydrate might also form.

Diamond *et al.,* (1964) studied the reaction products formed in calcium hydroxide-kaolinite and montmorillonite mixtures cured at 60 °C. They suggested that C-S-H was formed in the case of kaolinite, and C-S-H with C_3AH_6 formed with montmorillonite. They suggested in montmorillonite clay, that alumina released by the lime was partially incorporated in the C-S-H phase lattice. Diamond *et al.*, (1964) also found that tobermorite gel and calcium aluminate hydrate products were formed by the reaction of lime with mica and illite.

Diamond and Kinter (1966) from a literature review conducted on the long term reaction products of clay-lime mixtures, concluded that there are two main reaction products; calcium silicate hydrate and calcium aluminate hydrate. The calcium silicate hydrate may form in three different forms, C-S-H (gel), C-S-H (I) and C-S-H (II), depending on the reaction conditions.

When mixtures of illite and montmorillonite clay and lime were used, the resulting products were similar to those products formed using kaolinite. It might appear from the study of Croft (1964) that the hydrated calcium aluminates were metastable in the presence of carbon dioxide, because carbonated lime was observed. After curing, the montmorillonite mixtures were characterised by poorly crystallised forms of C_4AH_{13} and C-S-H. The reaction products for the mixed layered illite-montmorillonite were similar. A weak pattern of C-S-H was observed for illite.

Diamond and Kinter (1975) found that lime reacts instantaneously with hydrous alumina of a high surface area to generate a well-crystallised tetra calcium aluminate hydrate, C_4AH_{13}. Like Diamond and Kinter, Hilt and Davidson (1961), identified a reaction product very similar to the structure of C_4AH_{13} when they examined the kaolinite-lime mixture using X-ray diffraction. Diamond *et al.*, (1964) concluded that the reaction between lime and clay depends upon the reaction conditions. At 60°C kaolinite and montmorillonite produced calcium silicate hydrate C-S-H, and the kaolinite produced C_3AH_6; no crystalline calcium aluminate compounds were formed for montmorillonite. At lower temperatures the products from both clays were considered to be tobermorite gel, and the calcium aluminate hydrate at d spacing of 7.6 Å in the X-ray diffraction test.

Charles *et al.*, (1982) studied the reaction products of lime-treated southeastern United States soils. X-ray diffraction analysis, thermogravimetric analysis, and scanning electron microscopy were carried out in this investigation on six soil series, (Cecil, Chewacla, Eutaw, Sumter, Tatum, and Wilox). Six percent by dry weight of high calcium hydrated lime was employed as a stabilising agent. The properties of these six soils are illustrated in table 2.3.

Samples were compacted in a Harvard miniature compaction mould in 3 layers by using 25 blows/layer at the optimum moisture content (Ford, 1978). The compacted samples were sealed with plastic wrap to prevent moisture loss and then cured at 49°C (120° F) for 48 hours.

They identified, using thermogravimetric analysis (TGA), the presence of calcium alumina hydrate (C_4AH_{13}) in the Cecil soil only, (C_3AH_6) in the Cecil and Eutaw soils, and C-S-H (gel) in all types of soils except the Sumter. However, unknown products were also noted from thermogravimetric analysis at 440°, 450°, and 460°C.

Table 2.3 Soil properties used by Charles *et al.*, 1982

Soil Series	Family	Natural pH	Liquid Limit (%)	Plastic Limit (%)	Percentage passing No.200
Cecil	Clayey, Kaolinite	4.9	52	33	85
Chewacla	Fine- loamy.	8.2	24	17	43
Eutaw	Very fine, Montmorillonitic	5.4	70	37	99
Sumter	Fine-silty, carbonic	8.4	51	24	88
Tatum	Clayey mixed	4.3	33	24	83
Wilcox	Fine-montomorillonite	4.3	72	42	90

The montomorillonite contents in these soils were between 40% and 43 %. Charles *et al.*, (1982) found that the Cecil and Chewacla soils showed significant gains in strength after lime treatment and curing. Although the Tatum soil did not have a large increase in strength, the scanning electron micrographs suggest the formation of cementitious materials.

Eades *et al.*, (1962) identified, using XRD, the presence of calcium silicate hydrate and calcium carbonate in a field investigation carried out in Virginia, U.S.A. The soil had various clay components and the stabiliser was hydrated lime.

Unlike most researchers, Goldberg and Klein (1952) found only the presence of calcium carbonate when they studied the reaction products of the clay-lime mixture using X- ray diffraction. Air curing of the clay-lime mixtures leads to significant carbonation of the lime and the lime may be consumed before any pozzolanic reaction take place between the lime and the clay minerals.

Marks and Halliburton (1972) using DTA, studied the effect of sodium chloride as an additive in lime-soil stabilisation and they found a new peak at 880°C indicating a mineral due to the addition of salt. They suggested two explanations. First, that sodium chloride reacts with clay minerals, disturbing the aluminium bonding in the clay and consequently calcium ions may more easily unite with aluminium and silicate to form new minerals. Secondly, that an increase in the solubility of silicate due to the presence of sodium chloride makes silicate available for reaction with calcium at a greater rate than normal.

Clay-lime reactions can be summarised as a cation exchange process whereby Ca^{2+} ions from the lime displace sodium or magnesium ions naturally present in the soil. Flocculation and agglomeration, due to cation exchange, causes clay particles to clump together into larger sized aggregates. Carbonation, which is the reaction of lime with carbon dioxide from the atmosphere forms a weak cementitious product (calcium carbonate). The disadvantage of carbonation is it consumes the lime available for the pozzolanic reaction which is the main source of strength in clay-lime reactions. The pozzolanic reactions between lime and clay produce semi-crystalline calcium silicate hydrate, crystalline calcium aluminate hydrate and calcium aluminate silicate hydrate. These are the basic products in soil lime reactions and the main causes of enhanced soil characteristics, strength and volume stability.

2.9 EFFECT OF LIME ON THE ENGINEERING BEHAVIOUR OF SOIL

2.9.1 INTRODUCTION

As may be expected from the description of reactions described in the preceding section, the addition of lime affects many of the engineering properties of soils. These include the liquid limit (LL), plastic limit (PL), plasticity index (PI), optimum moisture content (OMC), maximum dry density (MDD) and unconfined compression strength (UCS).

2.9.2 EFFECT OF LIME ON PLASTICITY CHARACTERISTICS

Usually there is a general increase in the plastic limit on the addition of lime (Diamond and Kinter, 1964; Brandle, 1981; Sabry and Parcher, 1979; Akoto and Singh, 1981). The amount of lime needed to cause changes in the plastic limit varies from 1 to 4 % by dry soil weight, depending on the amount and type of clay minerals present in the soil (Bell and Coulthard, 1990). Hilt and Davidson (1960) studied the effect of adding lime on the plasticity of different types of soil. They pointed out that the plastic limit generally increased for all types of soils. The largest increase in plastic limit was obtained when montmorillonite was the principal clay mineral. The plastic limit increase for illite is less than montomorillonite, and kaolinite showed the smallest increase in plastic limit. Mateos (1964) showed that the minimum amount of lime required to be added to montmorillonite clays for maximum increase in plastic limit (PL_{mi}) is: PL_{mi}= (% 2 micron clay/35) + 1.25

However, the effect of adding lime on the liquid limit of soils is not so clear, and a general trend is not apparent. Some investigators reported that the liquid limit increases (Clare and Crutchley; 1957, Dawson, 1956; Croft, 1964, Akoto and Singh, 1981), while others reported that both increase and decrease can occur depending on the soil under test (Diamond and Kinter, 1964; Lund and Ramsey, 1959).

The liquid limit of montmorillonite decreases very rapidly, while the plastic limit increases. However, the liquid limit of kaolinite may remain constant after lime treatment or increase (Rogers, 1988; Arabi and Wild, 1989). As a result of the decrease in montmorillonite liquid limit, and the increase in plastic limit, the plasticity index falls rapidly. Kaolinite is rather variable and most researchers have identified an increase in liquid limit on addition of lime (Abdi, 1992). Some researchers have observed a decrease in liquid limit and plasticity index beyond 6% of lime addition by dry soil weight.

Dumbleton (1962) reported an increase in liquid limit and plastic limit at low lime content and gradual decrease in liquid limit and plastic limit with further increase lime content. He concluded that plasticity is affected by clay type, lime addition and time. Like Dumbleton, Abdelkader and Hamdani (1985) studied the effect of lime on the stabilisation of two soil samples obtained from Damanhour, D soil and Elfayoum, E soil, in Egypt. The clay fraction in both was mainly montmorillonite with a minor amount of kaolinite. They found an increase in the liquid and plastic limits with a decrease in the plasticity index for low lime content (see figure 2.11).

Sherwood *et al.*, (1993) studied London clay and found that the clay became easier to compact and workability was improved after adding lime. They also concluded that the liquid limit was altered with low lime contents, whereas the plastic limit required greater lime addition to attain maximum change (see figure 2.12). They also studied the effect of the curing period and found that different clays need different curing periods to achieve full modification.

Rogers *et al.*, (1997) studied the effect of lime modification on four different clays. They demonstrated that the liquid limit generally increases with low lime content. However, the plastic limit requires greater lime addition to attain a significant change. However, Clare and Crutchley (1957) found that the addition of 1 % lime raised the liquid limit of the clay from 72 % to 88 %, but any further increase in lime content reduced this value. They also found that increasing curing time reduces the liquid limit of 1% lime mixes, while an increase in the liquid limit values was observed after curing at higher lime content.

Anand *et al.*, (1996) studied the behaviour of lime treated Louisiana silty clay soil, in the U.S.A. They found that lime treatment resulted in an increase in plastic limit, and a decrease in liquid limit and plasticity index. Changing the curing time of the system generally resulted in further changes.

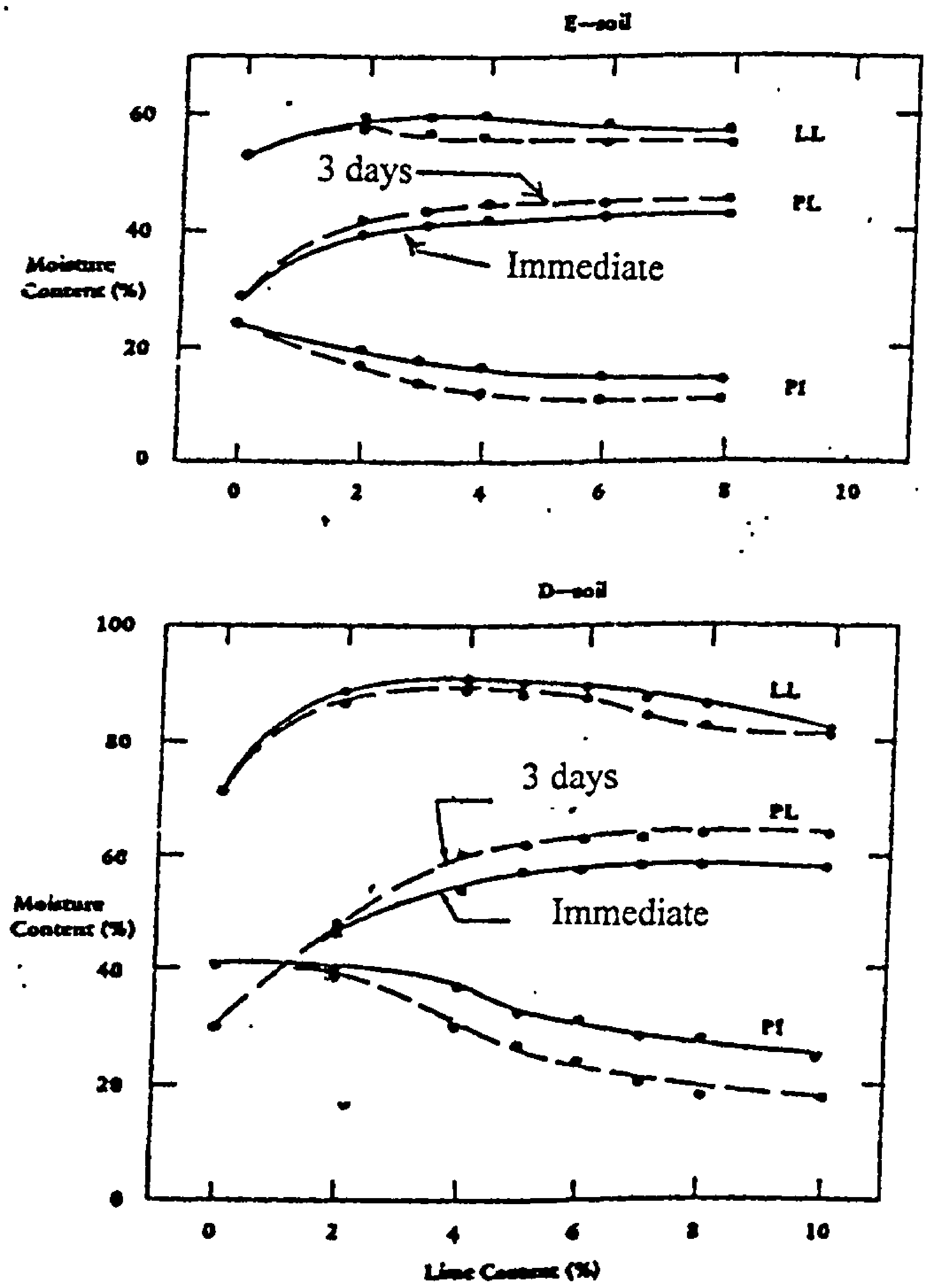

Figure 2.11 Effect of lime content on Atterberg limits, Abdelkader and Hamdani, 1985

The effect of curing time on the plasticity properties has been studied by many researchers. Most observed further decreases in the plasticity index and further increases in plastic limit with curing time (Akoto and Singh, 1981; Dumbleton, 1962). Wolfe and Allan (1964) reported a substantial increase in plasticity index for a number of lime- soil mixtures when cured for 2 days compared with samples tested immediately after the addition of lime. For longer curing periods (7 to 28 days) the effect was reversed, and the plasticity index significantly decreased in most cases. It is not usual to carry out Atterberg limit tests after such long curing periods as the new cementitious materials formed due to the pozzolanic reaction of lime with the clay minerals may produce misleading results as the soil after curing is effectively a different material.

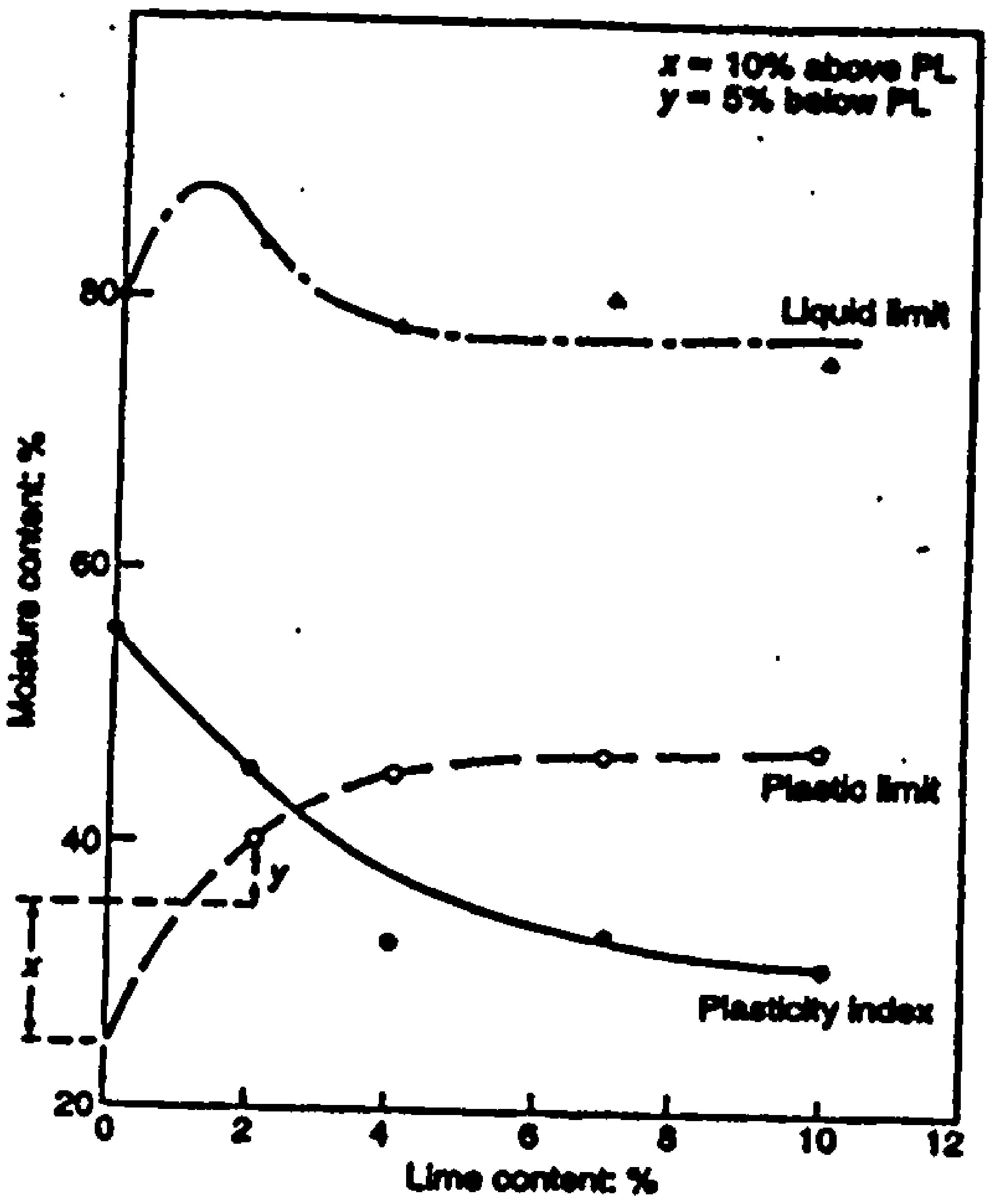

Figure 2.12 Effect of lime content on the plasticity properties of London clay, Sherwood *et al.*, 1993

Like Wolfe and Allan (1964), Arabi (1987) concluded that the effect of curing time and curing temperature on the plasticity properties of a lime-stabilised illite soil showed an increase in plasticity index with an increase in the curing period. He also found that the curing temperature had a more significant effect on plasticity properties compared to curing period. The curing temperature when determining the liquid limit should be a standard cool place (BS 1377:1975 Test 2(A)).

Elsekelly (1987) studied the effect of adding lime and sodium chloride on the stabilization of an Egyptian clay. He concluded that the liquid limit and plasticity index is dependent upon the clay content of the soil. The plasticity index decreases with the addition of sodium chloride. Further decreases in plasticity index occurred with a 2 % lime addition to the salt treated materials.

Howeedy and Zedan (1991) studied the using of local flyash, which is produced from burning wood coal, cotton and maize woods in Egyptian villages, alone or with lime for the stabilisation of alluvial soil deposits of the Egyptian Delta. The soil and flyash were taken at random from Banha city, Egypt. The hydrated lime is also locally manufactured. Physical, chemical and mineralogical properties of soil, flyash and lime are given in table 2.4 and 2.5.

Howeedy and Zedan (1991) observed a decrease in the plasticity index of the soil by 55% and 73% after the addition of 10% flyash and {(20% total binder (10% lime and 10% flyash)} respectively. The detailed effect of flyash and lime-flyash on the Atterberg limits of the soil is shown in table 2.6.

Apart from the increase or the decrease of the liquid limit, the increase in plastic limit is such that the plasticity index is usually reduced with the addition of lime.

Table 2.4 Physical properties of clayey soil and flyash, Howeedy and Zedan 1991

Properties	Clayey soil	Flyash
AASHO Classification	A-7-5 (16)	-
Natural water content, %	7.25	0
Liquid limit, %	74	0
Plastic limit, %	34	0
Shrinkage limit, %	14.5	0
Plasticity index, %	40	0
Specific gravity	2.6	8.52
Optimum water content %	28	47
Maximum dry density, pcf	96.4	60.65
Organic content %	1	0

Table 2.5 Chemical analyses of clayey soil, flyash and lime, Howeedy and Zedan, 1991

Mineral	Oxide content (%)		
	Clayey soil	Flyash	Lime
SiO_2	51.50	50.77	15.02
Al_2O_3	16.68	9.01	9.90
Fe_2O_3	9.92	6.82	0.65
TiO_2	2.52	1.77	0.25
Ca O	3.52	9.24	38.94
Mg O	4.75	5.70	5.07
Na_2O	1.96	2.70	1.65
K_2O	1.23	7.95	0.21
SO_3	0.68	1.42	1.16
L.O.I	6.96	4.83	27.12
Total	99.73	100.21	99.97

Table 2.6 Atterberg limits for soil, flyash-soil and lime-flyash-soil mixture, Howeedy and Zedan, 1991.

Atterberg limits	Clay	Percentage of flyash					Percentage of lime: flyash				
		2	4	6	8	10	2:2	4:4	6:6	8:8	10:10
L.L, %	74	72	71	61	57	56	71	68	65	58	22
P.L, %	34	35	36	36.5	38	38	35	36	38	40	41
P.I, %	40	37	35	34.5	18	18	36	32	27	18	11

In conclusion, most of the researchers reported an increase in liquid and plastic limit at low lime content and gradual decrease in liquid and plastic limit with further increase in lime content. Plasticity index decreases with an increase in lime content. The plasticity is mainly affected by clay type and curing time.

2.9.3 EFFECT OF LIME ON THE VOLUME STABILITY AND SWELLING POTENTIAL OF CLAY SOILS

Lime reduces the volume changes that occur in clay soils due to change of moisture content. Many investigators have studied the effect of adding lime on the shrinkage limit. The shrinkage limit is the moisture content below which shrinkage no longer occurs. It describes the limit between brittle and plastic states of a clay and is usually distinguished by a colour change (Al-Rawi and Awad, 1981). Generally, an increase in the shrinkage limit has been found after adding lime to a clay soil.

Wang and Handy (1966) observed that the addition of lime to a clay soil increased its shrinkage limit Like Wang and Handy, Mateos (1964) showed that a small quantity of lime, up to 4%, increased the shrinkage limit of clay soil and that quick lime is more effective in reducing the shrinkage characteristics of a soil than hydrated limes. Dumbleton (1962) also found that the lime was found more effective than cement in reducing the shrinkage of London clay and silty clay.

With a small addition of lime to a clay soil, the higher the calcium oxide content in the hydrated lime the more effective it is, but with about 8 % addition all limes cause a similar increase in the shrinkage limit (Bell and Coulthard, 1990). Tests carried out by the US Bureau of reclamation on clay soils indicated that the addition of 4 % of lime increased the shrinkage limit from 7 to 26 %.

In contrast, Stocker (1972) found that the shrinkage limit decreases with addition of lime. Abdelkader and Hamdani (1985) studied the effect of lime on the shrinkage potential of two Egyptian clay samples. They found insignificant change in the shrinkage potential of Damanhour soil, (D soil) while a 7 % drop was observed in Elfayoum soil (E soil) at 4 % lime after three days of curing. Both Damanhour and Elfayoum soils are mainly montmorillonite with a minor amount of kaolinite, see figure 2.13. It can be concluded that the shrinkage limit depends primarily on the clay type.

A reduction of swelling potential and swelling pressure are obtained after lime stabilisation of clay. These modified characteristics are attributed to substitution of other cations by calcium. Two main factors are involved in the reduction of swelling, decreased affinity for water of the calcium saturated clay and the formation of cementitious materials which prevent expansion (Mitchell and Hooper, 1961; Ingles and Metcalf, 1972). Mitchell and Hooper (1961) studied the influence of lime on the swelling characteristics of California expansive clay. They reported that swelling was reduced significantly after curing with lime. They found that the addition of 4% hydrated lime to the expansive soil reduced the swelling from 17% to 3%. They also observed that the addition of lime to such clay soil increased the shrinkage limit.

When clays are subjected to water they show intercrystalline swelling (Arabi and Wild, 1989). Water is strongly adsorbed at the negatively charged particle surfaces. Thus, an extensive adsorbed layer is formed due to the concentration gradient between the bulk solution and the electrical double layer (consisting of water molecules and exchangeable cations). The addition of lime modifies the electrical double layer, reducing the thickness of the adsorbed water layer and thus reducing the swelling capacity.

2.9.4 EFFECT OF LIME ON COMPACTION CHARACTERISTICS

The compaction characteristics of lime stabilised soils (i.e. maximum dry unit weight and optimum moisture content) have been extensively studied. Many investigators have stated that the addition of lime to a clay soil causes two major changes in their compaction characteristics. Firstly, a decrease in maximum dry unit weight, and secondly, an increase in the optimum moisture content (Andrews, 1966; Johnson, 1948; Lund and Ramsey, 1959). The reduction in maximum dry density and the increase in optimum moisture content are principally attributed to flocculation and agglomeration (Rogers, 1988; Cobbe, 1988). Also, the clay replacement with lime contributes to the reduction in maximum dry density because clay has a higher density than lime (Abdi, 1992). Lu *et al.*, (1957) concluded that the lime type affects the degree to which the compaction characteristics of a soil are changed by lime addition. They found that dolomitic lime decreases the maximum dry unit weight to a lesser degree than hydrated lime.

Lees *et al.*, (1982 a) studied the compaction characteristics of three types of soil containing 10, 30, and 50% of clay and the rest of the compositions are sand. They found that at 30 % and 50 % clay content, the addition of lime results in a 5 % drop in the maximum dry density. They pointed out an increase in the optimum moisture content with the increase of lime content, see figure 2.14. However, they reported that the addition of lime to 10% clay soil resulted in a noticeable increase in the maximum dry density indicating that at this stage the added lime is readily accommodated in the available voids of the granular fraction

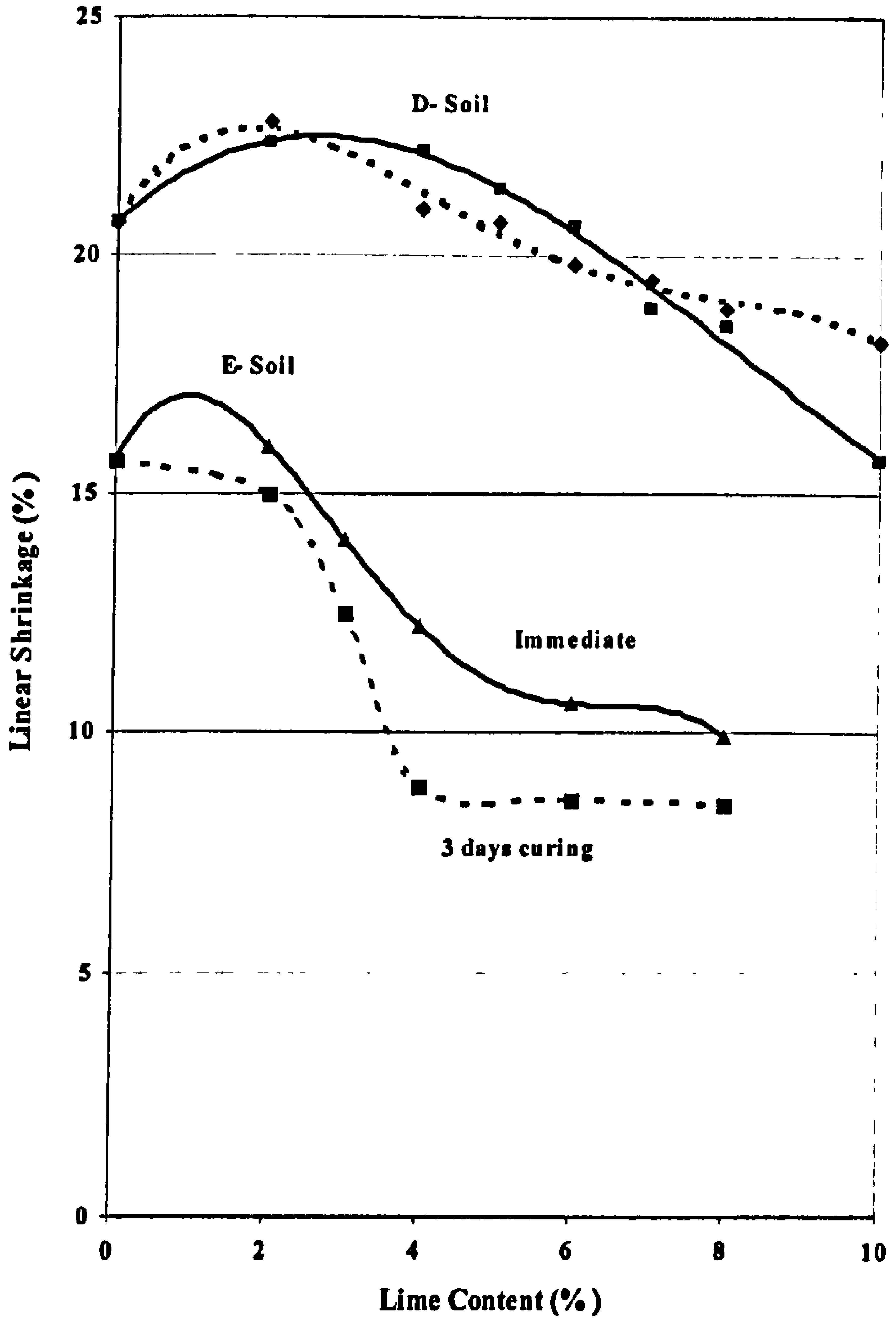

Figure 2.13 Effect of lime content on linear shrinkage, Abdelkader and Hamdani, 1985

Andrews and O'Flaherty (1968) concluded that the decrease in density was dependent not only upon the lime percentage, but also on the clay content and the clay minerals. Thus, the optimum moisture content increases with increasing clay fraction as the specific surface area increases and thus needs more water for lubrication. Also, the maximum dry density decreases as a result of the difference in density between soil and lime.

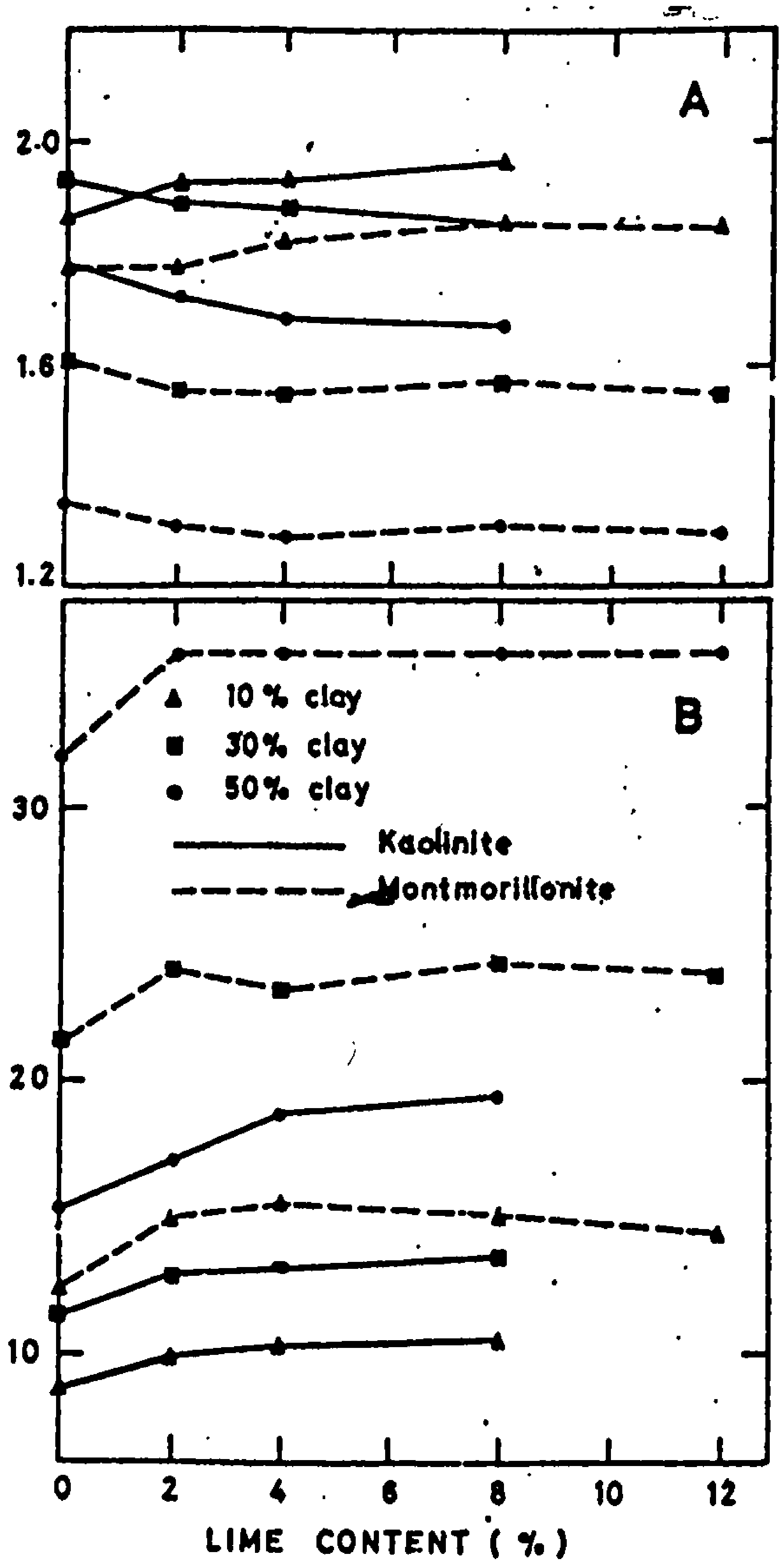

Figure 2.14 Effect of clay type and content on maximum dry density and optimum moisture content of lime treated soils, Lees *et al.*, 1982 a
A= Maximum dry density, B= Optimum moisture content

In lime stabilised kaolinite soils, higher densities are obtained, than for other expandable soils (i.e. montmorillonite). Croft (1964) reported that this was due to the greater ability of expandable clays to adsorb water than kaolinite. The rapid formation of cementitous products could cause the dry density reduction (Herzog and Mitchell, 1963). Mitchell and Hooper (1961) also confirmed the effect of the formation of cementitous products on the maximum dry density. They reported that the time delay between mixing and compaction could cause an increase in the optimum moisture content and a decrease in the maximum dry density. These changes in the compaction characteristics due to a time delay may be attributed to flocculation, carbonation and the pozzolanic reaction. The cementitious particles before compaction may provide greater resistance to particle rearrangement and compaction, and will further reduce the maximum dry density. It was found that the time interval between mixing and compaction could have a significant effect on the properties of the treated soil. For example, 24 hours delay between mixing and compaction led to a significant decrease in density and 30 % decrease in strength compared to the samples compacted immediately after mixing.

Arabi (1987) found a continous decrease in the final strength of cured lime clay as the time between mixing and compaction increased. The decrease in strength, it is believed, is due to the decrease in the maximum dry density as the time between mixing and compaction causes the pozzolanic reaction to start and then the cementitious materials, which cause the mixture to be more difficult to compact, are formed.

Mateos (1964) found that the addition of lime to montmorillonitic clay soils affects the shape of the moisture- density compaction curve so that a well-defined maximum density was not shown. He stated that the optimum moisture content for compaction should be that giving maximum strength. Normally, the moisture content for the maximum strength is located on the dry side of the compaction curve.

Abdelkader and Hamdani (1985) studied the effect of lime on the compaction characteristics of two Egyptian clay samples. They found significant changes in the MDD and OMC, see figure 2.15. The E soil showed larger reduction in MDD and smaller increase in OMC than did the D soil. Such behaviour is also typical of the lime-soil mixtures; an increase in OMC results from the additional moisture needed for hydration of calcium cations, and for immediate reaction with clay minerals. The decrease in MDD can be the result of flocculation of the soil particles (Herrin and Mitchell, 1961) or the formation of cementitious products in immediate reactions with the clay fraction (Diamond and Kinter, 1965). This causes resistance to densification and thus lowers density result.

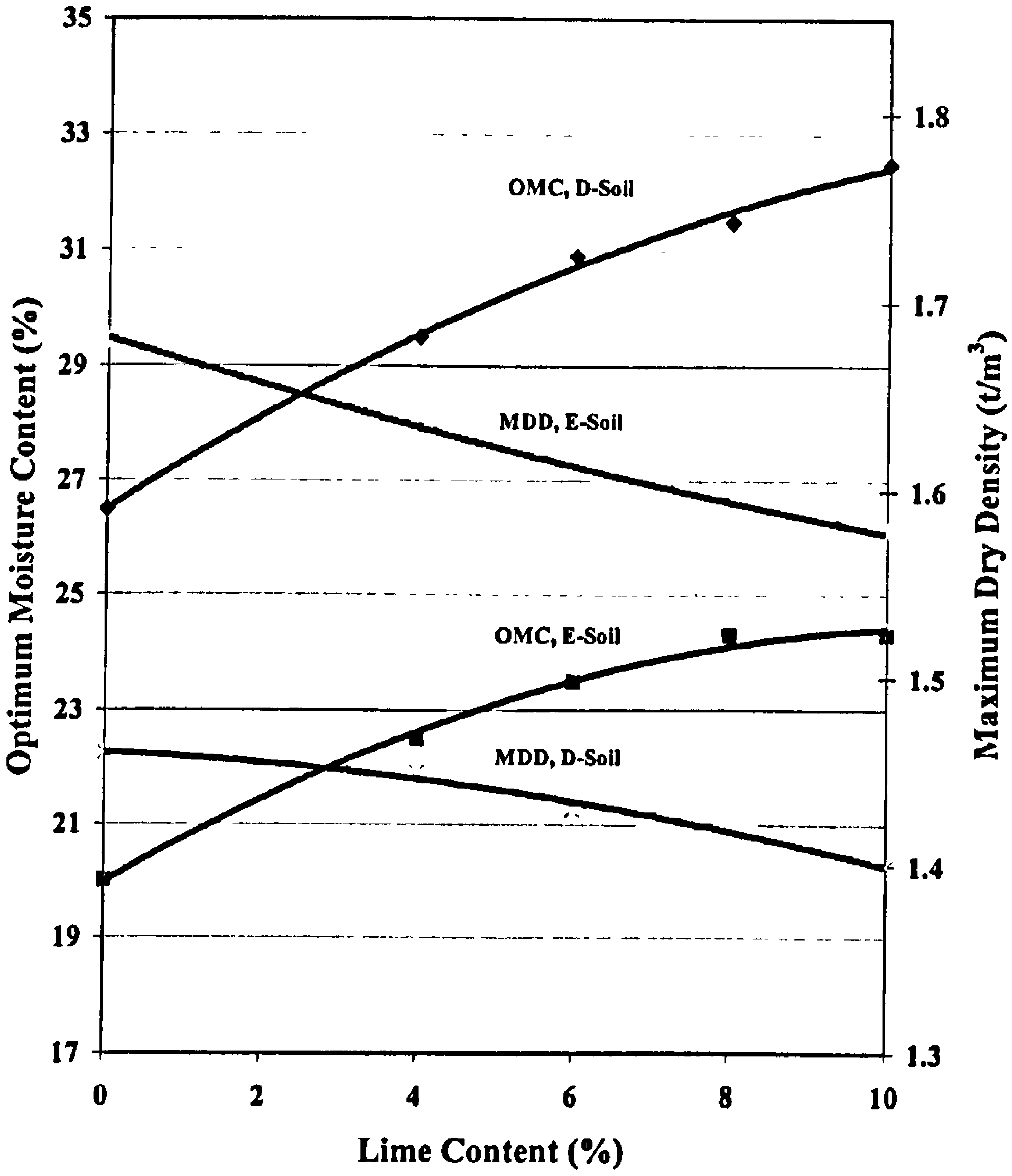

Figure 2.15 Effect of lime content on compaction characteristics, Abdelkader and Hamdani, 1985

Abdi and Wild (1993) studied the effect of lime percentage on the compaction characteristics of kaolinite clay. The MDD decreased with an increase in the lime content, while the OMC increased with an increase in the lime content, see figures 2.16 and 2.17.

Mohamed *et al.*, (1991) studied the effect of adding lime and cement on the compaction properties of three types of natural Egyptian soils (sand, sandy clay and clay). These three types of soils were brought from sites near Alexandria, Egypt. They found that both lime and cement caused a reduction in the maximum dry density and an increase in the optimum moisture content in all types of soils. They found that these changes were attributed to the increase in fines content as a result of the treatment process.

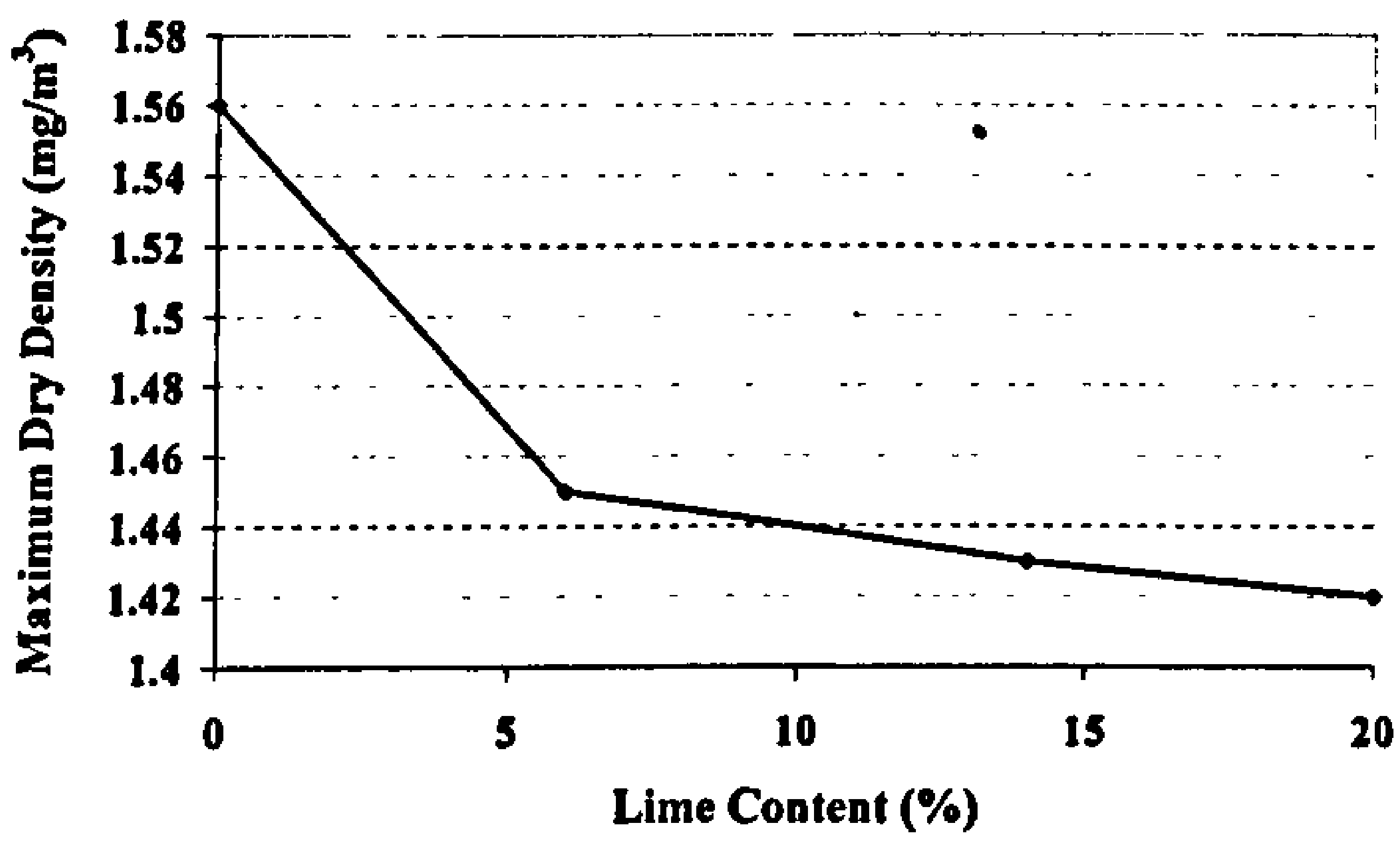

Figure 2.16 Maximum dry density vs. lime content for compacted kaolinite-lime-mixes, Abdi and Wild, 1993

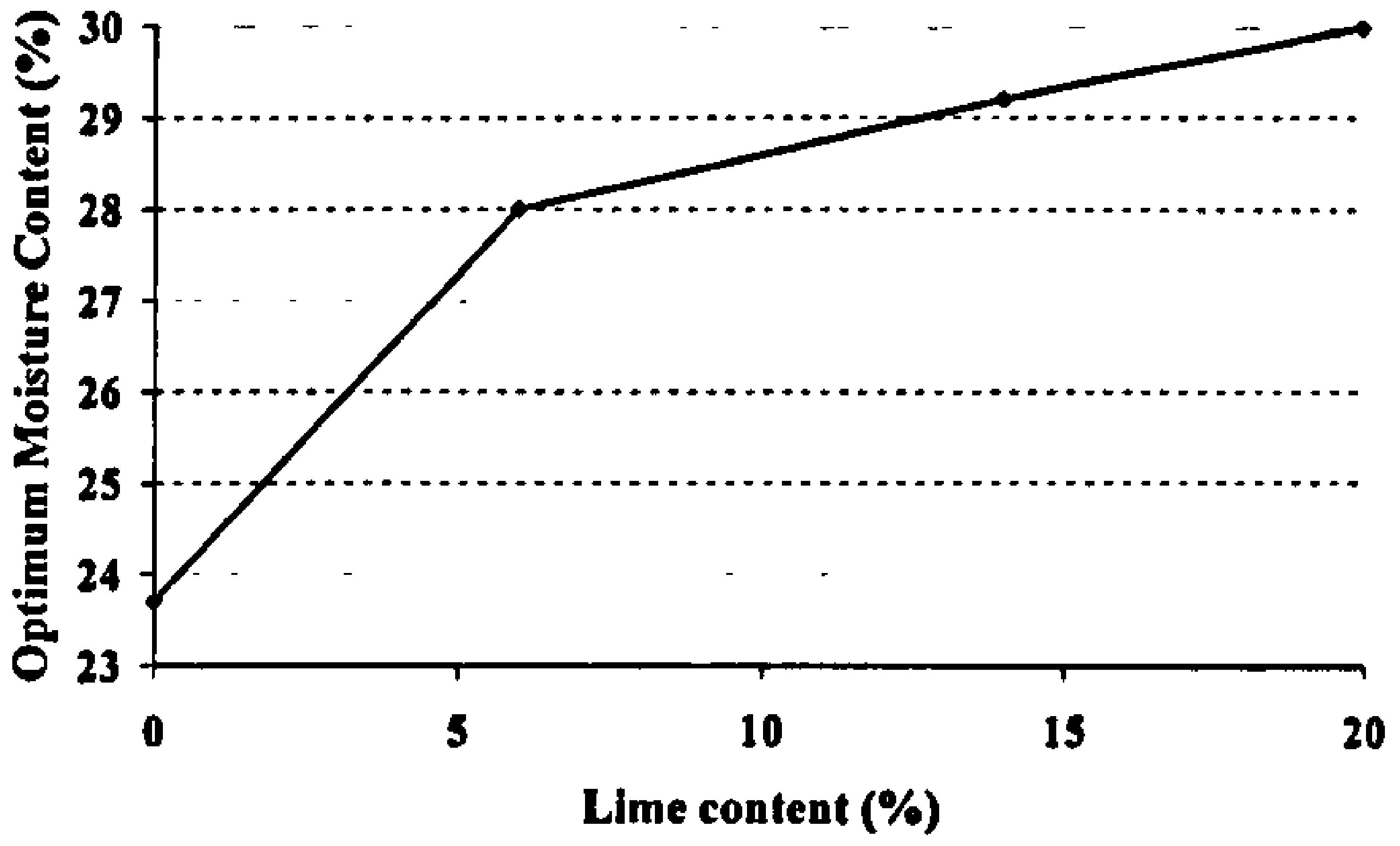

Figure 2.17 Optimum moisture content vs. lime content for compacted kaolinite-lime-mixes, Abdi and Wild, 1993

Unlike other researchers, Jan and Walker (1963) suggested that when lime was added to a soil, no significant reduction in maximum dry density is shown. However, a slight decrease is shown in the optimum moisture content with the addition of lime, with only one value contradicting this trend. The insignificant reduction in the maximum dry density, is thought to be due to the soil having a low clay content that gives a limited flocculation and agglomeration in the mixture. Therefore, no significant reduction in the maximum dry density is expected.

Elsekelly (1987) studied the effect of adding lime and salt on the compaction characteristics of an artificial test soil similar to the Egyptian clayey soil. He concluded that the addition of 2 % lime to salt-soil mixtures resulted in a decrease in the maximum dry density and an increase in the optimum moisture content.

In summary, the addition of lime to clayey soils decreases the MDD, and increases the OMC with an increase in the lime content. The decrease in the MDD is a result of flocculation of the clay particles while the increase in the OMC, is thought to be a result of the additional moisture needed for the hydration of calcium cations, flocculation and for the formation of cementitious products due to an immediate reaction with the clay fraction

2.9.5 EFFECT OF LIME ON STRENGTH CHARACTERISTICS OF CLAY SOILS

The strength of lime-treated soil mixtures has been extensively studied. It has been found that the strength increases and that the gain is influenced by several factors such as clay type, lime type and content, curing period and conditions and the time elapsed between mixing and compaction and chemical additives (Ingles and Metcalf, 1972; Bell and Coulthard, 1990).

1) Type of clay

The type of clay mineral present in a soil has been found to have an important effect on the strength properties as each type of clay has different mineralogy which affects the reaction products. All types of clay minerals are attacked by lime, those having three layer (montmorillonite) are more effective than two-layer clay minerals (kaolinite). For example, the reaction of lime with montmorillonite is quicker than kaolinite clays, although the final strength achieved is greater in kaolinite clays (Bell, 1988 a).

However, Mateos (1964) pointed out that montmorillonite and kaolinite clays achieve higher strength development when mixed with lime than illite or chlorites. Croft (1964) and Croft and Nettleton (1964) confirmed that the clay mineralogy has a major effect on the ultimate strength. Eades and Grim (1960) studied the strength development of lime curing on pure clay minerals. They found an increase in strength for kaolinite, illite, and montmorillonite with increasing lime content. They also pointed out that kaolinite produces a greater strength increase than does either illite or montmorillonite. Bell and Tyrer (1987) reported that although the initial strength of the expansive clays is rapidly increased the final strength achieved is greatest for kaolinite clay. Many other investigators confirmed that montmorillonite and kaolinite react better with lime than illite and chlorite clay soils (Thompson and Harty, 1973; Hilt and Davidson, 1960; Bell, 1996).

In contrast, Lees *et al.*, (1982 a) studied the effect of lime on the strength of artificial soils composed of sand, kaolinite and a calcium based montmorillonite. Lime treatment increases the UCS of both types of sandy clay soils investigated.

Montmorillonitic soil achieved a higher strength gain than kaolinite. In general, the majority of the strength increase due to lime addition is obtained at 2% lime content for the kaolinitic and 8% for the montmorillonitic soil (see figure 2.18). Lime treatment also significantly increased the elastic modulus of montmorillonitic soils, With 4% lime and after 28 days of curing, values of the elastic modulus for specimens with 10, 30, and 50% [1]of montmorillonite clay reached 32.0, 49.0, and 41.5 MN/m^2 respectively (an increase of 237, 145 and 232% over corresponding values before treatment). Little or no further increase in the elastic modulus occurred for lime contents higher than 2%; modulus values at this level (2% of lime) reached 15.5 and 14.1 MN/m^2 for specimens with 30 and 50% clay respectively (an increase of 101 and 182% over corresponding values before treatment).

Ahmed (1988) found that addition of lime to soil improves its strength. He also observed that lime is only active in soils containing an appreciable amount of clay. In contrast, in some cases, only a small amount of clay is needed in a soil for reaction with lime to be effective as the amount of silicate or alumina required to sustain a pozzolanic reaction in soils is relatively small (Bell and Coulthard, 1990).

Grim (1968) reported that organic matter has a high cation exchange capacity which directly affects the pozzolanic reaction. Arman and Munfakh (1972) reported that previous researchers explained that when lime is added to a soil that contains organic matter, some of the Ca^{2+} ions are used to satisfy the exchange capacity of the organic matter thus reducing the number of calcium ions available for the pozzolanic reaction. In their investigation they did not show any retardation of the pozzolanic reaction. In contrast (Sherwood, 1962; Rogers, 1988) observed that soil containing organic matter does not react sufficiently with either lime or cement as organic matter retards the normal reaction between lime and soil by decreasing the pH of the soil. Sherwood (1962) suggested that the type, rather than the total amount of organic matter is the critical factor retarding the reaction.

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[1] K_{10} contains 10% kaolinite and 90% building sand.

M_{10} contains 10% montmorillonite and 90% building sand.

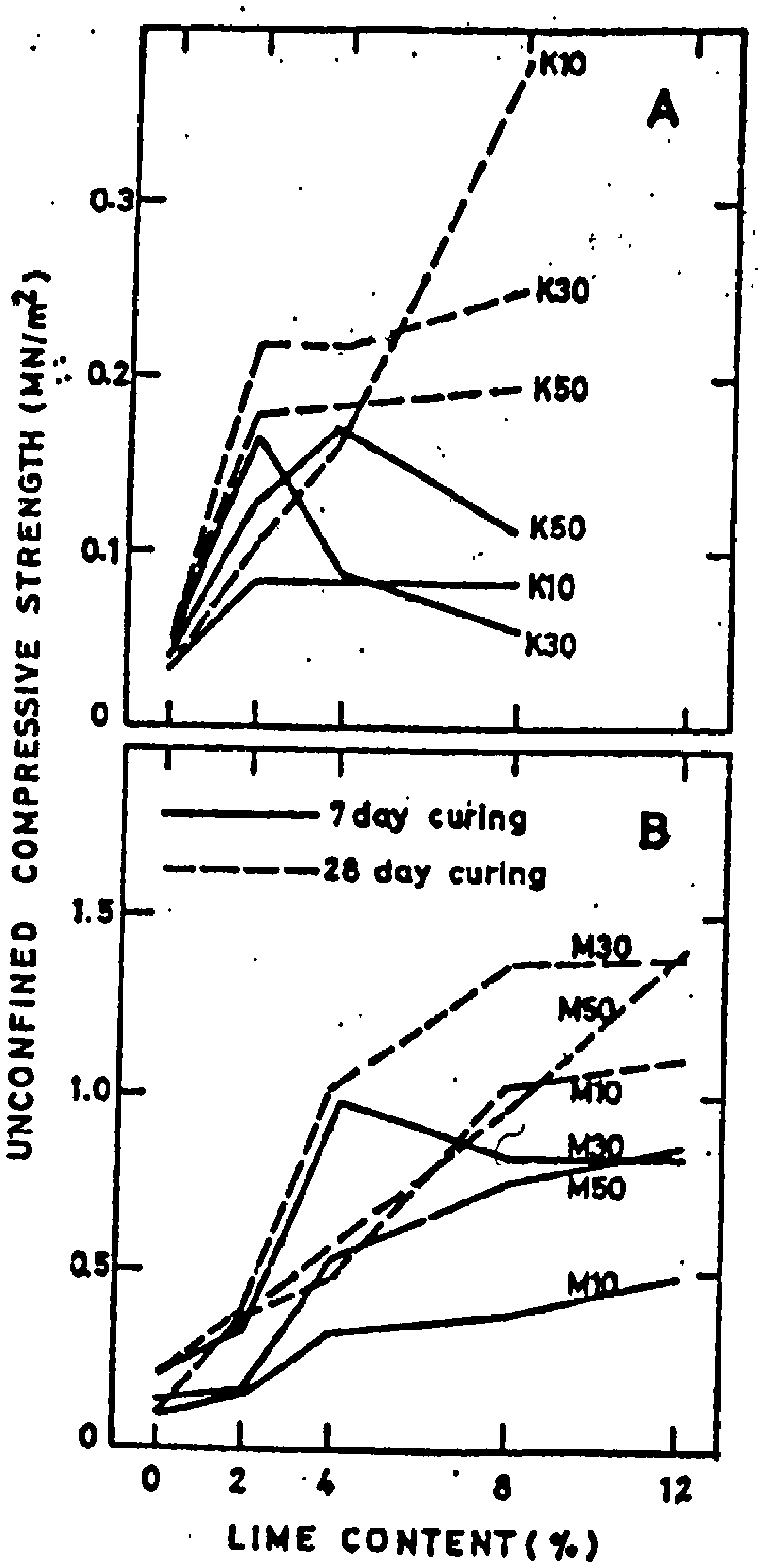


Figure 2.18 Effect of clay type and content on unconfined compressive strength of lime treated soils, Lees *et al.*, 1982 a

2) Type of lime

Quicklime is the most frequently used lime product for lime stabilisation in Europe. However, hydrated lime is used more often than quicklime in the United States. Generally, quicklime seems to be a more effective stabiliser than hydrated lime (Bell, 1988 a). Ingles and Metcalf (1972) found that montmorillonite clays produce lower strength when mixed with dolomite limes rather than with high calcium limes. Kaolinite clays achieved the greatest strength when mixed with semi-hydraulic lime, and the lowest strength with high calcium limes (Bell and Coulthard, 1990). Wang *et al.,* (1963) studied the effect of lime type on clayey soil strength. Dolomitic monohydrate lime was found to be more effective in increasing soil strength on curing followed by high calcium hydraulic and high calcium hydrated limes respectively. For low addition levels (up to about 5%) and for curing periods of 28 days or more, some of the limes were as effective as cement in improving the strength of the soil tested.

3) Lime content

Generally, lime addition to clayey soil increases its strength to a certain limit, and the addition of excess lime tends to decrease strength (Bell, 1996). The optimum lime content tends to range between 4 % and 8% with higher replacement values required in soils with higher clay fractions, see figures 2.19 and 2.20. Lime fixation, as proposed by Hilt and Davidson (1960), is between 2 and 4% lime. However, additional lime is required beyond the lime fixation point to produce a significant strength development due to the pozzolanic reaction. For economic reasons it is necessary to use the minimum amount of lime which achieves the required strength. To determine the appropriate amount of lime required for lime stabilisation, the pH quick test is widely used. This method is based on measuring the pH of soil–lime solutions containing different amounts of lime after 1 hour of reaction at room temperature. When lime is added to a soil the pH of the soil–lime mixture increases to a maximum of 12.4, (the pH of lime-saturated water). Any further increase in the lime content does not increase the pH value above this value. The optimum lime content for that particular soil is the amount required to achieve the pH value of 12.4 (Allan *et al.*, 1977). It is recommended to use only the optimum amount of lime to achieve the required strength for the project under study. It is believed that the optimum value of lime, which achieves the pH value of 12.4, varies from 4% to 8% by dry weight of soil.

However, Rosen and Marks (1974) pointed out that sometimes the lime content required to produce the desired strength is greater than that needed to obtain the maximum pH value. Therefore, it has been suggested that the most reliable method to evaluate the required amount of lime is to use the pH test to estimate the approximate lime content, and then prepare samples with a range of lime around this approximate level and test them. Abdelkader and Hamdani (1985) concluded that the unconfined compression strength increased from 200 kPa before treatment to 1240 and 2067 kPa after 28 days curing for 4% and 8% lime content for the E and D soils (section 2.9.3) respectively, see figure 2.21. Sodium chloride was added with the lime to both soils and a further increase in strength of about 276 kPa was produced in D soil. The use of salt is therefore recommended for lime stabilisation of the Damanhour soil but not for the Elfayoum soil.

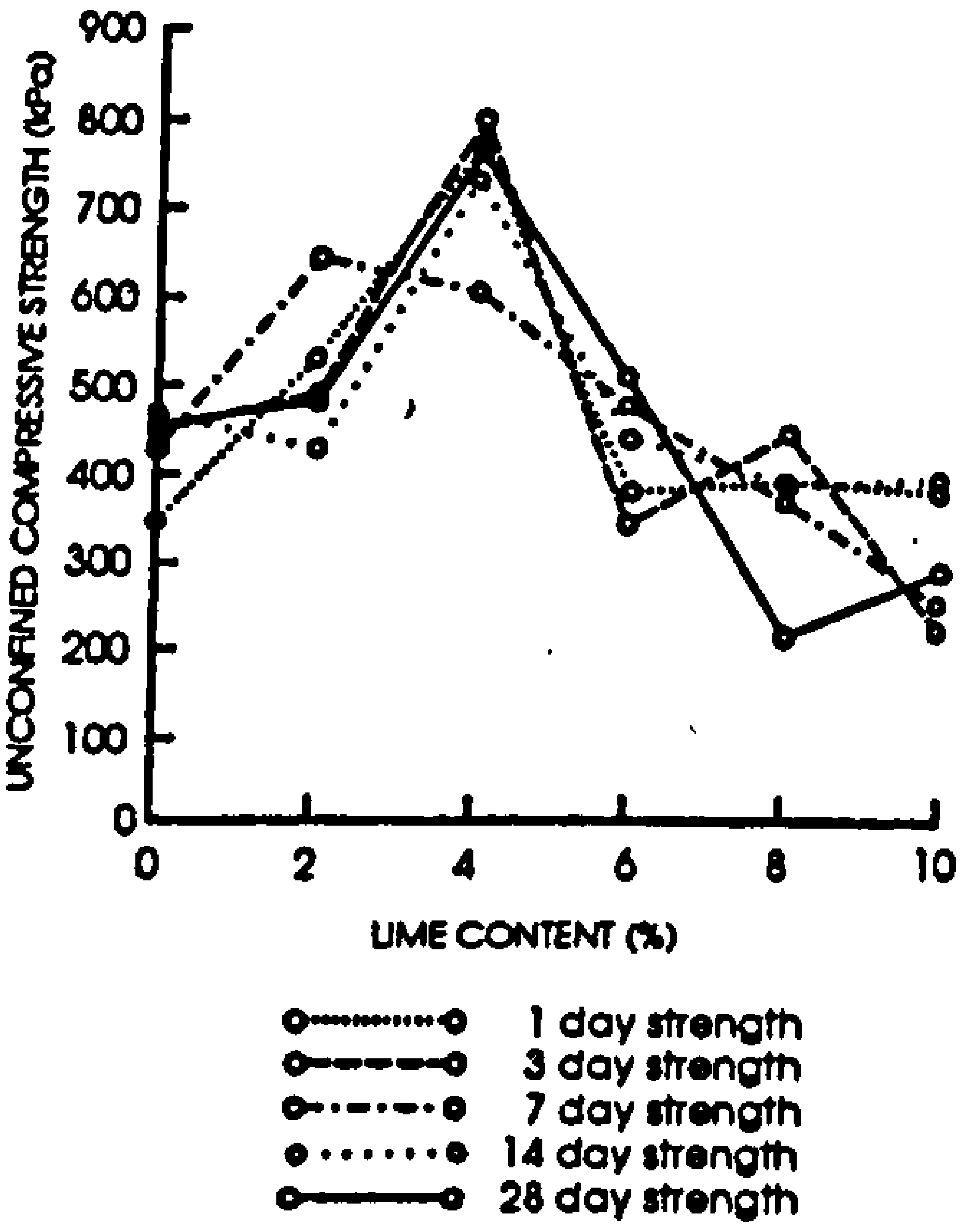


Figure.2.19 Unconfined compressive strength of montmorillonite with various 2 additions of lime, Bell, 1966

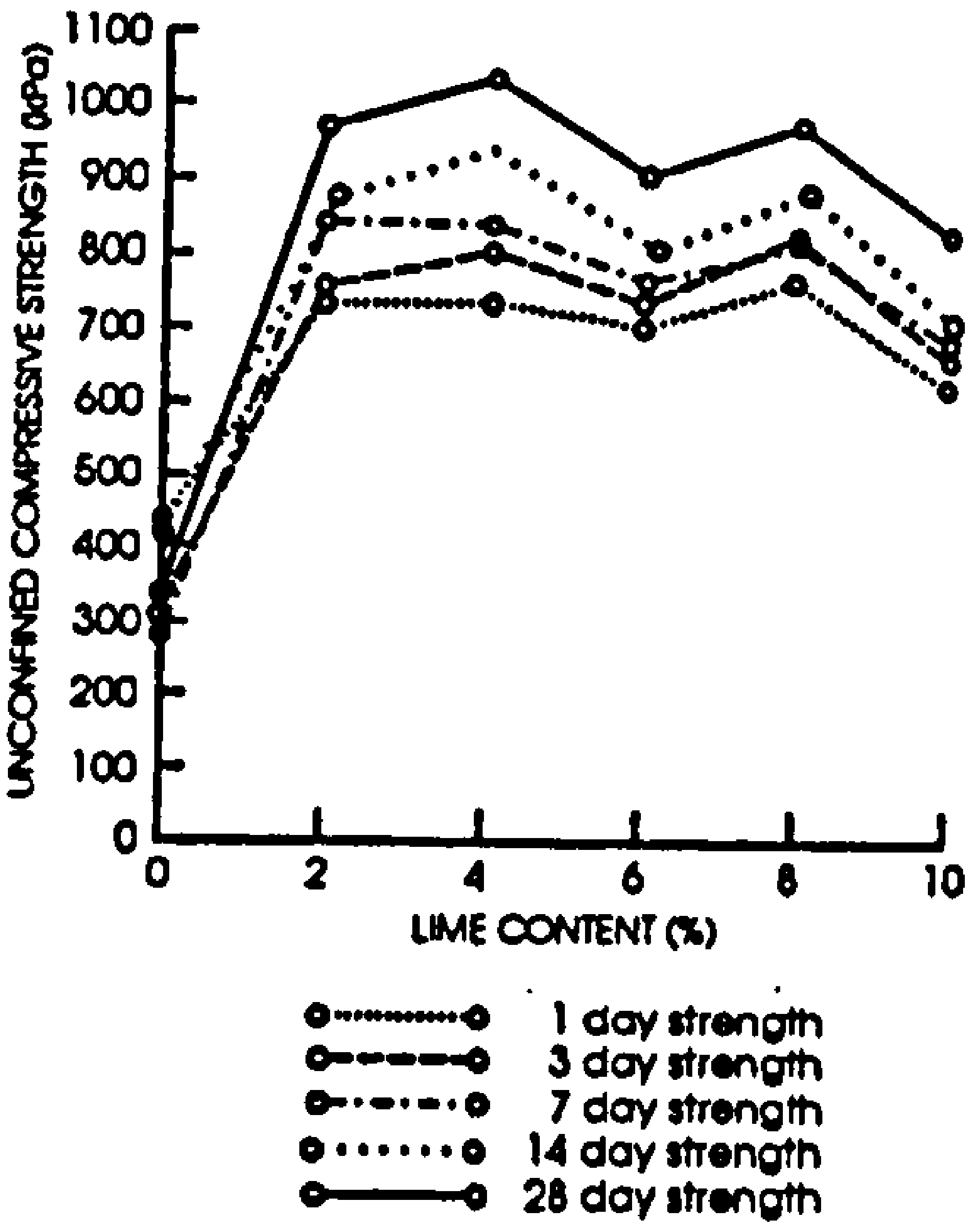


Figure 2.20 Unconfined compressive strength of kaolinite with various additions of lime, Bell, 1996

Howeedy and Zedan (1991) also studied the effect of the lime and flyash on the UCS of Egyptian soil. Table 2.7 and 2.8 summarise the effect of flyash and lime-flyash on the unconfined strength of soil.

Howeedy and Zedan (1991) defined reactivity as "the difference between maximum unconfined compressive strength of the stabilised material and that of the raw soils". In conclusion, they found that the strength of the soil increased with an increase in the amount of flyash and curing time. The addition of 10% flyash increased the strength of soil 23% and 123% after 7 and 14 days curing time, respectively. Also, the strength of the soil increased with an increase in the percentage of lime-flyash and curing time. The addition of 6% lime and 6% flyash increased the strength of the soil mixture 153% and 353% after 7 and 14 days curing time, respectively.

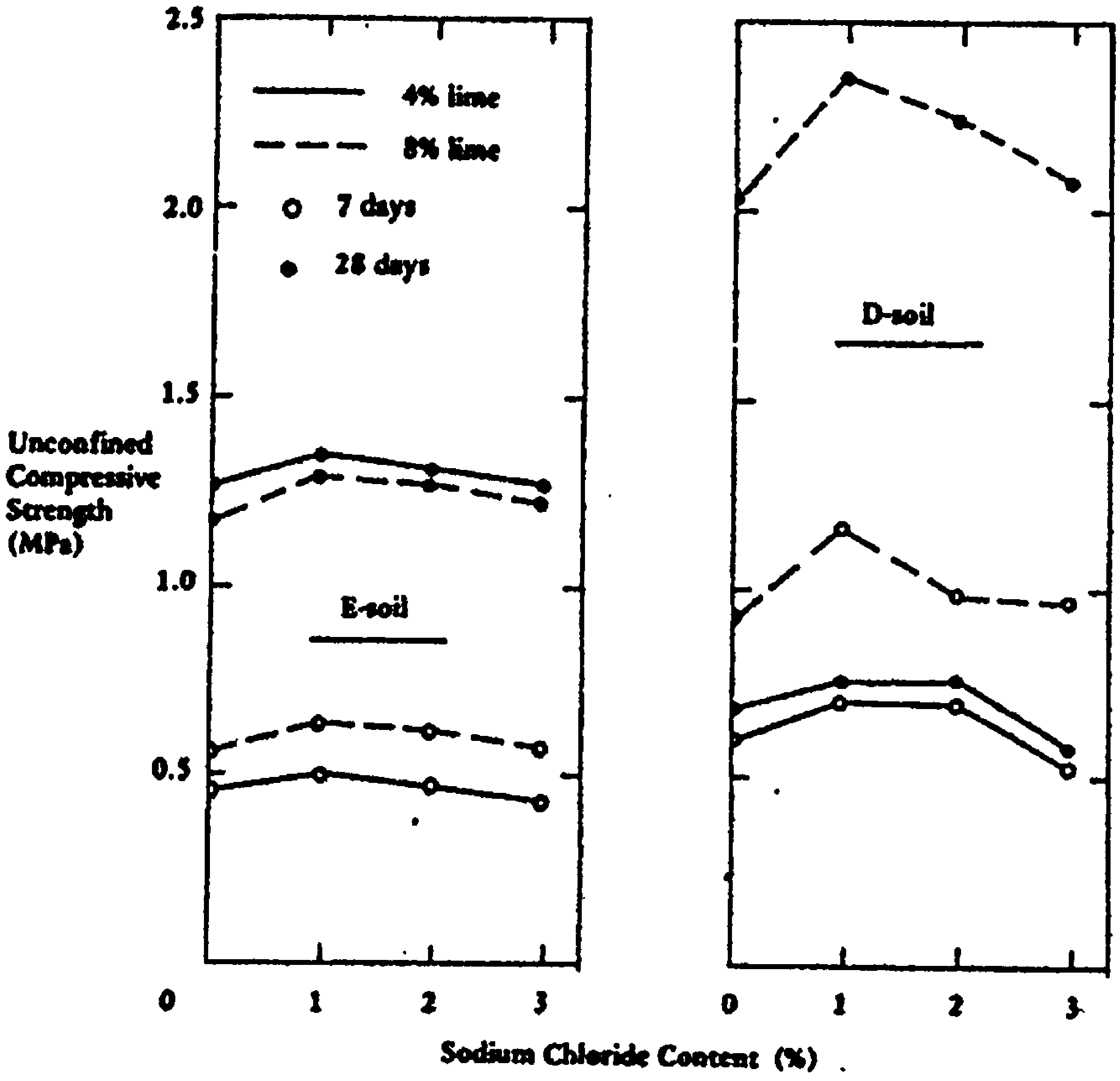


Figure 2.21 Effect of sodium chloride on the UCS of the lime treated soils, Abdelkader and Hamdani, 1985

Table 2.7 Unconfined compressive strength test results for soil and flyash-soil mixture, Howeedy and Zedan, 1991

Percentage of flyash	Unconfined compressive strength, psi	
	7 days curing	14 days curing
Soil	42.84	42.84
Soil + 4% flyash	49.89	80.45
Soil + 6% flyash	50.89	87.99
Soil + 10% flyash	52.62	95.50
Soil + 15% flyash	49.17	82.37
Flyash reactivity, psi	9.78	52.66
Percentage of reactivity	23%	123%

Table 2.8 Unconfined compressive strength test results for soil and lime-flyash-soil mixture, Howeedy and Zedan, 1991

Percentage of flyash	Unconfined compressive strength, psi	
	7 days curing	14 days curing
Soil	42.84	42.84
Soil + 2: 2 lime: flyash ratio	45.90	69.00
Soil + 4:4 lime: flyash ratio	93.00	128.46
Soil + 6:6 lime: flyash ratio	108.50	194.14
Soil + 8:8 lime: flyash ratio	105.29	152.00
Soil + 10:10 lime: flyash ratio	82.60	142.43
Flyash reactivity, psi	65.66	151.30
Percentage of reactivity	153%	353%

4) Curing conditions

Curing conditions (i.e. temperature, time and relative humidity) are of great importance in influencing strength increase and the final strength of soil mixtures. Laguros *et al.*, (1956) pointed out that the pozzolanic reaction rate increases with increasing temperature. Higher curing temperatures accelerate the reaction and result in higher early strength gain (Bell, 1988 a and b; Bell, 1996). This has been confirmed by many researchers including Marks and Halliburton (1972); Al-Rawi, (1981); Wild *et al.*, (1987). Mateos (1964) found that clay specimens cured at 35°C produced twice the strength of those cured at 25°C however, Thompson (1970) reported that no pozzolanic reaction can take place below 4°C because this low temperature retards the strength development.

Sabry and Parcher (1979) studied the effect of curing conditions on the unconfined compressive strength of clay soils. They reported that soils compacted at a moisture content above the optimum attain a higher strength after a short curing period than that obtained with lower compaction moisture contents. They suggested that the strength increase is due to the more uniform diffusion of lime and a more homogenous curing environment. They found also that the strength of soil can be improved by adding water after compaction and that the organic content, which was less than 1.5%, did not affect the soil lime reaction. It is unlikely that adding water after compaction would have a significant increase in strength as this water will not be distributed uniformly throughout the mixture and thus its effect on the pozzolanic reaction may be limited.

Anand *et al.*, (1996) tested Louisiana silty clay soil as a sub-grade for unconfined compression and the California bearing ratio (CBR) at five different moisture contents and dry density levels. They found that lime treatment increased the UCS, which was found to be directly proportional to the curing period.

Drake and Halliburton (1970) studied the effect of elevated curing temperature and reduced curing time on the curing of 2 cohesive Oklahoma soils. These 2 soils were treated with lime for 28 days at 80° F (26.6° C) and 100 % humidity. They found that 30-72 hours curing at 105°F (40°C) achieved the equivalent strength of 28 day old samples. Comparison of DTA data for the accelerated-cured and moist room cured samples indicated that both types of curing produced similar mineralogical products. The only concern about the accelerating of the curing process, it is thought, is that the pore water suction might increase, if the specimens are not entirely sealed, at the very high curing temperature. It is not recommended to use any acceleration for curing except in some special circumstances for the above reason and also due the difficulty of applying such a high temperature in the field.

Stabilised soils can be used as sub-grades, sub-bases or bases, so the UCS and CBR should be evaluated as appropriate. The soaking condition is considered to be one of the worst conditions to which a stabilised soil may be subjected. Stabilised soils, subjected to soaking for 24 hours prior to testing for compression strength lose strength. This strength loss is between 10 and 60% (Andrew, 1966). Al-Rawi and Awad (1981) found that increased curing temperatures produced a significant reduction in the loss in the UCS due to soaking. This confirms the suggestion above that the pore water suction affects the strength. When a sample is subjected to soaking it loses a great part of the strength increase due to suction.

5) Time elapsed between mixing and compaction

Mitchell and Hooper (1961) studied the effect of the time elapsed between mixing and compaction on the final soil strength. They reported that a delay of 24 hours between mixing and compaction led to as much as a 30% decrease in the as-cured strength compared to the values for samples compacted immediately after mixing. All samples were cured for 215 days in a moist atmosphere prior to testing.

Holt and Freer-Herwish (1998) studied the effect of mellowing period on the compaction characteristics of London and Oxford clays mixed with lime. They observed a further increase in the OMC and a further decrease in the MDD with increasing the mellowing duration.

6) Effect of additives

Lees *et al.,* (1982 b) studied the effect of adding sodium chloride on the strength of lime-soil mixes. They used the same soil mixture as reported in their previous paper (Lees *et al.*, 1982 a). They reported that an addition of sodium chloride to lime soil mixtures did not accelerate the development of the UCS of lime-kaolinite soil mixtures. The increase in UCS of kaolinite lime mixtures after 28 days curing is higher than that of montmorillonite lime mixtures. These strength gains were considerable, for soil with medium to high clay content, to between 100 and 300% increase in the UCS over that of untreated soils or treated with lime only. In most cases the maximum UCS gain is obtained with 1% of sodium chloride, see figures 2.22 and 2.23. They suggested that the strength increase was attributed to the formation of new materials. Abdelkader (1981) identified, using scanning electron microscopy and X-ray diffraction, that cementitious materials were formed when adding only lime and these were different from those formed when adding sodium chloride and lime.

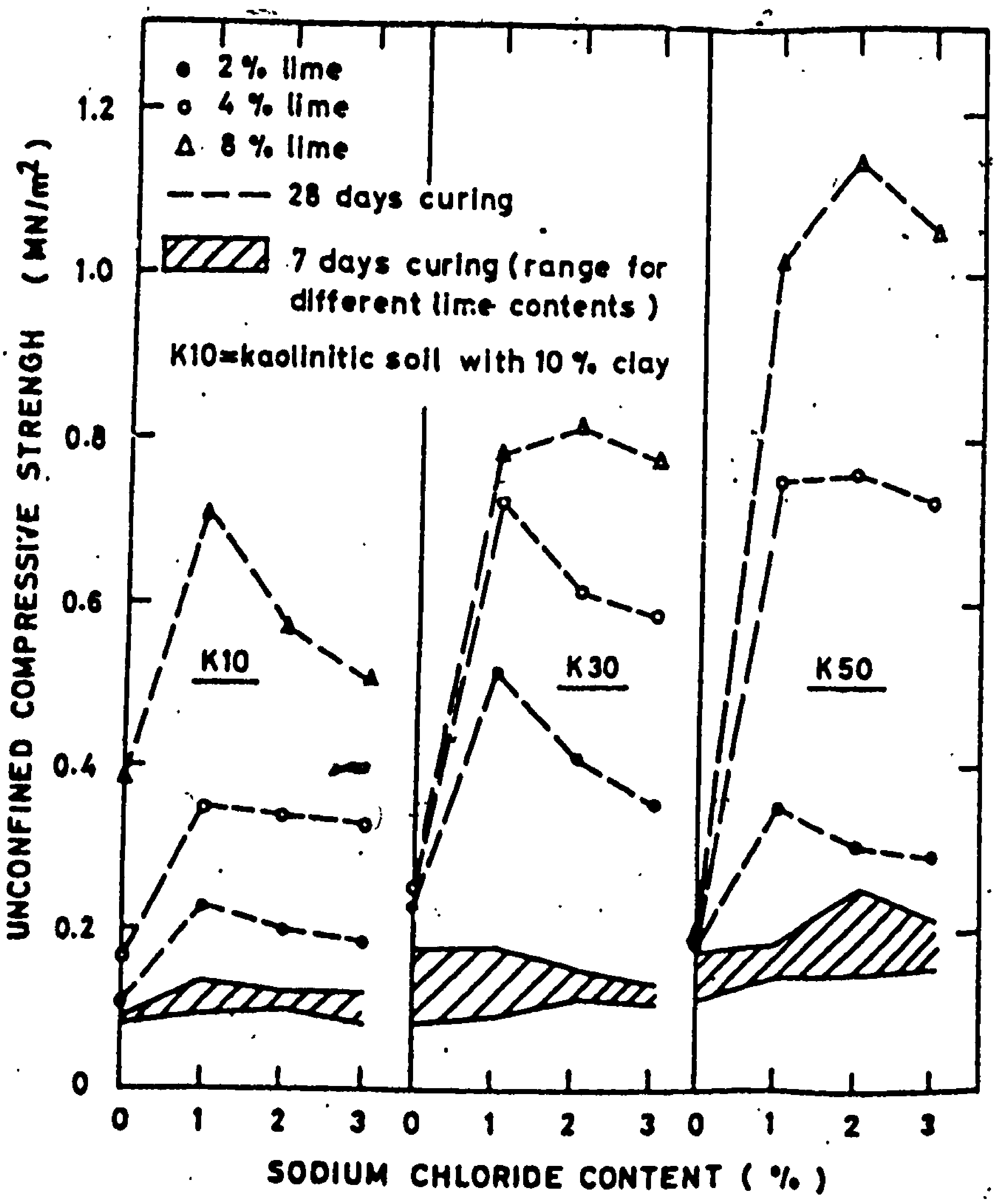


Figure 2.22 Effect of sodium chloride on unconfined compressive strength of lime treated Kaolinitic soils, after Lees *et al.*, 1982a

Davidson *et al.*, (1960) studied the use of chemical additives to improve the lime stabilisation of montmorillonite soils. Three Iowa soils ranging in clay content from 35 to 74% were studied in combination with varying amounts of lime and three inorganic components, sodium phosphate, sodium carbonate and sodium hydroxide.

Sodium hydroxide was found to be a suitable agent to accelerate the hardening mechanism. The optimum sodium hydroxide content is about 1 to 2% depending on the dry weight of the soil component. Sodium carbonate and sodium phosphate are not as effective as sodium hydroxide for improving lime stabilisation of montmorillonitic clay soil.

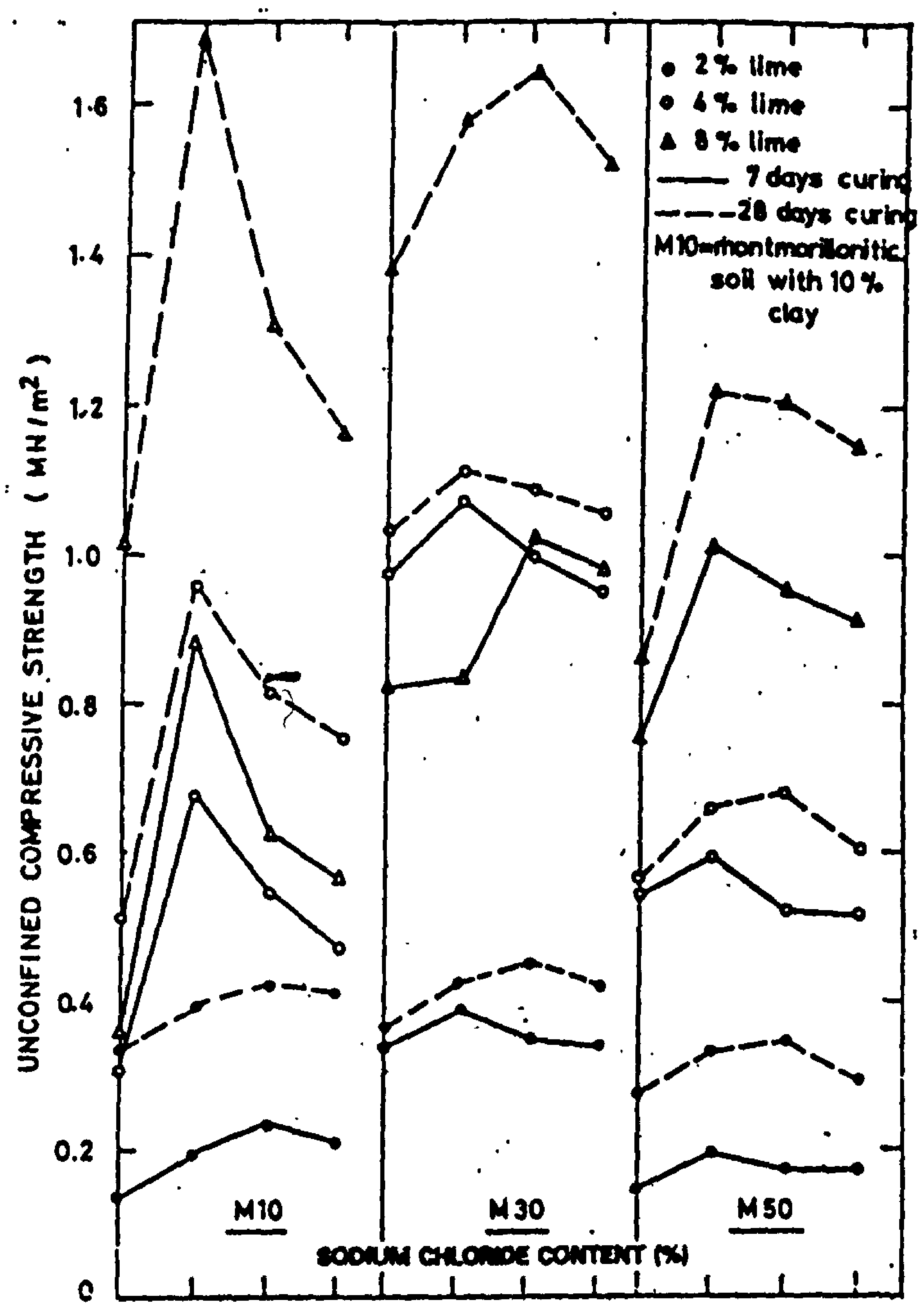


Figure 2.23 Effect of sodium chloride on unconfined compressive strength of lime treated montmorillonitic soils, after Lees *et al.*, 1982 a

The addition of lime also produces other effects on soil properties apart from that on the UCS. Elsekelly (1987) observed an increase in the UCS in all specimens at optimum moisture content (OMC) in clay- lime- sodium chloride mixtures. Also, the addition of lime to clay soils not only increases the compressive strength but also improves other mechanical properties such as tensile strength, shear strength, permeability and CBR (Brandl, 1981, Arabi, 1987; Goldberg and Klein, 1952; Jan and Walker, 1963).

Mohamed *et al.*, (1991) studied the effect of adding lime and cement on the California bearing ratio test (CBR) of the Egyptian soils. They observed an increase in the 5 mm penetration CBR values with an increase in both lime and cement content. They found also that cement is an effective stabilizer for both sand and sandy clay soils and gives high CBR values at low cement content (2%). Also, the maximum CBR value of the clay treated with lime was obtained at the optimum compaction moisture content, whereas cement treatment shows a higher CBR value at a moisture content 2% lower than the optimum compaction value.

2.10 SUMMARY

This chapter presents a detailed literature review on the structure of soil and clay minerals, with special reference to montomorillonite, kaolinite and illite. Expansive soils are discussed, including damages caused by expansive soils, properties of expansive soils and some suggested methods of treatment for the expansive soils are also discussed . Lime stabilisation including soil lime reactions and the reaction products are discussed in detail. The effect of lime on the engineering properties of soil including the plasticity characteristics, volume stability, compaction, and strength are discussed in detail. Effects of type of clay, type of lime, lime content, curing conditions, time elapsed between mixing and compaction, and the effect of other additives in conjunction with lime, on the strength properties are discussed. The addition of lime to the clayey soil increases the strength, decreases the volume change and alters the plasticity characteristics.

In the next chapter soil stabilisation using ground granulated blastfurnace slag (GGBS) will be discussed. Reactions of soil- GGBS-lime systems and the effect of GGBS activated by lime on the properties of clayey soil will be presented.

CHAPTER THREE

REVIEW OF SLAG STABILISATION

3.1 INTRODUCTION

Blast furnace slag is produced as a by-product during the manufacture of iron in a blast furnace. It results from the fusion of a limestone flux with ash from coke and the siliceous and aluminous residue remaining after the reduction and separation of the iron from the ore. Iron blast furnace slag, consists essentially of silicates and alumino-silicates of lime and other bases (Lee, 1974). Molten blast furnace slag has a temperature of 1300-1600°C and is chilled very rapidly to prevent crystallisation. The granulated material thus produced is known as granulated blast furnace slag. It is a latent hydraulic product that can be activated with lime, alkalis, and Portland cement to give hydraulic properties (Gupta and Seehra, 1989). The latent hydraulic properties of blast furnace slag were discovered in Germany in 1862 (Bijen, 1996). Now shortage in cement production has resulted in the need to develop alternative binders to cement for soil stabilisation.

Ground granulated blast furnace slag, GGBS, can be incorporated in cementitious materials to modify and improve certain properties (Nixon and Gaze, 1981), to conserve non-renewable natural resources and to utilise industrial by-products. The possibility of recycling or processing materials to use as partial replacements for cement in concrete, or to stabilise soils, has great economic benefits in all areas of the construction industry (Wild and Tasong, 1999).

Blast furnace slag has a glassy, disordered, crystalline structure which can be seen by microscopic examination which is responsible for producing a cementing effect. GGBS is cementitious on its own. It is a hydraulic material and therefore requires no additives for hydration and hardening to take place other than water if hydrated at elevated temperature and for a long time (Song *et al.*, 2000; Data sheet of North East

Slag Cement Ltd. Nov. 1997 and others). Higgins (1998) observed that GGBS on its own has only mild cementitious properties and in conventional concrete it is used in combination with Portland cement whose alkalinity provides the catalyst to activate the cementitious properties of the GGBS. He also reported that lime (calcium hydroxide) could provide the necessary alkali for activation.

The use of GGBS is well established in many applications where it provides good durability, high resistance to chloride penetration, resistance to sulphate attack and protection against alkali silica reaction (ASR). GGBS has also been used for many years in road bases (Lee, 1974). Its use in soil stabilisation is, however, still a novel process in the U. K, although it has been used in South Africa (Wild *et al.*, 1998). GGBS has also never been used in soil stabilisation in Egypt.

Blending cement with GGBS produces well-established sulphate-resisting properties in concretes. Therefore it is suggested that GGBS may produce similar sulphate-resisting properties in lime-stabilised clays (Wild *et al.*, 1996). The reaction products are similar in both hydrated lime with clay minerals and Portland cements, which are mainly C-S-H, C-A-H and C-A-S-H gels (these phases are defined in section 2.8.5). Since Portland cement hydration products are similar to those of GGBS-Portland cement, then it is possible that those of lime-GGBS-clay are comparable with those of cement /GGBS blends. A review of literature on lime-GGBS-clay mixtures will therefore help in understanding and extrapolating the results to establish the behaviour of lime-GGBS-clay blends (Regourd, 1980; Smolczyk, 1980; Kinuthia, 1997).

3.2 PROCESSING OF BLASTFURNACE SLAG

Processing refers here to the cooling method to which the molten slag is subjected. Three different cooling methods have been used to cool the slag, and the method of cooling determines the physical and chemical properties of the resulting slag material, see figure 3.1 (Lee, 1974).

AIR-COOLED SLAG

In this method the molten slag is slowly solidified by leaving it in air. Usually, the molten slag collected from the furnace is taken to an open-air area where it is allowed to cool down. This produces the highest density of the three types which can be used in road construction after crushing and grading. This slag is used as a road stone or as concrete aggregate.

FOAMED OR EXPANDED SLAG

In this method the liquid slag is exposed to water, at low water/slag volume ratio, which leads to expansion and the formation of a lightweight material. This material is used mainly in concrete blocks and for in-situ lightweight concrete.

RAPID COOLED OR GRANULATED SLAG

This method involves cooling the molten slag at high water/slag volumes using high-pressure water jets. The excess water causes the slag to cool rapidly. This method of cooling produces a granular product. Granulated slag has marked hydraulic-setting properties when ground to a powder and mixed with an alkali agent such as lime. This material is normally mixed with ordinary Portland cement (OPC) to produce Portland blastfurnace cement and super-sulphated cement.

3.3 COMPOSITION AND PROPERTIES OF GGBS

3.3.1 CHEMICAL COMPOSITION

The chemical composition of blastfurnace slag varies depending upon the nature of the ore, the limestone flux and the blastfurnace conditions. The major oxide components of slag are calcium, magnesium (basic oxides), silica and alumina (acidic oxides). Sulphur is also present and small quantities of compounds of iron and manganese (Lee, 1974). However, the rapid quenching in water to produce granulated slag results in the formation of a slag glass consisting of a disordered network of calcium, silicon and aluminium ions bonded with oxygen. Minor components such as sodium, potassium and titanium are also integral parts of this structure (Haynes, 1985). Table 5.5 in chapter 5 shows the chemical composition of GGBS from the Appleby group Ltd., U. K and an Egyptian slag.

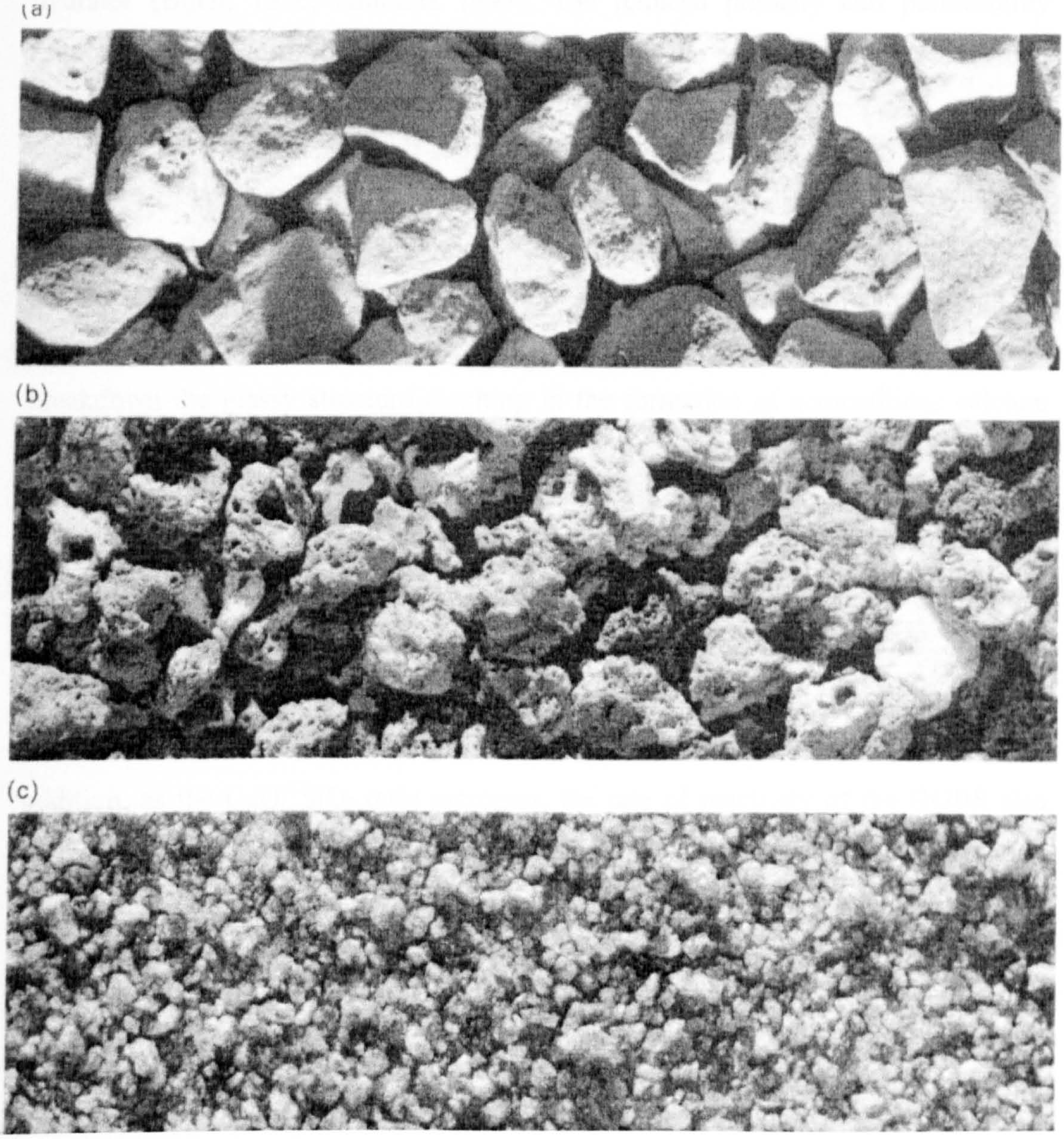


Figure 3.1 Three forms of blastfurnace slag: (a) air-cooled; (b) foamed (or expanded); (c) granulated, Lee, 1974

Portland cement and GGBS are broadly similar in chemical and physical properties. GGBS, used with Portland cement, has been found to produce new properties not normally found in Portland cement. Due to the relatively high silica content in

GGBS compared to Portland cement, there is enhanced C-S-H formation compared to using Portland cement alone. This enhanced C-S-H occupies pore spaces, normally occupied by calcium hydroxide in the hydration of Portland cement, which leads to reduced porosity and permeability of GGBS hydrates compared to cement hydrates (Bijen, 1996; Kinuthia 1997). The reduced porosity and permeability reduce the volume of voids and this, together with the resultant stronger structure, provide resistance to frost damage.

Granulated blast furnace slag has a low reactive potential. Its hydraulic reactivity depends on chemical composition, glass phase content, particle size distribution and surface morphology (ACI, 1989). Activators such as alkalis and sulphates, which are released during the hydration of Portland cement, are able to react with and breakdown the glassy structure resulting in the formation of cementitious calcium silicates and aluminate hydrates.

Various hydraulic parameters have been proposed to relate composition to reactivity; most of these imply an increase in reactivity with increasing CaO, MgO or Al_2O_3 and a decrease with increasing SiO_2. However, BS 6699, British Standard specifications for GGBS for Use with Portland cement (1986), contains a requirement that the $(CaO + MgO + Al_2O_3)/ SiO_2$ should be greater than 1. In addition, as the CaO/ SiO_2 ratio increases, the rate of reactivity of the GGBS also increases up to a limiting point when increasing the CaO content makes granulation to a glass difficult. For optimum hydraulicity the CaO/ SiO_2 ratio would need to be around 1.5. In most applications activation of GGBS is required

3.3.2 PHYSICAL PROPERTIES

Although granulated blastfurnace slag has been used to produce blended cement for a long time, other slags such as that air-cooled are not used (Yamamoto and Makita, 1986). When air-cooled blastfurnace slag is crushed, its physical properties make it suitable as an aggregate. It breaks to give a good cubical shape; it has a rough surface giving good frictional properties and a good hire fire resistance. It has relatively high water absorption, due to its high porosity (Lee, 1974). Recent studies carried out by Mostafa *et al.,* (2001 a, b) confirmed that the air-cooled slag exhibited significant reactivity at room temperature. They suggested that air-cooled slag may be used in the production of low strength building units, or may be used with the granulated slag in the production of blended cement.

3.4 SLAG ACTIVATION

GGBS can be activated in different ways but the most common is chemical activation. In chemical activation, an activator is required and/or an alkali medium. Many activators have been suggested to activate GGBS. The most commonly used activators are calcium hydroxide, calcium sulphate, ordinary Portland cement, sodium hydroxide, sodium carbonate and sodium sulphate (Gjorv, 1989). Wu *et al.*, (1990) suggested that sodium hydroxide, sodium sulphate and potassium aluminium sulphate can be used as activators for GGBS and can help in breaking the Si-O and Al-O bonds. Wild and Tasong (1999) employed lime as an activator in their study in the influence of GGBS in the sulphate resistance of lime-stabilised kaolinite. They observed that the optimum lime/GGBS ratio is 1:5 to activate the GGBS, and to prevent attack caused by excess sulphate solution. Wild and Tasong, also observed that the lime activated GGBS hydration reaction is quicker than the pozzolanic reaction of lime with clay.. Due to its high alumina and silica content, the main reaction products of GGBS activated by lime are C-A-S-H gel and hydrotalcite- type phases containing magnesium.

Portland cement is one of the most commonly used activators. The reaction of GGBS with Portland cement and water is a complex process. Water hydration of Portland cement produces mainly calcium hydroxide $Ca(OH)_2$ and C-S-H gel. In the hydration of blended Portland cement, although minor amounts of alkalis are released, GGBS is mainly activated by the hydration product $Ca(OH)_2$ (Hakkinen, 1993; Bijen, 1996). Thus lime in the form of $Ca(OH)_2$, may be added either as an additive or released from Portland cement hydration. GGBS, due to its high alumina and silica content, produces slightly different hydrates from those formed when using ordinary Portland cement. The main reaction products of GGBS hydration are calcium silicate hydrate, calcium aluminate hydrate and a small amount of calcium hydroxide (Higgins *et al.*, 1998).

Many investigators have reported that the required amount of lime to activate GGBS is low and that higher amounts retarded the activation of alkali activated cement (AAS), (Daimon, 1980). Douglas *et al.*, (1991) observed that 3 % by weight of hydrated lime can retard the setting time of alkali activated GGBS concrete. The GGBS hydraulic reactions are slower than the hydration of Portland cement and have a "pore-blocking" effect which leads to a greater ultimate strength and lower permeability (Maphee *et al.*, 1989). This together with the reduced $Ca(OH)_2$ and other improved binding and absorptive effects, enhances the resistance of GGBS concrete to sulphate attack.

Calcium sulphate is not only a successful activator but also plays an important role as a reactant (Taylor, 1990; Daimon, 1980). A reactant participates significantly in the reaction process while an activator creates an appropriate environment for the reaction process without necessarily playing a significant role in the reaction. To understand how the addition of GGBS alters the soil properties, GGBS hydration should be studied in some detail.

3.5 SLAG HYDRATION

Many authors have explained the clay-lime reaction system as the principal reaction in clayey soil stabilisation. They observed that the main reaction products in clay lime reaction in general are calcium silicate hydrate (C-S-H) with a low Ca/Si ratio, crystalline calcium aluminate phases such as C_3AH_6 and C_4AH_{13} and calcium silicate aluminate C-A-S-H phases (C_2ASH_8), (Croft 1964; Diamond *et al.*, 1964; Wild *et al.*, 1989). In cement terminology, the following abbreviations are used: C: CaO; A: Al_2O_3; S: SiO_2; H: H_2O.

The addition of GGBS to a clay-lime system modifies the clay-lime reaction products. GGBS provides additional alumina, calcium, silica and magnesia to the mixtures depending on the type and amount of GGBS replacement, (Regourd, 1980; Smolczyk, 1980). Since the principal reactants introduced by GGBS are also present in the clay-lime system, the reaction products of clay-lime-GGBS system are relatively similar to those of clay-lime system.

The effectiveness of GGBS hydration depends primarily on many factors. These are the chemical composition of the GGBS; alkali concentration of the reacting system, fineness of the GGBS, glass content of the GGBS, and temperature (Kinuthia, 1997). Caijun and Day (1993) studied the hydration of Canadian GGBS and they found that when GGBS is in contact with water, a Si-Al-O rich layer forms on the GGBS particle surfaces. This layer may absorb H^+ resulting in an increase in OH^- and pH of the solution but this is insufficient to break the Si-O and Al-O bonds to allow formation of the C-S-H, C-A-H and C-A-S-H components. The initial reaction during GGBS hydration produces coatings of aluminosilicate on the surface of GGBS grains within a few minutes of exposure to water and these layers are impermeable to water, inhibiting further hydration reactions (Daimon, 1980). Therefore, GGBS used on its own shows little hydration. Caijun *et al.*, (1993) for example, found only a small amount of C-S-H was formed after 150 days of moist curing. To understand how the GGBS stabilises soil, the hydration products are discussed in some details in the next section.

3.6 HYDRATION PRODUCTS

GGBS hydration products using an alkali are mainly calcium silicate hydrate and hydrotalcite type phase containing magnesium (Tasong *et al.*, 1999; Wild *et al.*, 1998). Song *et al.*, (2000) reported the formation of the hydrotalcite type phase in higher pH pastes along with C-S-H. They also observed that the pH of the mixing solution may affect the nature of C-S-H and its Ca/Si ratio. Also, the solubility of Si increases with pH while that of Ca decreases, pastes with a higher pH pore solution have C-S-H with a lower Ca/Si ratio. Talling (1989) using XRD, studied lime alkali activated GGBS. He identified the presence of C_4AH_{13}. Ettringite ($C_3A.3CaSO_4.32H_2O$) is also a principal hydration product in Portland cement and in GGBS-Portland cement blends. The formation of $Ca(OH)_2$ during OPC hydration produces an alkaline environment suitable for dissolution of Al_2O_3 and SiO_2. These are liberated from the GGBS and/or any other source in the reacting system such as clay or Portland cement. In the presence of $Ca(OH)_2$, $CaSO_4$ reacts with Al_2O_3 to form ettringite ($C_3A.3CaSO_4.32H_2O$).

When sodium hydroxide (Na OH) is used to activate GGBS, the principal reaction products are C-S-H, C_4AH_{13} and C_2ASH_8. Regourd (1980) stated that C_2ASH_8 does not form when $Ca(OH)_2$ is the activator, as it is unstable in the presence of calcium hydroxide. In the presence of gypsum, and without alkali, Regourd (1980) identified the formation of C-S-H, ettringite ($C_3A.3CaSO_4.\ 32H_2O$) and aluminium hydroxide Al $(OH)_3$. Regourd also observed the presence of ettringite and gypsum after 28 days in the absence of alkali. However, Regourd detected the presence of C-S-H and ettringite only while gypsum was consumed in the presence of alkali. Song *et al.,* (2000), in their study on GGBS hydration activated by Na (OH) observed the formation of C-S-H gel, and, in a high pH environment, a hydrotalcite like phase containing magnesium at later stages of hydration.

The microstructural features in the GGBS/activator mixture comprise of a poorly crystallised hydrated layer on the grain surface (Daimon, 1980; Richardson *et al.*, 1994). The hydration products of GGBS are found to be more crystalline than the hydration products of Portland cement, and so add density to the cement paste (Taylor, 1990; Smolczyk, 1980).

3.6.1 HYDRATION MECHANISM OF PORTLAND CEMENT-GGBS MIXTURE

When water is added to a GGBS cement mixture, the hydration process can be summarised as follow (North East GGBS Cement Ltd, 1997):

- Water begins to combine with Portland cement and calcium silicate hydrate begins to form.
- The other reaction products of Portland cement are calcium hydroxide and later sodium and potassium hydroxides.
- These alkalis activate the GGBS which reacts with the water to produce hydrates similar to those produced by the Portland cement hydration.
- The excess silicates and aluminates from the GGBS hydration combine with the calcium hydroxide in a pozzolanic reaction.

The above sequence is in chronological order. The first stage starts immediately and stage four takes much longer. Therefore, the strength development of Portland cement/ GGBS is slower than Portland cement alone.

3.6.2 MICROSTRUCTURE OF PORTLAND CEMENT-GGBS SYSTEM

Calcium silicate hydrate (C-S-H) is the principal binding phase in hardened ordinary Portland cement (OPC) pastes, and in all pastes containing OPC that has been partially replaced by GGBS. In GGBS/OPC blends the C-S-H is present in " inner product" (Ip) within the space originally occupied by either slag or Portland cement grains, or as "outer product" (Op) in the originally water-filled spaces. The composition of the blend governs the morphology and composition of the Op C-S-H and the Ip C-S-H (Richardson and Grooves, 1992; Richardson *et al.*, 1993). Ip and Op C-S-H are morphologically distinct from one another. Ip normally has a dense homogenous morphology with only very fine porosity. Ip with a fine dense morphology is shown in the bottom left of figure 3.2. The morphology of Op C-S-H varies with chemical composition: at high Ca/Si ratio it has fibrillar morphology, and it changes to foil –like with a reduction in Ca/Si ratio. The top right of figure 3.2 shows the foil like morphology of Op C-S-H; this morphology is responsible for the improved durability of high slag cement systems (Richardson, 2000; Richardson and Cabrera, 2000).

3.6.3 NANOSTRUCTURE OF C-S-H

Solid state ^{29}Si MAS NMR (magic angle spinning nuclear magnetic resonance) provides quantitative information on the fraction of silicon present in different tetrahedral environments, Q^n, where n denotes the connectivity of the silicate tetrahedron ($0 \leq n \leq 4$). Thus Q° represents isolated tetrahedra, Q^1 denotes chain end group tetrahedra, Q^2 middle groups, Q^3 branching sites, and Q^4 cross-linking sites in the three dimensional framework. This terminology is illustrated in figure 3.3 which shows schematic representations of pentameric silicate chains (Richardson *et al.,* 1993; Richardson, 2000). An average chain length of the aluminosilicate of the C-S-H can be calculated from the ^{29} Si single pulse data from equation 3.1

Figure 3.2 Transmission electron micrograph showing a region of slag derived Ip (bottom left) with fine dense morphology and foil- like Op C-S-H in a 90% slag 10% C_3S blend hydrated for18 months at 20 °C, Richardson and Cabrera 2000

$$\overline{CL} = \frac{2}{\left(\frac{Q^1}{Q^1 + Q^2}\right)} \tag{3.1}$$

In cement- slag blends, C-S-H also contains aluminium. The Al/Ca ratio varies with Si/Ca ratio according to equation 3.2 (Richardson and Groves, 1993; Richardson and Groves, 1997) and this affects the NMR spectra.

$$\text{Si/Ca} = 0.4277 + (2.366 \times \text{Al/Ca}) \tag{3.2}$$

The Al/Si ratio can be calculated from equation 3.3 and the average chain lengths using equation 3.4.

$$\text{Al/Si} = \frac{\frac{1}{2}Q^1(1Al)}{Q^1 + Q^2(0Al) + Q^2(1Al)} \tag{3.3}$$

$$\overline{CL} = \frac{2}{\left(\dfrac{Q^1}{Q^1 + Q^2(0Al) + \frac{3}{2}Q^2(1Al)}\right)} \qquad (3.4)$$

Figure 3.4 illustrates single pulse ^{29} Si NMR spectra for three 5M KOH- activated slag specimens hydrated for 7 days at 20° C, with high, medium and low aluminium content. Three major peaks were formed, at ~ –79 ppm, ~-82 ppm and ~-85 ppm, these peaks being attributed to Q^1, Q^2(1Al) and Q^2respectively. The peak formed at ~82 ppm is prominent with high Al/Si ratio, and it becomes less prominent with reducing Al/Si ratio. Results of the deconvolution of the spectra reveals that with decreasing Al/Si ratio of slag, a marked decrease in Al/Si of C-S-H, chain length and hydration degree were observed (Richardson, 1999). Increasing silicate chain length leads to a more crystalline and more homogenous C-S-H product. The reaction of GGBS activated by alkali with clay is slightly different and the next section explains this reaction in some detail.

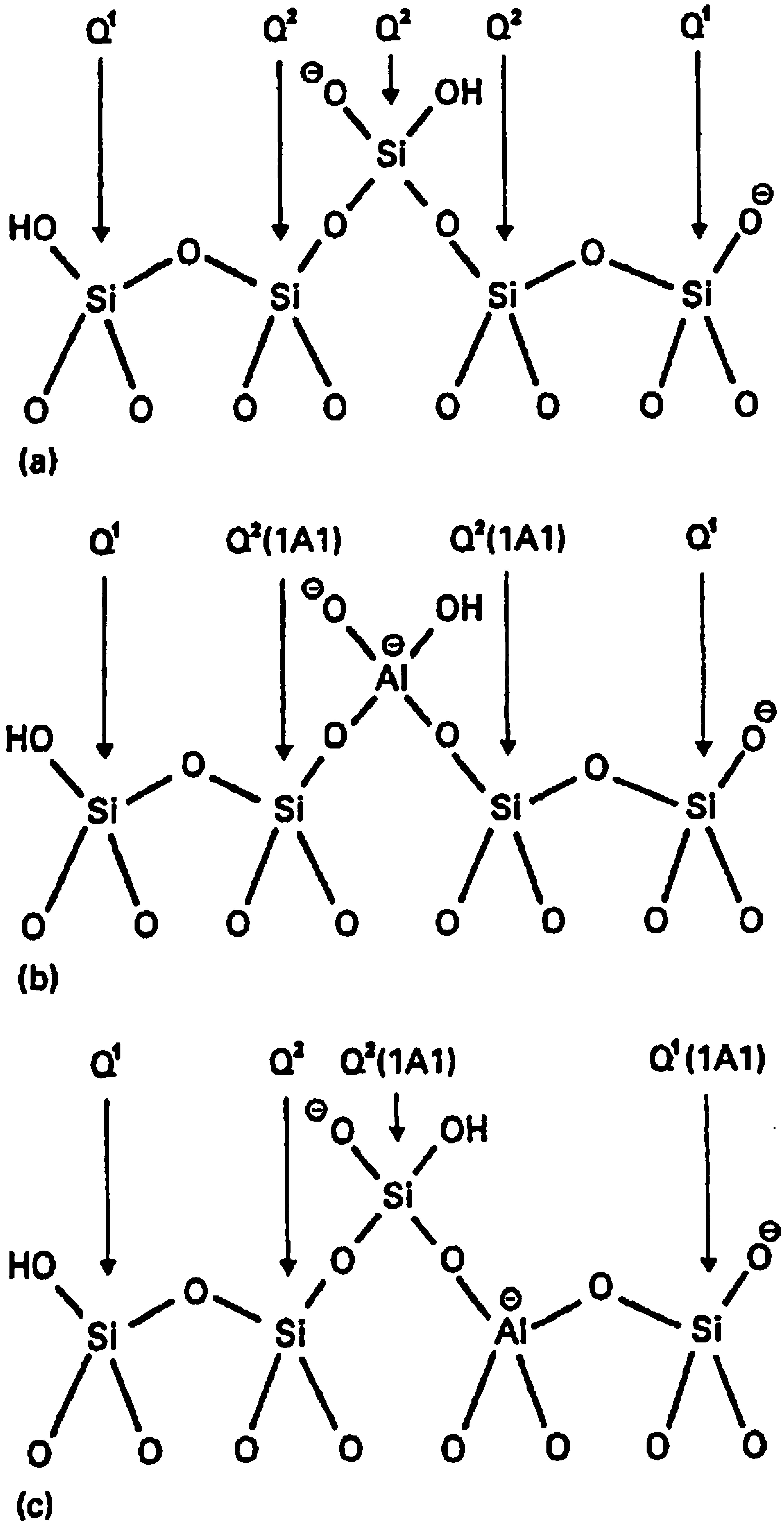


Figure 3.3 (a) Schematic representation of a pentameric chain for the structure of C-S-H. Q^1 and Q^2 units are identified; the middle Q^2 unit is the bridging tetrahedron (b) Same as (a) but with Al substituted for Si in the bridging site. (c) Same as (a) but with Al substituted for Si in a non-bridging Q^2 site, Richardson *et al.*, 1993

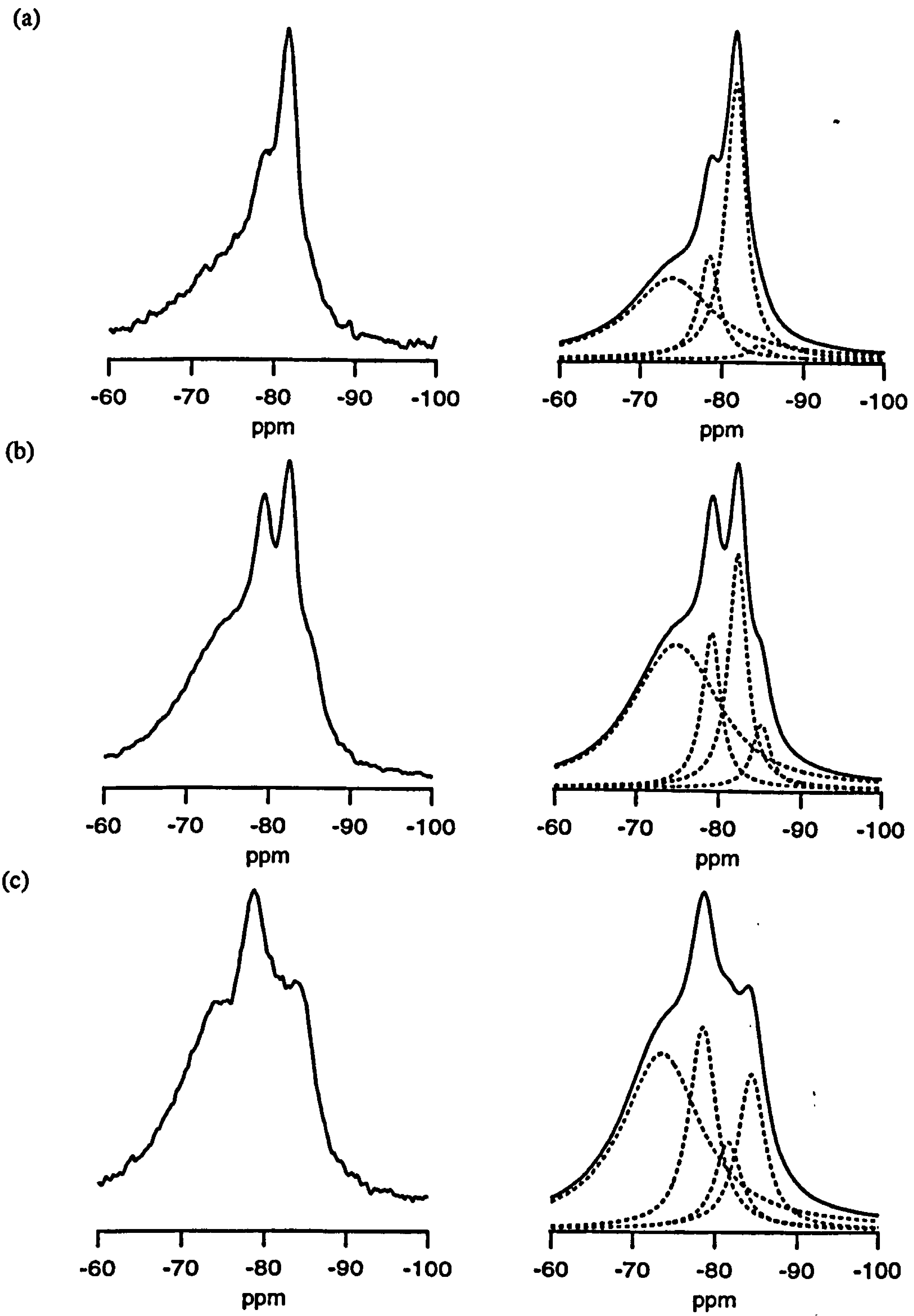


Figure 3.4 Single pulse ^{29}Si NMR spectra (left) and fits (right) for three 5M KOH-activated slag (hydrated or 7 days at 20° C, with S/S =0.4) with (a) high Al content, (b) intermediate Al content, and (c) low Al content, Richardson, 1999

3.7 CLAY-LIME-GGBS REACTIONS

Clay-lime reactions and the hydration of GGBS activated by lime were explained in some detail. The clay–lime–GGBS reaction is different from the clay- lime reaction. Indeed there are two competing reactions rather than one. The first reaction is the hydration of GGBS activated by lime to produce C-A-S-H gel and hydrotalcite type phases containing magnesium (Meng *et al.*, 1998). This reaction is also known to consume lime. The second reaction is the clay- lime reaction to produce C-A-S-H and calcium aluminates and alumino- silicates. In contrast to the pozzolanic reaction of clay with lime, which is slow, the slag hydration, activated by lime, is much quicker (Tasong and Wild, 1999). The strength of clay-lime-GGBS mixtures is governed by the same factors observed in GGBS–OPC blend hydration. These factors include properties of the C-S-H gel such as its amount, porosity, permeability and structure. The lime in the lime-clay mix supplies the required alkaline environment for GGBS activation and hydration.

Wild *et al.*, (1999) suggested that the total binder content (GGBS and lime) would be determined by the required engineering properties. They also recommended that the lime content should be partially replaced with 60 to 80% GGBS to reduce sulphate expansion. Wild *et al.*, (1999) reported that lime replacement by GGBS enhances strength and using a GGBS-lime system instead of lime only leads to a reduction in total binder content. However, the degree of replacement should not exceed a certain percentage to keep a minimum lime content sufficient to fully activate the GGBS. Therefore, preliminary strength and swelling tests must first be conducted in order to establish an appropriate lime-GGBS content.

Sulphate may be present in clay soils in significant percentages. Clay-lime-GGBS reactions are slightly different in the presence of sulphate. The next section describes in some detail the effect of sulphate on clays and on the clay-lime-GGBS reactions.

3.8 EFFECT OF SULPHATE ON THE SWELLING BEHAVIOUR OF CLAYS

Serious problems of swelling and heave have been observed where sulphates are present in lime stabilised clayey soil and this swelling is associated with ettringite formation. Mitchell (1986) studied the failure of lime-stabilised pavement bases. He reported large amounts of heave leading to pavement failure in the Stewart Avenue lime-stabilised sub-base in Las Vegas, U.S.A. Mitchell noted that the first signs of the pavement failure occurred two and a half years after the project completion. He observed that the density of the lime stabilised soil in the failed sections was lower than that of the undamaged sections. He also observed that the moisture contents of the damaged sections were higher than those of the undamaged sections. Large amounts of ettringite ($C_3A.3CaSO_4.32H_2O$) and thaumasite ($CaSiO_3.CaCO_3.CaSO_4.15H_2O$) were found in both the failed and un-failed sections. The swelling and failure were attributed to the expansion due to the formation of ettringite and thaumasite which, when exposed to water, produced swell greatly in excess of that exhibited by the untreated soil.

Hunter (1988) studied the same damaged sections. He observed that the areas of serious damage were found near to a source of water. It was concluded that the availability of pore-water is the most important factor in heave.

Snedker and Temporal (1990) reported 60% heaving on the Banbury section of the M40 motorway between Oxford and Birmingham in the U. K. It was suggested that the lime-stabilised sections of the motorway were subjected to sulphate attack and ettringite was formed.

In the kaolinite-lime- gypsum system where expansion has been shown to be related to ettringite formation, expansion increases with an increase in sulphate content and also increases linearly with water absorption (Abdi and Wild, 1993). Tasong *et al.*, (1999) also reported severe disruption when lime-kaolinite mixtures were exposed to sodium sulphate (Na_2SO_4) solution and this disruption is associated with the formation of ettringite. They found also that the progressive replacement of lime by

GGBS resulted in a progressive modification of the microstructure, with respect to ettringite morphology.

Regourd (1980) reported that in the presence of sulphate ions, the first hydrates formed around the GGBS grains dissolve and a second layer of hydrates of a coarser structure form which does not prevent the penetration of water. This encourages further reactions including sulphate attack. In the U. K, gypsum is abundant in some soils such as Oxford clay and Kimmeridge clay.

The formation of ettringite in systems containing GGBS does not necessarily result in expansion and swelling. For example, in super-sulphated cement 80-85% of GGBS is blended with 10-15% of calcium sulphate and about 10% ordinary Portland cement as an activator. Although ettringite is the principal hydration product and a substantial amount of sulphate is present in the mixture, the cement has no tendency to expand. Calcium sulphate is consumed rapidly and the GGBS particles form nucleation sites on which well developed ettringite crystal form. The ettringite forms by a manner such that little expansion occurs. It is also highly resistance to attack by external sulphate. The well established sulphate resisting properties imparted to cement by blending with GGBS suggests that blending lime with GGBS might impart similar sulphate-resisting properties to lime-stabilised clay. In both sulphate containing lime stabilised clays and hydrated Portland cements, similar phases are present i.e C-S-H and C-A-S-H gels, $Ca(OH)_2$ and ettringite (Taylor, 1990; Kinuthia 1997).

The engineering properties of lime stabilised clay are affected significantly by the presence of sulphates, which may occur either in the raw stabilisation materials, in the water used for mixing and/or in the ground water. Mitchell (1986) suggested that the change in soil properties is due to the modified cation exchange process and pozzolanic reactions due to the presence of sulphate. This effect depends on the sulphate concentration, the metal cation, the amount of lime added and the curing conditions.

Additional Ca^{2+} cations and SO_4^{2-} anions would result from the presence of gypsum in lime stabilised clay soil. The extra Ca^{2+} cations lead to an increase in the overall number of cations attracted to the clay particle surfaces. Sulphate also affects the nature and type of reaction products formed in the lime stabilised soil. In the absence of sulphate, calcium silicate hydrate gel C-S-H with crystalline or semi-crystalline calcium aluminate hydrate (C_4AH_{13}) and calcium alumino silicate hydrate.(C_2ASH_8) are formed after the dissolution of silica and alumina from the clay (Mitchell , 1986; Arabi and Wild, 1986).

In the presence of sulphate, the reaction product will be modified and a layer of colloidal product consisting of a complex calcium-sulpho-aluminate-silicate hydrate ($C\text{-}A\text{-}S\text{-}\bar{S}\text{-}H$) will be formed on the kaolinite plates (Wild *et al.*, 1993). Other products can be formed due to the formation of regions rich in sulphate and poor in silica developing within this colloidal layer and a high sulphate product low in silica is formed on the surface of the solid clay particles as ettringite ($C_3A3C\bar{S}H_{32}$). The colloidal gel of ettringite has the ability to attract many water molecules, causing inter-particles repulsion and an overall expansion of the system. When water is present, ettringite will cause significant expansion. Mitchell (1986) reported the formation of thaumasite in lime clay stabilised clay.

In the absence of calcium sulphate, increasing the GGBS to lime ratio but keeping the total GGBS and lime constant results in a significant increase in strength. This indicate that the GGBS hydration reaction activated by lime is faster than the normal clay-lime reaction. Wild *et al.*, (1998) also observed an increase in strength by partially substituting lime with GGBS in a lime stabilised clay in the presence of gypsum particularly in the first few weeks of curing.

3.9 EFFECT OF GROUND GRANULATED BLAST FURNACE SLAG ON THE ENGINEERING BEHAVIOUR OF SOIL

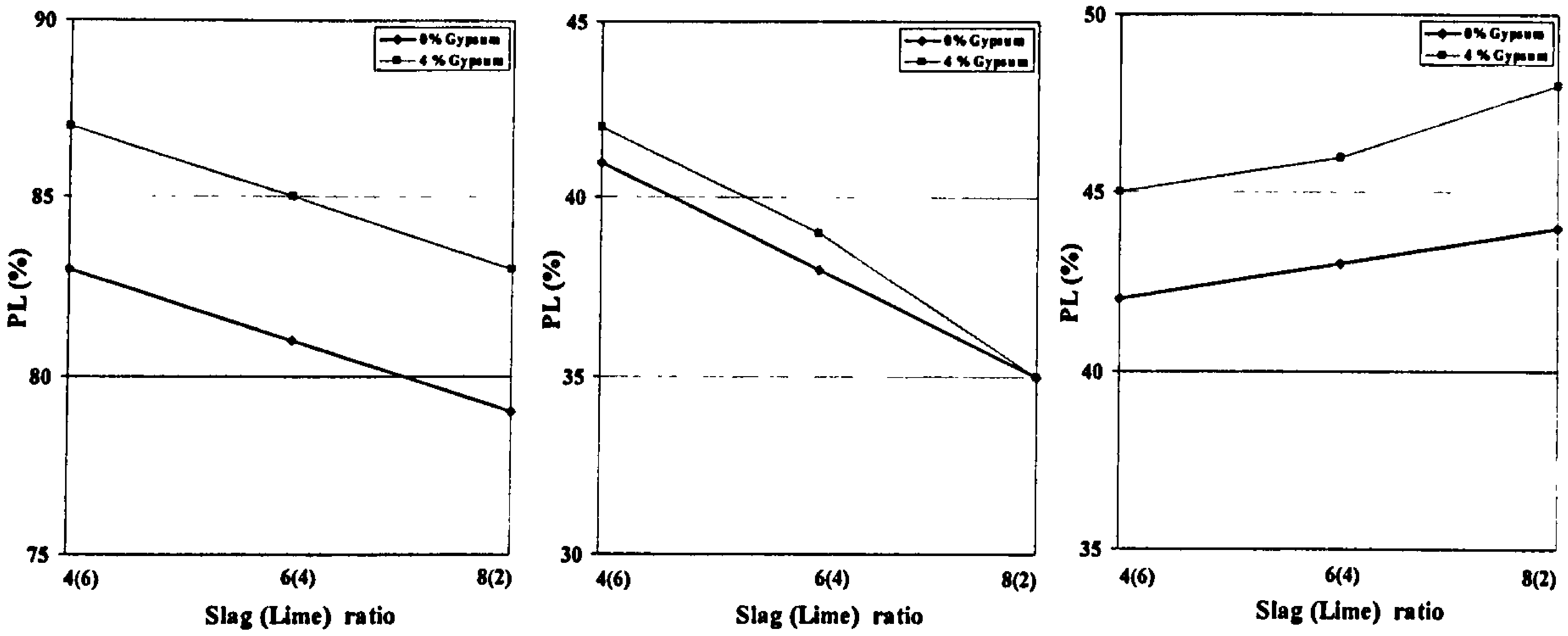
3.9.1 INTRODUCTION

As may be concluded from the preceding sections, GGBS activated by lime affects many of the engineering properties of soils. In the next section the effects of GGBS activated by lime on the liquid limit (LL), plastic limit (PL), plasticity index (PI), optimum moisture content (OMC), maximum dry density (MDD), volume stability and swelling potential and unconfined compressive strength (UCS) will be discussed in some detail.

3.9.2 EFFECT OF GGBS ON THE CONSISTENCY CHARACTERISTICS

GGBS addition to the clay soils generally alters the consistency characteristics of the clayey soil. Akinmusuru (1991) studied the effect of adding granulated blast furnace slag on the consistency, compaction characteristics and strength of Lateritic soil. The granulated blastfurnace slag content varied from 0% to 15% by dry soil weight. He observed a decrease in both the liquid and plastic limits and an increase in plasticity index with increasing GGBS addition.

Wild *et al.*, (1996) studied the effect of adding lime, and GGBS activated by lime on kaolinite clay. They observed that addition of small amounts of lime to kaolinite produces a marked increase in the plastic limit. The liquid limit may increase or decrease but in such a way that there is an increase in the plasticity index with increasing lime percentage. The addition of GGBS and lime to kaolinite alters the Atterberg limits giving a small reduction in the liquid limit and a marked decrease in plastic limit, thus producing an increase in plasticity index with a decrease in lime/ GGBS ratio. Wild *et al.*, (1996) also observed that these trends were maintained when gypsum was present in kaolinite, but gypsum has the ability to produce slightly higher liquid limits and plasticity indexes, figure 3.5.



a) b) c)

Figure 3.5 Changes in (a) liquid limit, (b) plastic limit and (c) plasticity index with composition, kaolinite-10 wt % (GGBS/lime) mixes with and without 4% gypsum , Wild *et al.*, 1996.

3.9.3 EFFECT OF GGBS ON THE VOLUME STABILITY AND SWELLING POTENTIAL OF SOILS

Higgins *et al.*, (1998) in a site trial investigated the effect of using GGBS and lime on the swelling characteristics of kaolinite and Kimmeridge clay. This trial showed that GGBS was completely successful in reducing swelling caused by sulphate. Higgins and Kennedy (1999) also carried out a full site trial using GGBS and lime on a temporary diversion to carry the A421 Tingewick bypass traffic. This site contained a sulphate–containing boulder clay. They used GGBS activated by lime in particular sections and lime and cement in other sections. They found that the temporary diversion performed well over a year. They did not observe any swelling problems in the sections which were treated with GGBS activated by lime while observing indications of expansion at the section which was treated with lime and cement without using GGBS.

Higgins *et al.*, (1998) studied the effect of GGBS on the strength and swelling properties of lime-stabilised kaolinite in the presence of sulphate. They found that kaolinite clay containing gypsum and stabilised with lime produced large expansion when exposed to water. The addition of GGBS to the clay-lime-gypsum system results in a great reduction in expansion. Higgins *et al.*, (1998) concluded that the laboratory investigation and a full-scale trial demonstrated that lime/GGBS combinations can be successfully used for soil stabilisation in the presence of sulphate to prevent swelling attack. They also found that substitution of GGBS for lime could significantly reduce swelling and heave in the presence of sulphates. Wild *et al.*, (1999) also pointed out that to eliminate problems of sulphate expansion 60 to 80 % of the lime for stabilisation is replaced by GGBS. The total binder content is determined by the engineering properties required.

Wild *et al.*, (1996) studied the effect of GGBS on reducing the swelling potential of kaolinite in the presence of sulphate. They suggested that the swelling reduction is not a result of increased interparticle bonding, because the addition of GGBS tends to produce a slight decrease in strength, suggesting no improvement in interparticle bonding. GGBS hydration in the presence of calcium sulphate produces C-S-H gel and ettringite (Taylor, 1990). Including GGBS in clay-lime-gypsum mixes produces two competing reactions. GGBS hydration activated by lime in the presence of calcium sulphate to give C-S-H gel and ettringite and the kaolinite-lime reaction in the presence of calcium sulphate to give a colloidal CAS$\bar{S}$H product and ettringite. Also, there will be competing nucleation sites for ettringite on the kaolinite particle surfaces and on the GGBS particle surfaces (Wild *et al.*, 1996). Therefore, the hydration reaction of GGBS becomes the dominant reaction, (as it is faster than clay-lime reaction) and growth of ettringite crystals on GGBS particles starves the kaolinite particles of available sulphate which is necessary for expansion. Due to a lack of sulphate on the clay particles, there is no ettringite nucleation on the clay particle surfaces. The nature of the reaction product which forms on the clay particles is modified and the clay particles do not suffer any extreme swelling when exposed to water, (Wild *et al.*, 1996).

Tasong *et al.*, (1999) also observed that there are a number of factors which govern the swelling caused by ettringite formation. In the kaolinite-lime-gypsum system, expansion has been found to be related to ettringite formation. Expansion increases with an increase in sulphate content and also increases linearly with water absorption.

3.9.4 EFFECT OF GGBS ON THE COMPACTION CHARACTERISTICS OF SOIL

Akinmusuru (1991) observed that the addition of GGBS to soil increased the maximum dry density up to 10% granulated blast furnace slag addition, above which it decreased. This could be due to an increase of fine powder in the mixture leading to decrease the proportion of the coarse material making it difficult to attain good compaction, figure 3.6.

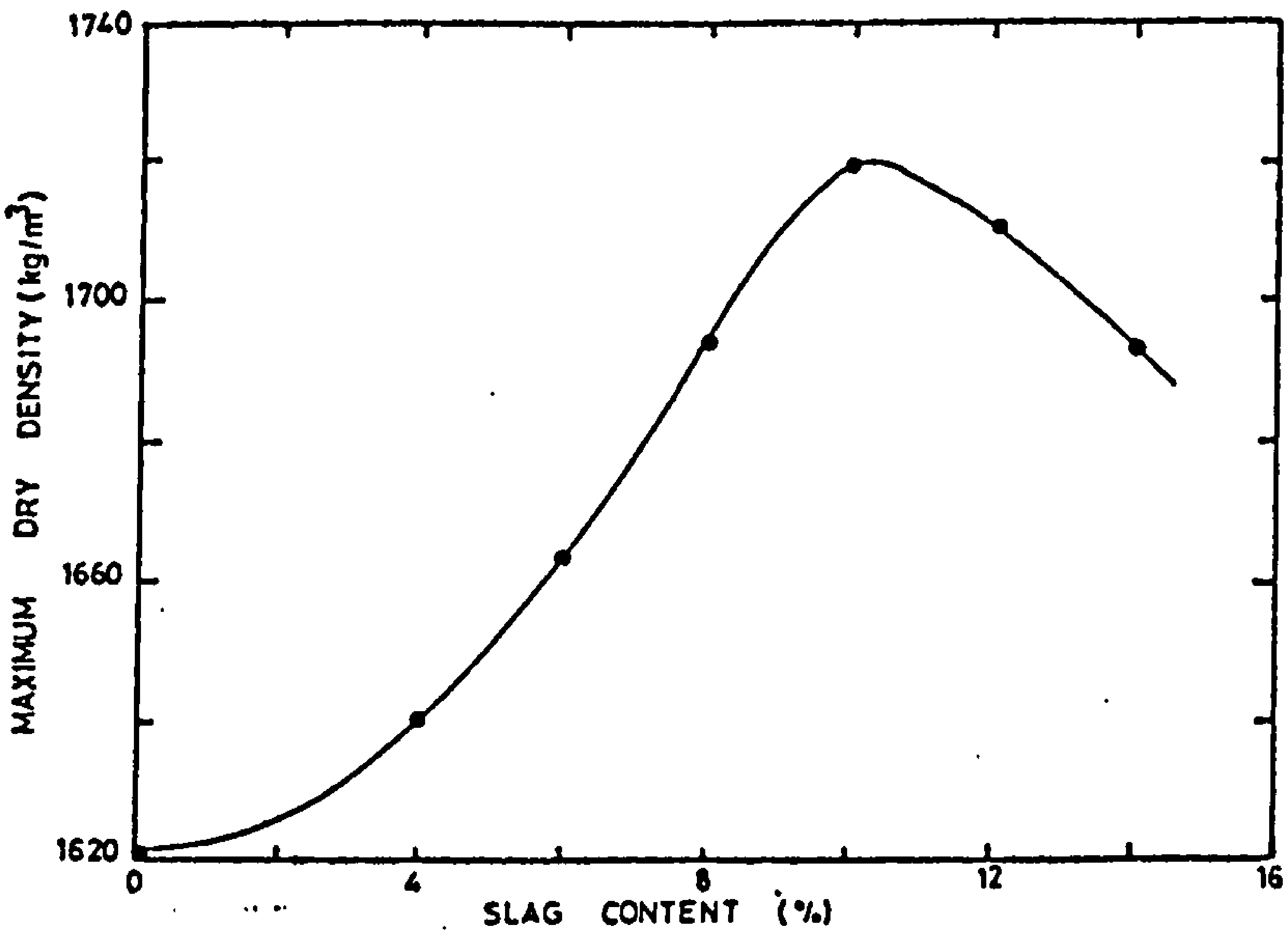


Figure 3.6 Effect of slag content on maximum dry density, Akinmusuru, 1991

Wild *et al.*, (1996) found that the addition of lime to kaolinite dramatically decreases the maximum dry density and increases the optimum moisture content. However, a decrease in lime/GGBS ratio produced a slight increase in maximum dry density and a slight and non-systematic variation in the optimum moisture content. The presence of gypsum gives a slight increase in maximum dry density and optimum moisture content.

3.9.5 EFFECT OF GGBS ON THE STRENGTH OF SOILS

The main advantages of using lime/GGBS relative to using lime only are a slower early rate of strength development giving more time to finish the construction, and an increase in long-term strength which improves the performance.

The strength gain using GGBS activated by lime has been investigated by many authors. Gupta and Seehra (1989) studied the effect of lime-GGBS on the strength of soil. They found that lime- GGBS soil stabilised mixes with and without addition of gypsum, or containing partial replacement of GGBS by fly ash produced high unconfined compressive strength (UCS) and California bearing ratio (CBR) in comparison to plain soil. Gupta and Seehra (1989) concluded that partial replacement of GGBS with fly ash further increased the UCS.

Wild *et al.*, (1996) studied the effect of GGBS on kaolinite in the presence of sulphate. They observed, without added gypsum, an increase in strength as the GGBS/lime ratio increased. When gypsum was added, the strengths of the specimens with high GGBS/lime ratios did not increase greatly whereas specimens with low GGBS/lime ratios showed a significant increase in strength. This suggests that gypsum had a significant effect on the lime-kaolinite reaction, figure 3.7. When gypsum is absent, increasing the GGBS/lime ratio but keeping the total stabiliser constant (10%) results in an increase in strength. This results from the GGBS activated by lime reaction being more rapid than the lime-kaolinite reaction.

When gypsum is present it enhances strength development at low GGBS/lime ratios because gypsum has greater accelerating effect on the lime-kaolinite reaction than it does on the GGBS hydration reaction. Wild *et al.*, (1996) also studied the effect of curing period on lime, GGBS and gypsum on the UCS of kaolinite clay. They observed a significant increase in compressive strength with an increase in the curing periods at 6% lime without adding gypsum. Addition of 4% GGBS to the same mixture significantly decreased the UCS. Addition of 4% gypsum to the same mixture greatly increased the UCS with an increase in the curing period, see figure 3.8.

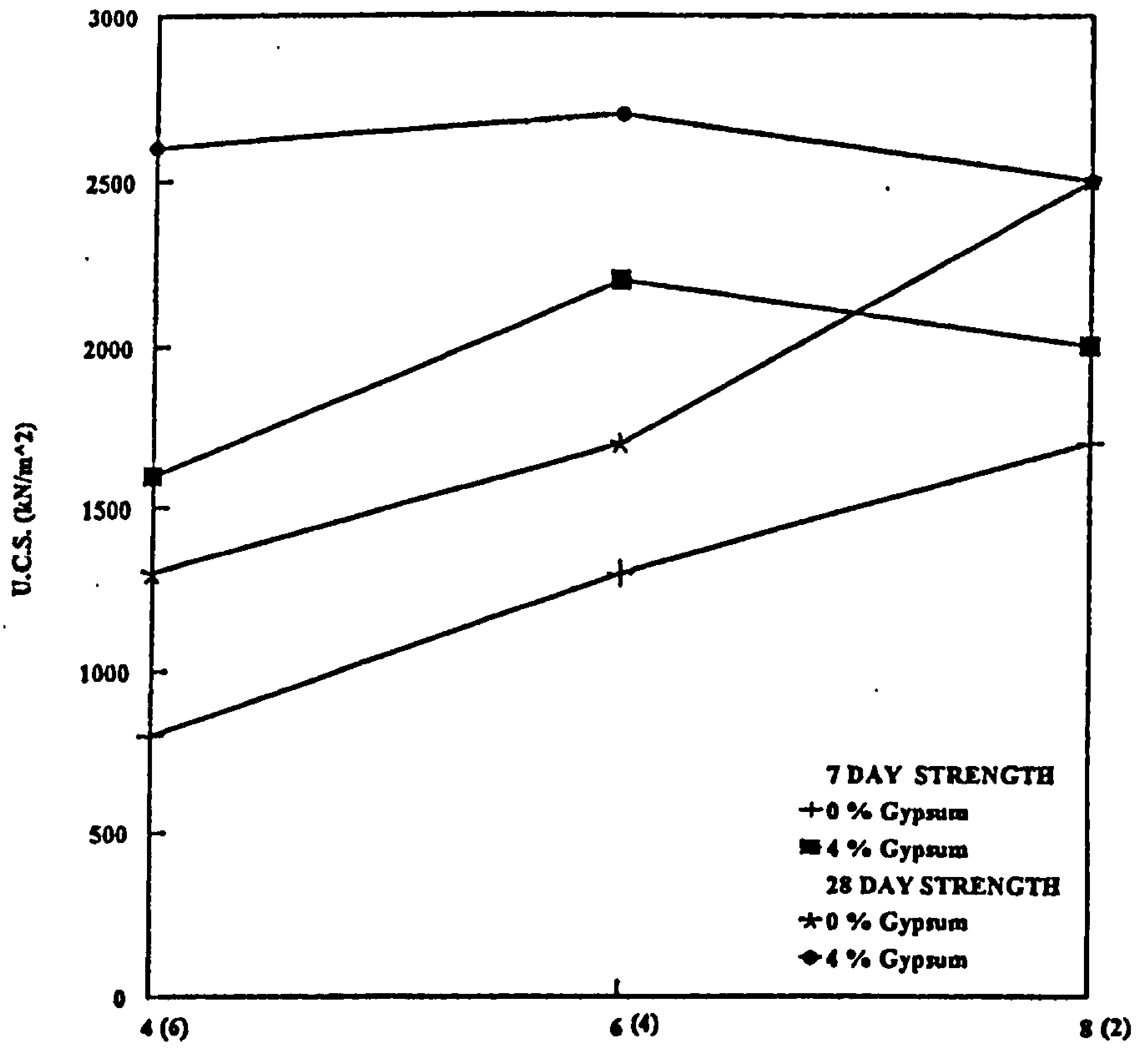


Figure 3.7 UCS vs. composition GGBS/lime for kaolinite with and without gypsum (4 % by dry soil weight) cured for 7 and 28 days, Wild *et al.*, 1996

Higgins (1998 b) used GGBS stabilisation in full-scale trials. In these trials, lime was added initially to wet soils at an accurately controlled rate (10 kg/m^2) and then rotavated in to a depth of 300 mm. The treated layer was then compacted. Subsequently, GGBS was spread at between 10 and 40 kg/m^2 (depending on the strength required) and then rotavated in. The stabilised material was compacted before being trimmed to level of the final wearing surface and sealed with bitumen emulsion, then the final wearing surface was constructed. All these trials were a complete success and subsequent tests on the stabilised subgrade confirmed the achievement of satisfactory density and strength.

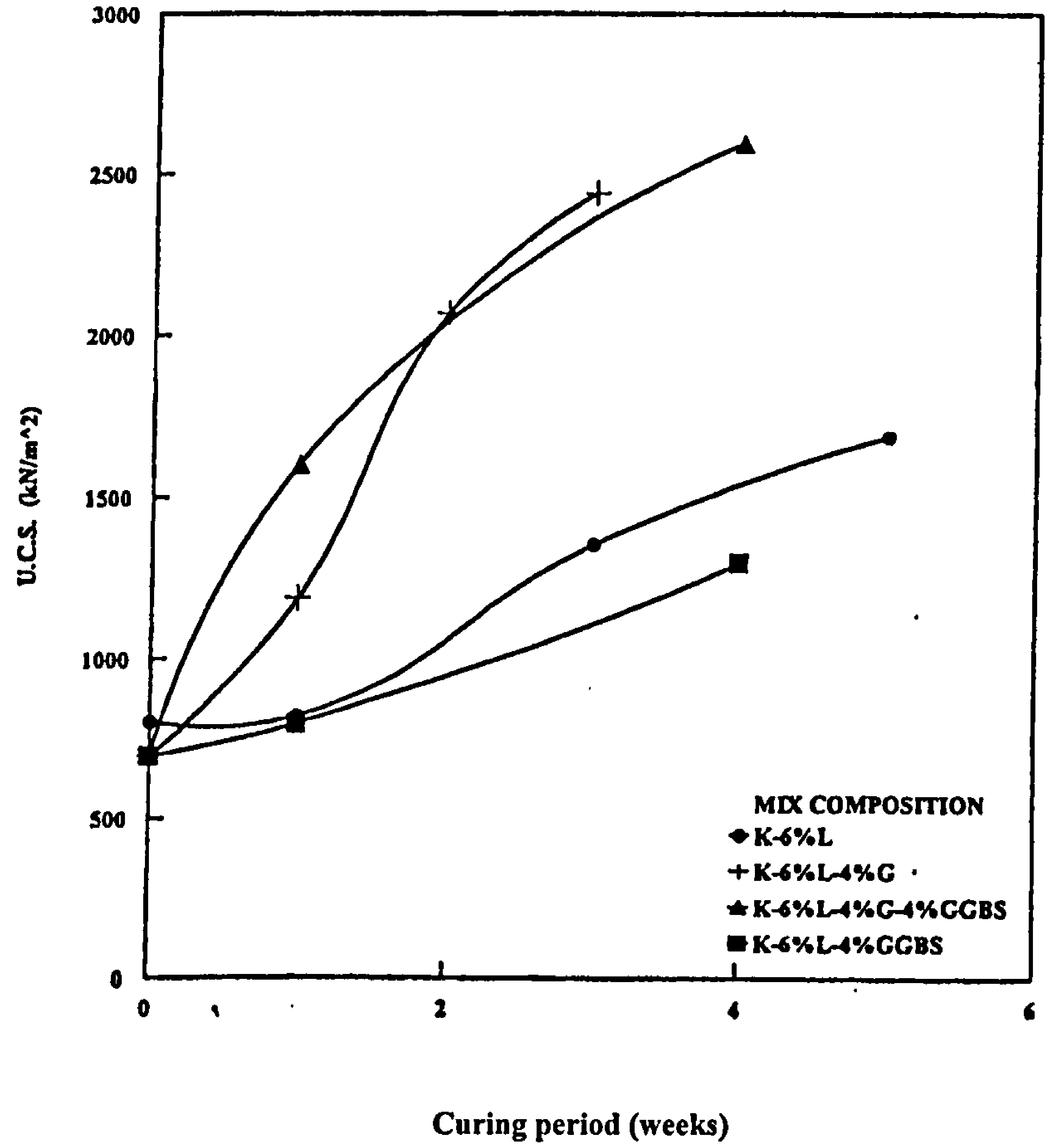


Figure 3.8 UCS vs. curing time for kaolinite-6 % by dry soil weight of lime containing 4% by dry soil weight of Gypsum and/or 4 % by dry soil weight of GGBS, Wild *et al.*, 1996

Akinmusuru (1991) studied the effect of adding GGBS on shear strength parameters, figure 3.9. He observed that the cohesion increased with increasing the GGBS content up to 10% and then subsequently decreased and the angle of internal friction φ decreased with increasing GGBS percentage. Akinmusuru (1991) also stated the CBR increased with an increase in GGBS percent up to 10% GGBS and then started to decrease.

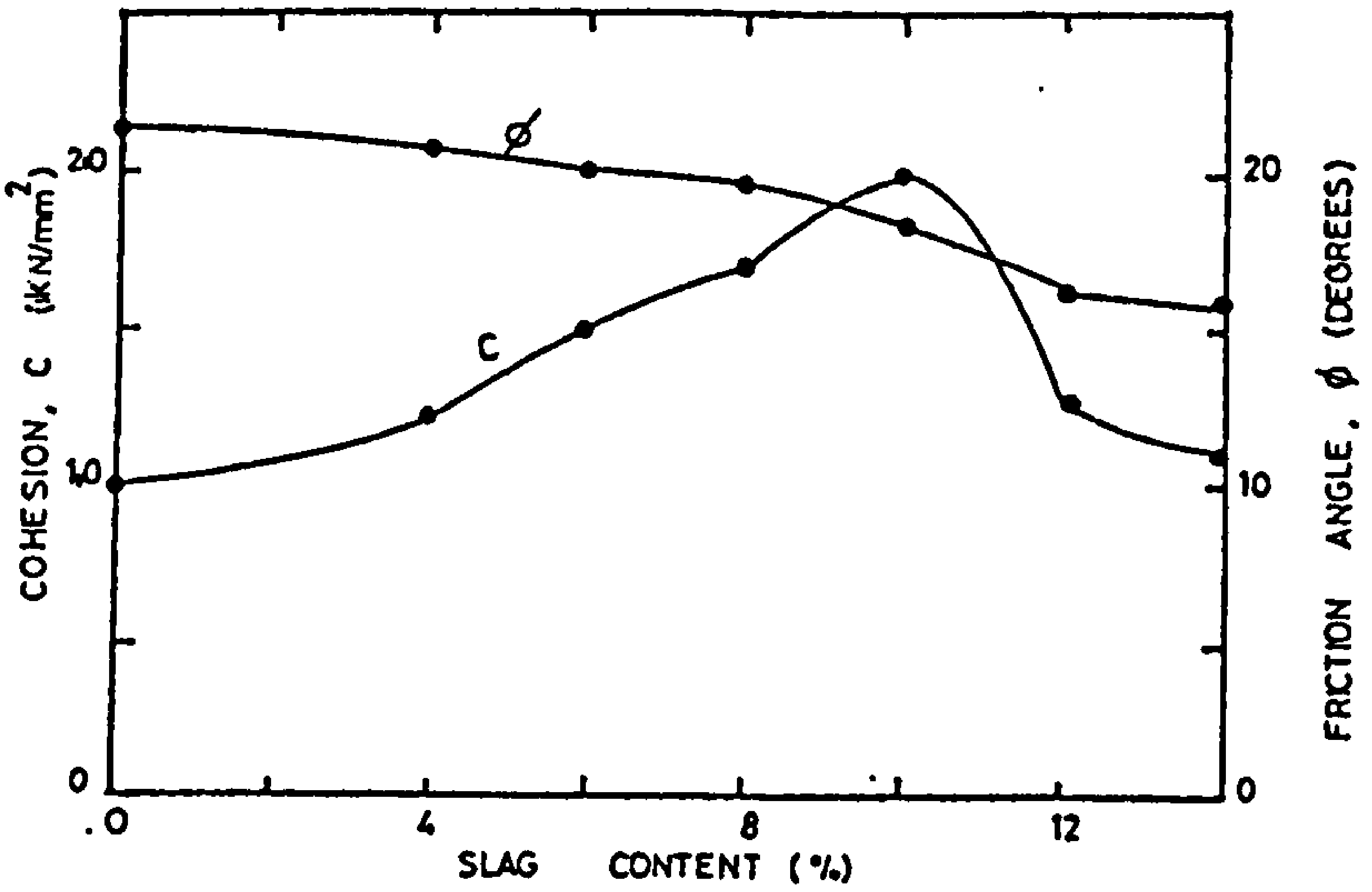


Figure 3.9 Effect of slag content on strength parameter, Akinmusuru, 1991

Wild *et al.*, (1998) studied the influence of GGBS and gypsum on kaolinite and Kimmeridge clay when stabilised with lime. They found that a partial substitution of lime with GGBS produced improved strength for both kaolinite and Kimmeridge clay. In the case of kaolinite the effects are more significant in the presence of gypsum. In the presence of gypsum, the most significant strength enhancement of kaolinite after 28 days, was at high lime/GGBS ratio, and is due to the contribution of gypsum to the longer-term kaolinite-lime-gypsum reaction. In the absence of gypsum, the most significant strength enhancement was at low lime/GGBS ratio, due to lime activated GGBS hydration. The greatest short-term strength enhancement was for low lime/GGBS mixtures with gypsum, due to the accelerating effect of gypsum on the lime-activated GGBS hydration. Wild *et al.*, (1998) observed no effect with GGBS alone on both the 7 and 28 day strengths of kaolinite whilst it provided significant strength enhancement in the case of Kimmeridge clay. It was suggested that the Kimmeridge has a particular oxide (not known) that can partially activate the GGBS.

Substitution of lime by GGBS with Kimmeridge clay provides maximum strength at different lime/GGBS ratios depending on the total stabiliser percentage. In the case of kaolinite, strength increases with an increase in GGBS/lime ratio, although the strength falls at zero lime content, figures 3.10 and 3.11.

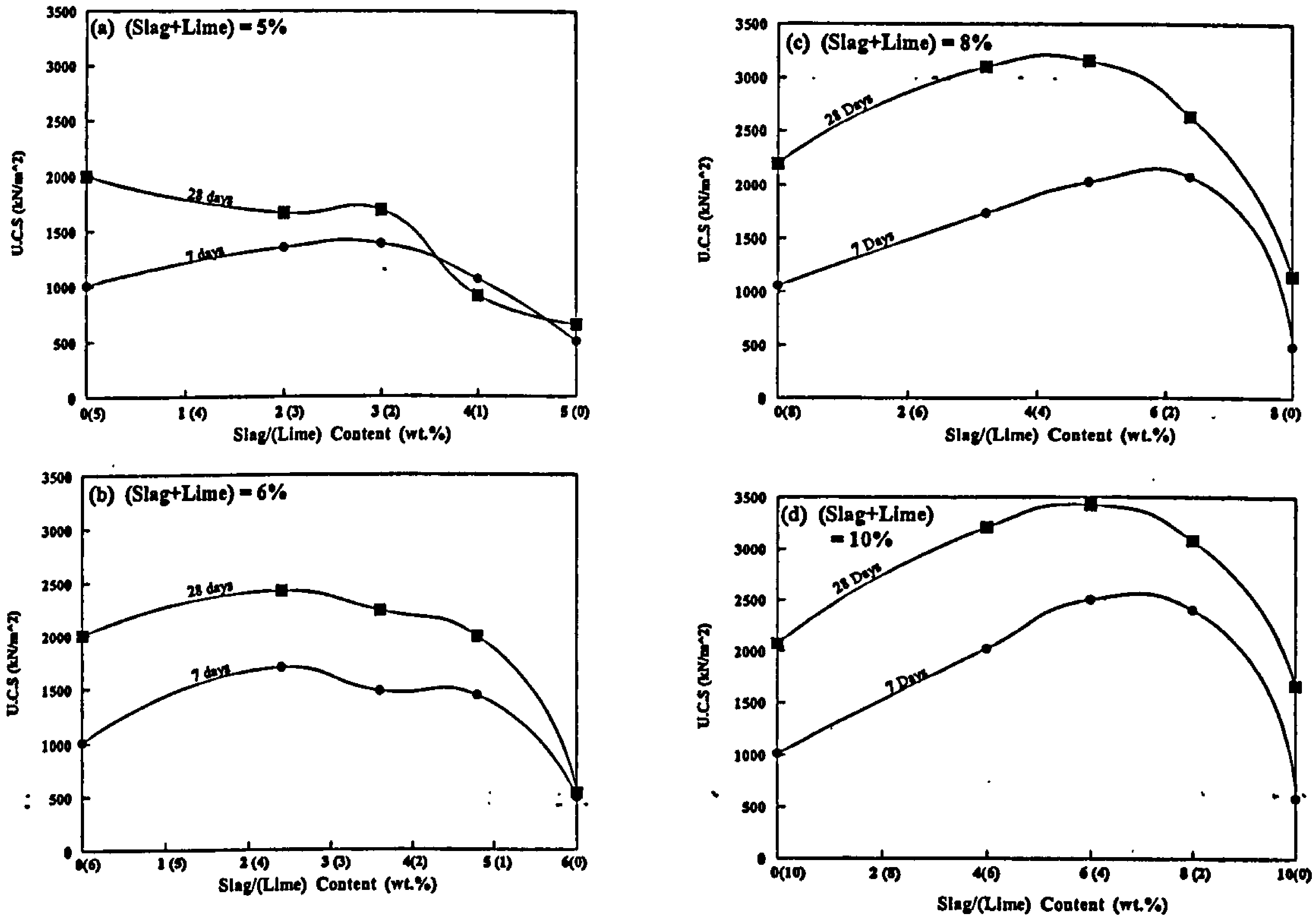


Figure 3.10 UCS vs. GGBS/lime content for Kimmeridge -lime-GGBS with and without (a) 0.93% (b) 1.86% and (c) 2.79%SO_3 equivalent of gypsum and moist cured for 7 and 28 days at 30°C and 100% relative humidity, Wild *et al.*, 1998

Wong (1992) studied the effect of GGBS from different sources and with different gradings on the performance of pavement materials. He observed higher early strengths from finer GGBS. He found also that GGBS is preferred when longer working times are required and that GGBS can be as effective as Portland cement in controlling erosion.

Higgins *et al.*, (1998) observed that the optimum lime/GGBS ratio to achieve the maximum UCS is 1:5 for kaolinite clay and that the optimum is about 2:3 for Kimmeridge clay.

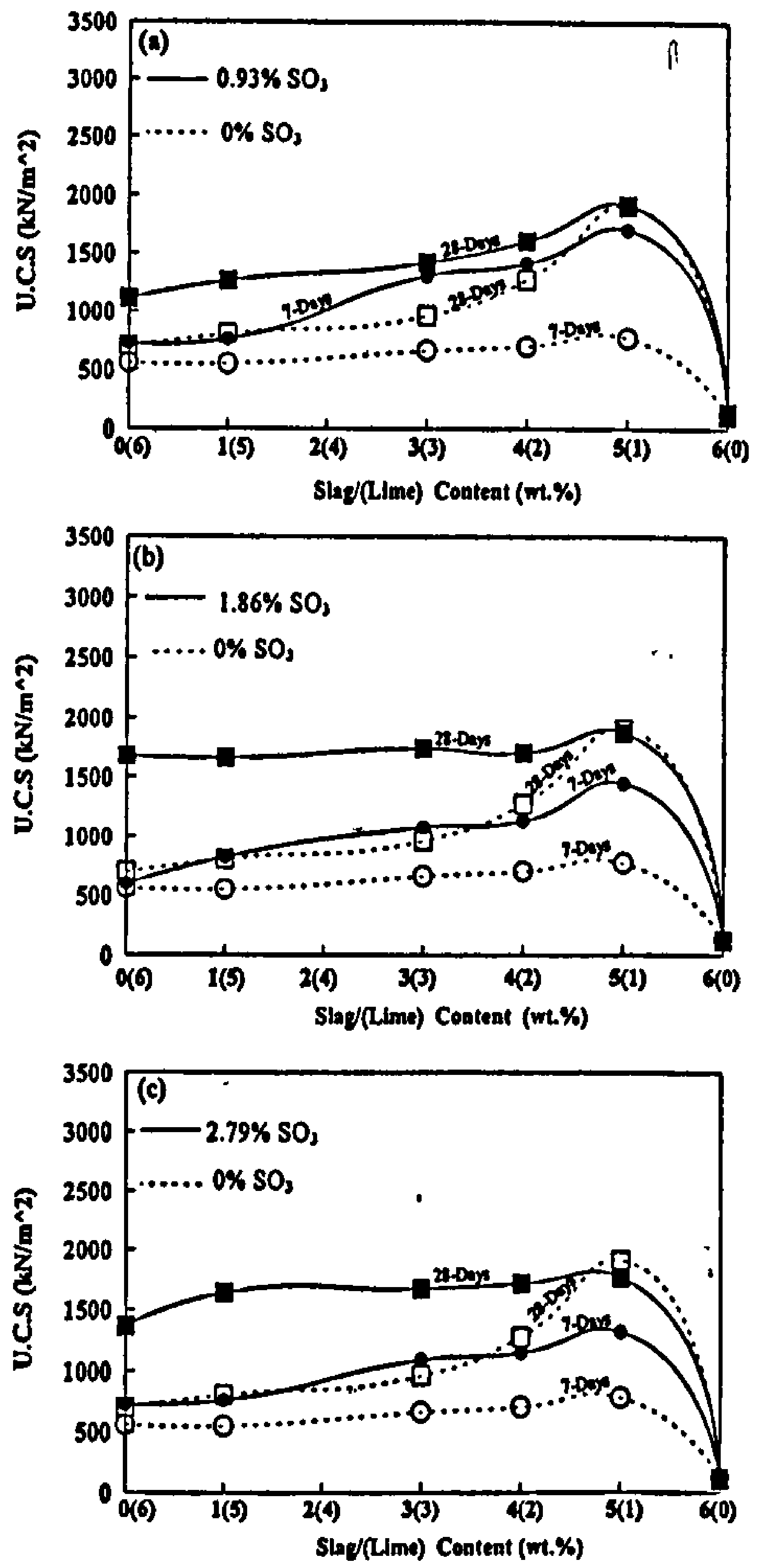


Figure 3.11 UCS vs. GGBS/lime content for kaolinite clay-lime-GGBS containing a total stabiliser (TS) content (lime + GGBS) of: (a) 5; (b) 6; (c) 8; (d) 10% by weight and moist cured for 7 and 28 days at 30°C and 100% relative humidity, Higgins *et al.*, 1998

3.10 ADVANTAGES OF USING GGBS

The lime/GGBS combination offers several advantages over lime only for soil stabilisation (Higgins 1998).

1. A slower early rate of strength development gives considerably more time for construction operations (Wong 1992).

2. In the long-term, the combination of lime/GGBS produces greater strength than lime alone. This combination (GGBS and lime) will improve the durability and performance of structures.
3. For clays containing sulphates deleterious swelling due to the formation of ettringite, is inhibited.
4. Higher percentages of replacement of lime with GGBS, with only sufficient lime to activate the GGBS, are the most effective in preventing sulphate attack (Wild and Tasong, 1999).
5. The addition of GGBS reduces significantly the permeability of stabilised soil which in their natural state have high permeability. The addition of GGBS can reduce the coefficient of permeability to 10^{-6} cm/s, which satisfied the requirements for water retaining structures (Yamanouchi *et al.*, 1982).
6. Due to the shortage in cement production and its cost in the developing countries, using GGBS in soil stabilisation can offer an alternative and cheap effective binder and also can help in conservation of the environment.

3.11 SUMMARY

There are four main conclusions from the work described in this chapter.

- GGBS on its own has only mild cementitious properties so it needs an alkali to be fully activated. The most common alkalis are OPC, calcium hydroxide and sodium hydroxide. In some cases the alkali may be found in the natural soil itself, see section (3.9.5).

- The principal hydration products of GGBS activated by lime are calcium silicate hydrate (C-S-H), hydrotalcite type phase containing magnesium and in some cases crystalline calcium aluminate hydrate (C-A-H) and calcium silicate aluminate hydrate C-A-S-H.

- Two competing reactions are obtained on adding GGBS activated by lime to a clay soil. Hydration of GGBS activated by lime to produce C-A-S-H gel and hydrotalcite phase containing magnesium which is fast. The second slow reaction is the clay-lime reaction to produce C-S-H and in some cases C-A-S-H.

- The addition of GGBS activated by lime to the clay soil alters the consistency limits giving a small decrease in the liquid limit and a marked decrease in plastic limit, thus producing an increase in the plasticity index on decreasing the lime to GGBS ratio. The addition of GGBS activated by lime can also suppress the swelling potential of the clay soil, decrease the maximum dry density and increase the optimum moisture content. The addition of GGBS to clayey soil in some cases markedly increases the unconfined compressive strength of clayey soil.

Chapter two and three described many techniques for chemical soil stabilisation which include lime, cement, flyash and GGBS activated by lime. Lime and GGBS activated by lime stabilisation have been described in detail. Using GGBS as a soil stabilisation agent is still novel in Egypt and GGBS is produced locally in very large amounts in Egypt as a by-product during iron manufacturing. Therefore, this investigation aims to utilize GGBS only, and GGBS activated by lime, to stabilise Egyptian clayey soils to improve their properties as sub-grades for highways. In the next chapter the scope of the investigation and all objectives of the research are described in detail.

CHAPTER FOUR

SCOPE OF THE INVESTIGATION

Chapters two and three covered large areas of chemical soil stabilisation, which include lime, cement and GGBS activated by lime. Lime and GGBS activated by lime stabilisation have been described. Using GGBS and lime as soil stabilisation agents may impart new properties to clay soils. For example, a slower early rate of strength development gives considerably more time for construction operations. However, it may impart greater strength than using lime alone in the long term, which improves the durability and performance of structures, and reduces the permeability of stabilised soil. Using GGBS in soil stabilisation is still a novel process in the U. K. and it has never been used in soil stabilisation in Egypt. GGBS is produced locally in very large amounts in Egypt as a by-product during iron manufacturing and it is relatively cheap comparing to cement and other additives.

The main aim of the project was to investigate the stabilisation of Egyptian clayey soils using GGBS with and without lime. For reasons explained in chapter five, a simulated Egyptian test soil is used.

The main objectives of the project were

1. To investigate the effect of GGBS, with and without lime, on the engineering behaviour of a simulated Egyptian soil
    a. on the plasticity characteristics.
    b. on the compaction characteristics.
    c. on the unconfined compressive strength.
    d. on the swelling potential.

2. To identify the reaction products of the stabilised soil to identify mechanisms by which the changes in engineering properties are achieved. The techniques used are
    a. X ray diffraction

b. Scanning electron microscopy
c. Differential thermal analysis
d. Nuclear magnetic resonance

The parameters that will be varied during the programme are

a. Amounts of stabilisation agents (GGBS and lime).
b. Curing conditions (temperature and time).

The test programme was divided into four main phases:

1. The first phase was to select and characterise the test materials used in the investigation including River Aire soil, montmorillonite and kaolinite clays, compositions of the simulated test and the pure clay soils, GGBS and lime. Base line characteristics including particle size distribution, specific gravity, liquid and plastic limits, maximum dry density, optimum moisture content and the identification of the main oxides of the soils and stabilisers.

2. The second phase of the study was to determine the effect of GGBS only on the engineering behaviour of the test soil which included unconfined compressive strength (UCS) of these mixtures. The effects of varying curing period and curing conditions were also studied and included a study of the influence of GGBS on the compaction, plasticity and swelling characteristics of the test soil.

3. The third phase was to investigate the effect of GGBS in conjunction with hydrated lime as an activator on the engineering behaviour of the test soil which included the UCS, the compaction, plasticity and swelling characteristics. The effects of the amount of GGBS replacement by lime and the effects of varying curing period and curing conditions were also investigated.

4. The fourth phase was to investigate the reaction products of clay with GGBS activated by lime. The analyses were carried out on mixtures of pure clay minerals of the reactive clay portion of the simulated Egyptian soil mixed with varying amounts of GGBS and lime, cured under different curing conditions for different periods.

The research project was designed to enable both qualitative and quantitative evaluation of the effects of GGBS, with and without lime, on the engineering behaviour of the test soils to be made. In the next chapter the properties of the materials used in this investigation will be described in detail.

CHAPTER FIVE

TESTING MATERIALS

5.1 INTRODUCTION

This chapter describes the materials used in this investigation and the reasons for the selection of these materials. It is divided into two main sections, soils (soils from Egypt and the U. K), and stabilizers (ground granulated blast furnace slag and lime). Two test soils were used. The first, described as the test soil was designed to have similar properties to a typical Egyptian clay soil. However, in order to facilitate the identification of clay, GGBS and lime reaction products in stabilized soils a second "pure clay" test soil was designed in which the very low reactivity, non clay minerals were omitted.

5.2 TYPICAL EGYPTIAN CLAYS (GEOGRAPHY AND MINERALOGY)

5.2.1 INTRODUCTION AND GEOGRAPHY

Egyptian clayey soils are mainly located in the Nile Delta and valley. This area comprises the land under the influence of the River Nile and mostly consists of cultivated land. The River Nile is located in Egypt running from the south (Aswan), passing through Cairo, to the north (Alexandria). After passing Cairo the Nile pursues a north-westerly direction for about 20 km, and then the Nile is divided into two branches, the Damietta and Rosetta. Each branch is composed of very finely divided minerals with a comparatively small amount of sand. The depth of the alluvial deposits varies from one place to another. For example, the average depth in the south is 6.7 m while in the north is 11.2 m (Abdelkader, 1981).

5.2.2 MINERALOGY

Many researchers have studied the mineralogy of the alluvial clay in the Nile region. Fayed (1970), Fayed and Hassan (1970) carried out an extensive study of the River Nile Delta in Egypt. The materials used in this study were obtained from 25 boreholes drilled in the Delta sediments. The boreholes were selected to form a sampling grid all over the Delta (see figure 5.1 for boreholes location). Extensive investigations, including X ray diffraction and scanning electron microscopy, were carried out on 187 selected samples representing the variation in the soil types at different depths. They reported that montmorillonite and kaolinite were the predominant clay minerals in the shallow soils, with minor amounts of chlorite and illite. The non-clay minerals were found to be mainly quartz, feldspars and calcite.

Wahdan and Abdel-Aal (1977) in their extensive study on the Nile Delta soil reported that montmorillonite was the predominant clay found (40-70%) in all the specimens investigated while kaolinite was detected in small quantities (7-15%). Chlorite and mica were also found in minor amounts. Many other researchers have studied the mineralogical make-up of Egyptian clay and they have also found that the main clay mineral in Egyptian clay is montmorillonite followed by kaolinite and chlorite. Calcite, quartz and feldspars were found to be the predominant non-clay minerals, (Basta *et al.*, 1974; Abdou *et al.,* 1980; Abdelkader, 1981). Elsekelly (1987) carried out X-ray fluorescent test on two samples of the Nile Delta soils. He found that the predominant constituent is silica, followed by aluminum oxide, ferric oxide and other compounds. Calcium oxide was more abundant than sodium oxide. Therefore, the clay is mainly calcium-based montmorillonite.

Naga *et al.,* (1981) found that the clay minerals were mainly montmorillonite (50-70%) in all specimens investigated, followed by kaolinite (10-15%), mica, and chlorite. Illite was also found in minor amounts. They also observed that calcite is the predominant non-clay mineral followed by quartz and feldspars.

5.3 NATURAL SOIL

This section initially describes typical Egyptian clay soils and then the test soil manufactured using U.K. soils to simulate a typical Egyptian soil.

The main objective of this investigation, outlined in chapter four, was to evaluate the use of GGBS, with and without lime as an activator, as a stabilizer for Egyptian clayey soil used as sub-grade in the construction of roads. It is, however, difficult to transport large amounts of soil from Egypt to the U.K. Therefore, it was decided to use materials which are available in the U.K. to produce a test soil with similar characteristics to those found in Egypt.

El-Sekelly (1987) studied the effect of sodium chloride and hydrated lime used as stabilizers for Egyptian soil. He obtained some soil samples from the Nile Delta near the city of El-Mansourah, near the bank of the River Nile. The engineering properties of the samples were determined and X-ray diffraction and X-ray fluorescence tests were carried out. The engineering properties of the Egyptian clayey soils tested by Elsekelly, are given in table 5.1, The main clay minerals are given in table 5.2 and chemical analysis by X-ray fluorescence in table 5.3. Montmorillonite was found to be the predominant clay mineral followed by kaolinite and chlorite. Quartz, feldspars, and calcite were found to be the main non-clay minerals in the samples. He found that no soil in the U.K. could be a good representative of this sort of Egyptian soil. However, he found that after carrying out many trials, a combination of 80% of River Aire described in section 5.4.1 and 20% of pure calcium montmorillonite was similar in behaviour to the Egyptian clayey soil, located in Delta area, in terms of engineering properties and mineralogical composition. As the object of this research is to study the effect of GGBS with and without lime on stabilization of Egyptian clayey soils, located in Delta area, the same type of the artificially blended soil was used in this investigation as the test soil.

5.4 TEST SOIL COMPONENTS

The test soil comprised River Aire soil, commercial montmorillonite clay (Berkbond No. 1) and Speswhite kaolinite clay. The engineering properties of these soils are given in table 5.1.

5.4.1 RIVER AIRE SOIL

This soil was collected from the bank of the River Aire at Rawcliffe, North Yorkshire. It was air-dried, crushed to finer than 600 μm and placed in tins. These tins were sealed and stored until they were needed to produce the test soil. Differential thermal analysis was carried out on this soil and it was found that the main clay minerals were kaolinite, mica and chlorite. X ray fluorescence was also carried out on this soil. These results are in agreement with El-Sekelly, 1987. The engineering properties are given in table 5.1, the main clay minerals are given in table 5.2, chemical properties of soil in table 5.3 and the particle size distribution curve using the micrometric sediGraph in figure 5.1. The detail of this test including the repeatability is described in chapter 6.

5.4.2 COMMERCIAL MONTMORILLONITE CLAY (BB1)

This soil is a calcium montmorillonite and was supplied by Steetley Bentonite and Absorbents Ltd., Milton Keynes, U. K, under the trade name of Berk Bond No.1 (BB1). Differential thermal analysis was carried out on this soil and the main clay minerals were found to be montmorillonite and a small amount of kaolinite. The engineering properties are given in table 5.1, the main clay minerals are given in table 5.2, the chemical properties of the montmorillonite are given in table 5.3 and the particle size distribution curve in figure 5.2. X-ray fluorescence revealed that silicon oxide is the predominant compound and the amount of calcium oxide is five times that of sodium oxide.

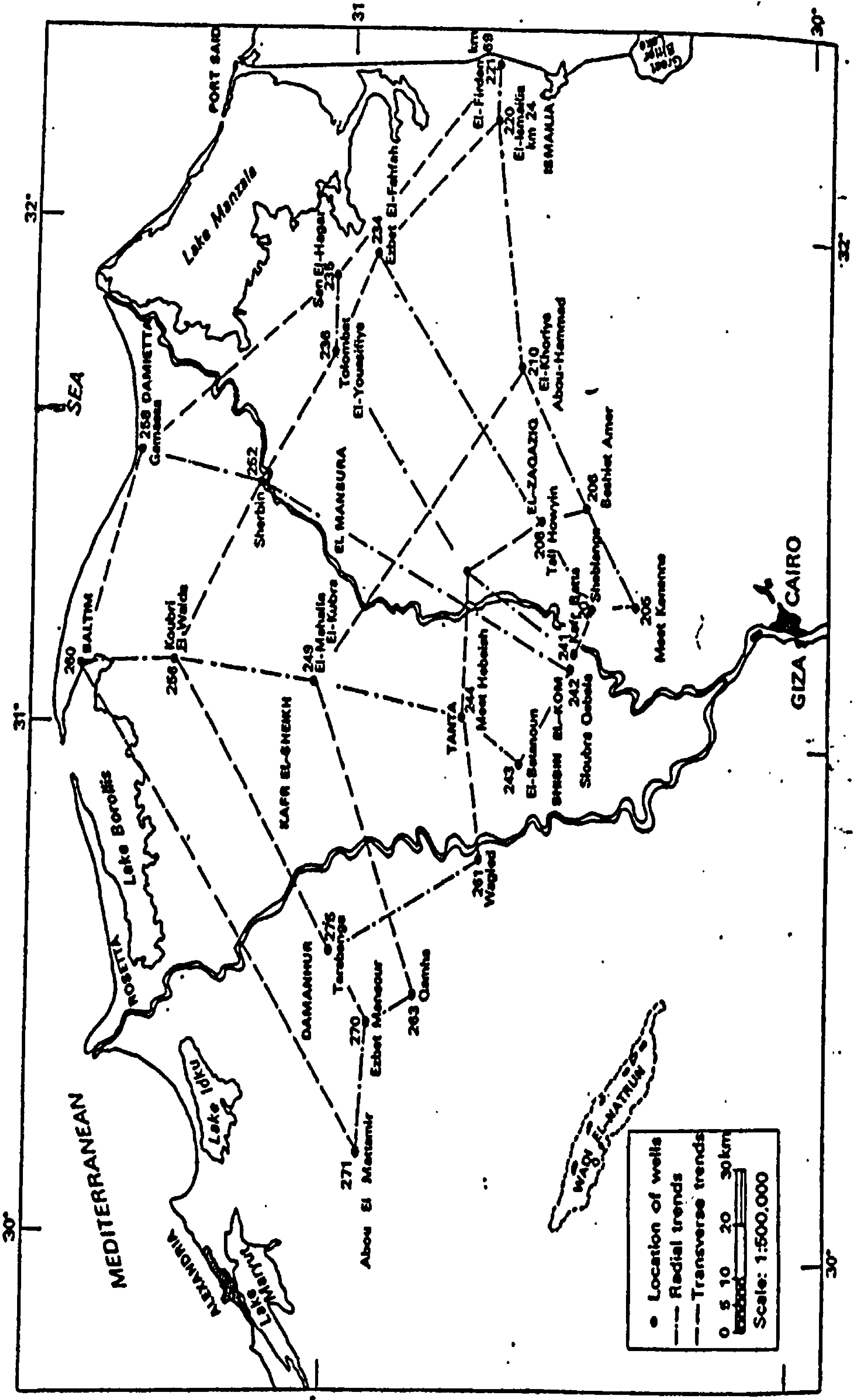


Figure 5.1 Location map of wells, after Fayed and Hassan, 1970

5.4.3 INACTIVATED CALCIUM MONTMORILLONITE CLAY

During testing it became apparent that, due to the high iron content of BB1 (10.62%), the magnetic field in the NMR tests was disturbed and it was not possible to get clear signals using this sort of clay. There were also problems with interpretation of DTA test results when using River Aire soil. Therefore, an inactivated calcium montmorillonite, with a smaller percentage of iron but which otherwise has similar physical properties, classification, grain size distribution and chemical properties, was used to produce samples of stabilised soil for use in NMR tests to identify reaction products.

This material was also supplied by Steetley Bentonite and Absorbents Ltd. Differential thermal analysis was carried out on the material and the main clay minerals were found to be montmorillonite and a small amount of kaolinite. The chemical properties of this soil are given in table 5.3. Inactivated calcium montmorillonite was used, together with Speswhite kaolinite to produce a pure clay test soil, see section 5.5 and appendix 5.1.

5.4.4 SPESWHITE KAOLINITE

This soil is pure kaolinite and was supplied by WhiteChem Ltd., Newcastle-under-Lyne, Straffordshire, U.K. under the trade name of specwhite china clay. Differential thermal analysis and X ray fluorescence were carried out on this soil and the main clay mineral was found to be kaolinite. The engineering properties are given in table 5.1.

5.5 TEST SOIL

A mixture of 80% River Aire soil and 20% of BB1 by dry weight was found to be similar in mineralogy, physical properties, chemical constituents and general engineering properties to Egyptian clayey soils (Elsekelly, 1987). Therefore it was employed as test soil. The main clay minerals are given in table 5.2 and the chemical analysis by X-ray fluorescence in table 5.3. The main clay minerals are montmorillonite, followed by kaolinite with minor amounts of illite and Chlorite, while the main oxides are silica, alumina, iron, and calcium.

This test soil was used to study the effect of GGBS, with and without lime, on the compaction properties (MDD and OMC), plasticity characteristic (LL, PL, PI) tests, the UCS and the free swelling tests, under varying curing conditions and periods. Details of these tests are explained in chapter 7.

5.5.1 BLENDING OF MATERIALS TO PRODUCE THE TEST SOIL

The mixtures were prepared by weighing out and blending dry suitable quantities of the different materials. The blending was carried out in a Hobart mixer at the lowest speed for 3 minutes and then completed by hand mixing for an additional 2 minutes to ensure homogenous mixtures. These procedures were maintained constant during the test programme. These samples were stored in sealed containers until they were required for producing test mixtures.

5.6 PURE CLAY TEST SOIL

Artificial soil mixtures have some advantages over natural soil in fundamental studies. These include avoidance of problems caused by sampling, disturbance and material availability and variability. Artificially blended soil batches can be made in the laboratory and can give properties that do not vary from one batch to another (Kinuthia, 1997). Having known constituents artificial soils give results which can be more easily interpreted using the analytical techniques.

The test soil was used for evaluating of the effect of GGBS and lime on general engineering properties. However, as explained in section 5.4.3, the montmorillonite used in the test soil meant that NMR data was difficult to interpret. The inactive, non-clay minerals, in the test soil also meant that DTA and XRD tests were difficult to interpret due to the noise caused by these non-reactive minerals. Also, in using the test soil the reaction products formed a relatively low percentage of the total soil. Therefore, a second test soil (described as a "pure clay test soil"), comprising the reactive clay mineral components of the test soil (but replacing BB1 with inactivated calcium montmorillonite for the reason given in 5.4.3) was developed for use in XRD, SEM, DTA and NMR tests. This resulted in easier analysis of the clay-GGBS-lime reaction products. The soil comprised 55% inactivated montmorillonite and 45% specwhite kaolinite.

Table 5.1 Engineering properties of the natural soils

Property / type of soil	U. K soils			Egyptian soils*	
	River Aire soil	Commercial clay (BB1)	Spes white kaolinite	Sample 1	Sample 2
Textural composition	%	%	%	%	%
Gravel 20-2 mm	0	0%	0	0	0
Sand 2-0.063 mm	20%	0%	0	29	15
Silt 0.063-0.002 mm	60%	15%	20	51	45
Clay < 0.002 mm	20%	85%	80	20	40
Physical properties					
Specific gravity	2.60	2.68	2.60	2.67	2.65
Liquid Limit %	43	140	63	43	56
Plastic Limit %	N.P	88	34	27	33
Plasticity Index %	N.P	52	29	16	23
Organic matter content %	1.50	N.A	N.A	N.A	N.A
Modified proctor compaction					
Maximum dry density Mg/m^3	1.72	-	1.56	1.593	1.521
Optimum moisture content %	17.1	-	27	20.9	23
Classification					
Casagrande	MI	CE	MH	MI	MH
Textural	Silty Loam	Clay	Clay	Silty Clay Loam	Clay
Engineering AASHTO	A-5	A-7-5	A-7-5	A-7-6	A-7-5
Activity = P.I/ % Clay	-	0.55	0.36	0.8	0.58

* El-Sekelly (1987)

N.P= none plastic

Table 5.2 The main clay minerals of soils, El-Sekelly, 1987

Type of soil/ Minerals found	U. K soils			Egyptian soils
	River Aire soil	Commercial Clay (BB1)	Test soil	
Clay Minerals	Montmorillonite Kaolinite Chlorite Mica	Montmorillonite Kaolinite Illite Mica	Montmorillonite Kaolinite Illite Mica Chlorite	Montmorillonite Kaolinite Chlorite Mica
Non-clay Minerals	- Quartz Feldspars	Calcite Quartz Feldspars	Calcite Quartz Feldspars	Calcite Quartz Feldspars

Although Atterberg limit tests were first carried out on the portion less than 425 μm, only small changes were obtained on the addition of GGBS to the test soil due to the presence of inactive non-clay minerals. Therefore, Atterberg limit tests were also carried out on the same sample of pure clay test soil with and without GGBS and/or lime, see section 7.3.

5.7 STABILISERS

5.7.1 GROUND GRANULATED BLAST FURNACE SLAG (GGBS)

GGBS was supplied by Frodingham Cement Co. ltd, Brigg Road Scunthorpe, North Lincolnshire, U.K. It was stored in airtight plastic bags until it was required. Physical properties and chemical composition together with a typical composition for Egyptian GGBS for comparison are given in tables 5.4 and 5.5 respectively. The particle size distribution curve is shown in figure 5.5.

Table 5.3 Chemical Analysis by X-ray Fluorescence

Type of soil/	Soils from the U.K						Soils from Egypt	
Chemical found	River Aire	Commercial .Clay (BB1)	Inactivated calcium Montmorilloni	Speswhite kaolinite	Test Soil	Pure Clay test soil	Sample 1*	Sample 2*
SiO_2	62.11%	56.94 %	50.49%	46.31%	57.98%	48.61%	58.38%	53.33%
Al_2O_3	8.18	14.24	16.34	35.12	9.40	24.79	12.46	16.16
Fe_2O_3	4.29	10.62	4.60	0.68	5.56	2.83	9.80	9.90
TiO_2	0.67	0.48	0.75	0.04	0.66	0.43	2.26	1.80
Ca O	6.22	2.23	5.62	0.06	5.42	3.12	5.61	4.63
Mg O	1.57	2.68	3.20	0.12	1.80	1.81	2.69	2.83
K_2O	1.68	0.43	1.76	1.52	1.43	1.65	1.23	2.19
Na_2O	0.95	0.43	0.86	0.30	0.85	0.62	1.67	1.60
$Mn_3\,O_4$	0.11	0.01	0.09	0.01	0.10	0.05	0.14	0.15
P_2O_5	0.43	0.06	0.12	0.10	0.36	0.11	0.31	0.32
SO_3	0.70	N.A	N.A	N.A	N.A	N.A	N.A	N.A
Loss on ignition	13.54	11.70	15.20	14.58	14.59	14.90	4.58	7.25

* El-Sekelly, (1987)

In order to ensure an acceptable level of reactivity, the chemical modulus is calculated according to B S: 6699 (1986) as shown:

$$\frac{(CaO)+(MgO)+(Al_2O_3)}{(SiO_2)}$$

This modulus should be more than 1 and the lime: silica ratio (CaO/SiO_2) should also be less than 1.4.

The German standard DIN: 1164 (1967) suggests that the chemical moduli

$$\frac{CaO+MgO+(1/3)Al_2O_3}{SiO_2+(2/3)Al_2O_3}>1.0 \quad \text{and,}$$

$$1.4<\frac{CaO+1.4MgO+0.56Al_2O_3}{SiO_2}<2.2$$

should be satisfied. GGBS used in this investigation was found to be reactive and satisfied both the British standard and the DIN requirements.

Egyptian GGBS is produced by Halwan Steel Company, Cairo, Egypt. Egyptian GGBS has approximately equal percentages of silicon and calcium oxides, while the percentage of calcium and magnesium oxides are lower compared to the Frodingham GGBS. Egyptian GGBS also satisfies the BS and DIN requirements.

Table 5.4 Physical properties of the stabilisers

Property	Lime*	GGBS**
Physical form	Dry white powder	Off-white powder
Melting/decomposition temperature	580° C	
Bulk density kg/m^3	480	1200
Specific gravity	2.3	2.9
Specific surface m^2/kg	300-1500	

* Data supplied by Buxton Chemical company

** Data supplied by Appleby Group limited

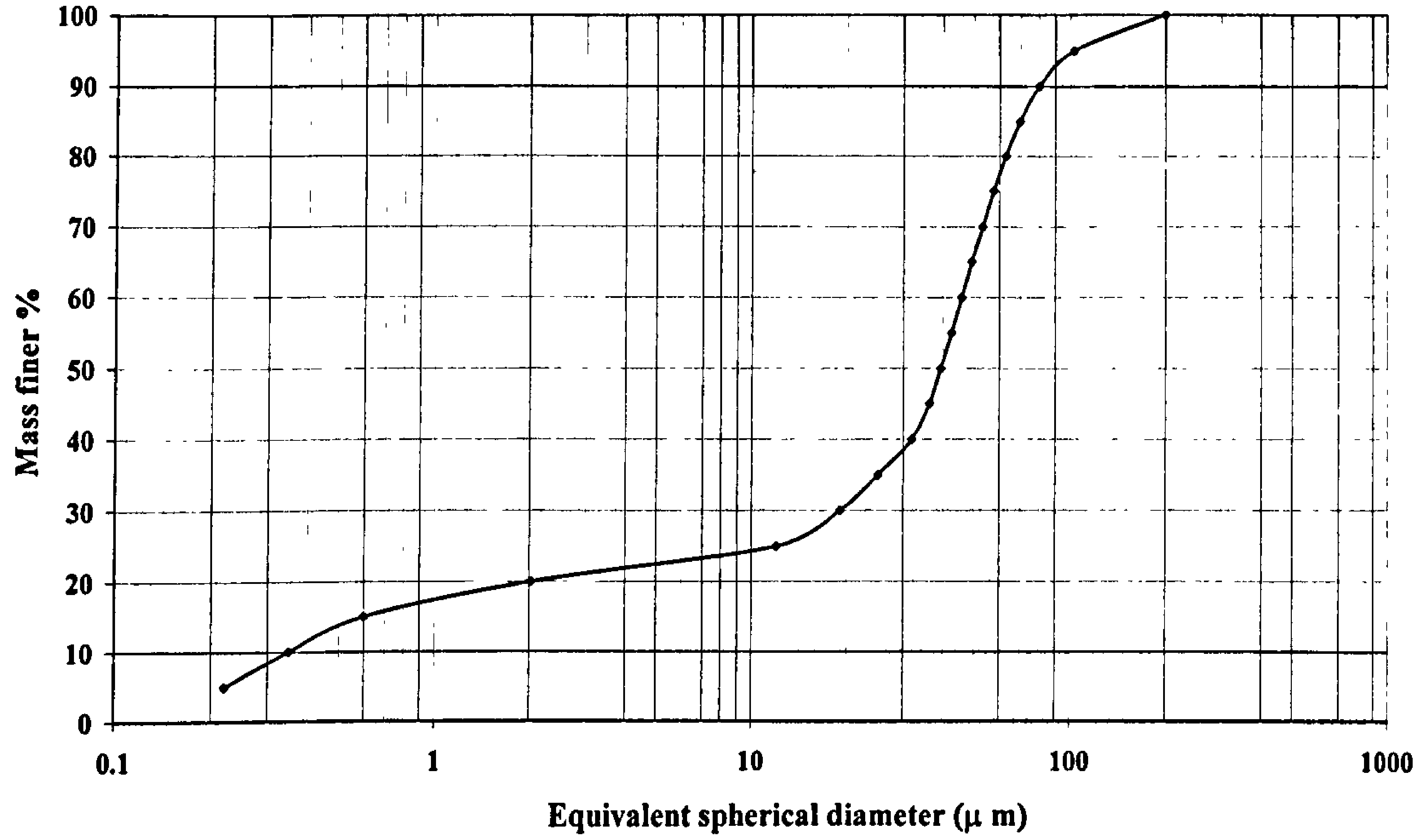


Figure 5.2 Particle size distribution of River Aire soil

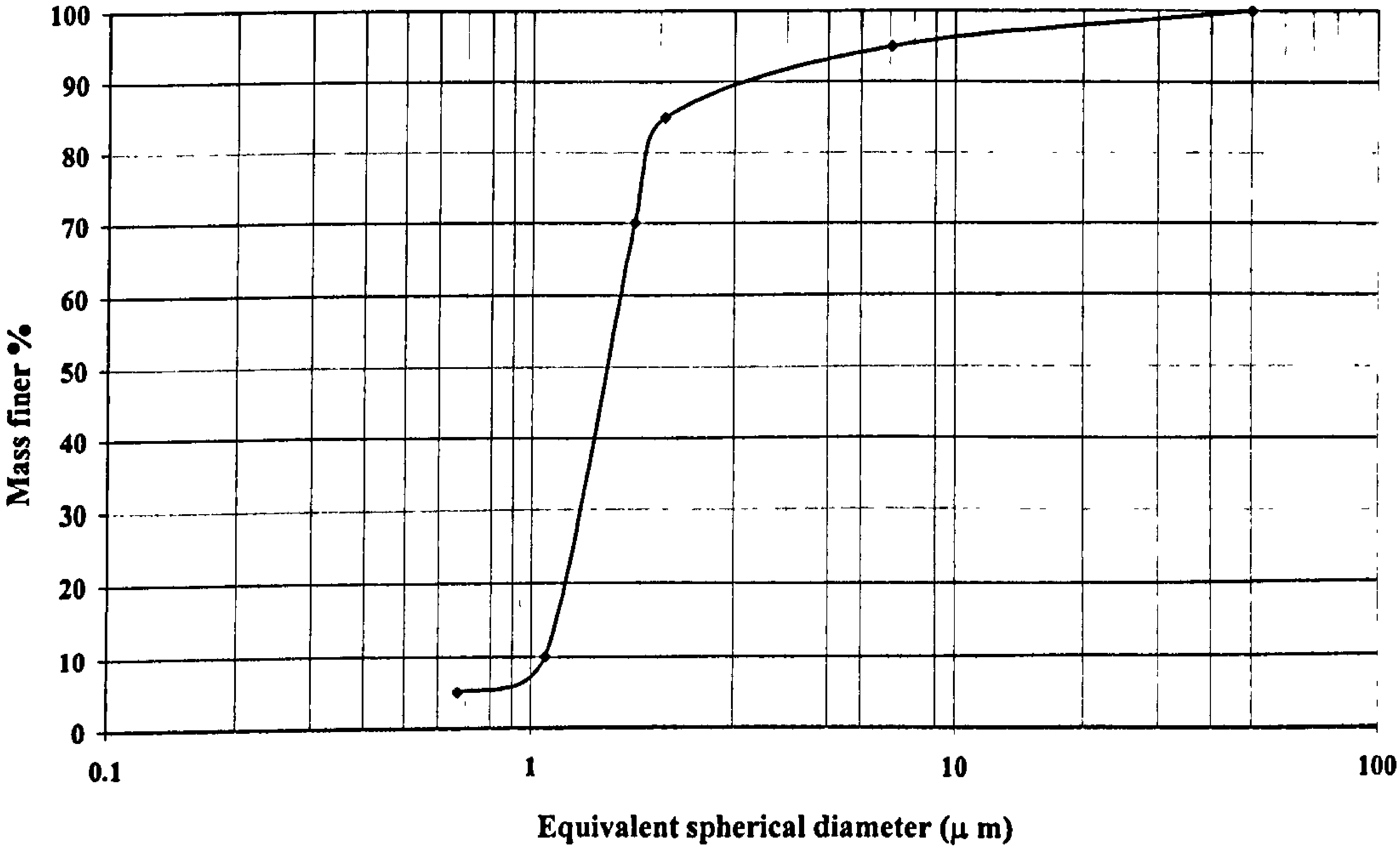


Figure 5.3 Particle size distribution of montmorillonite

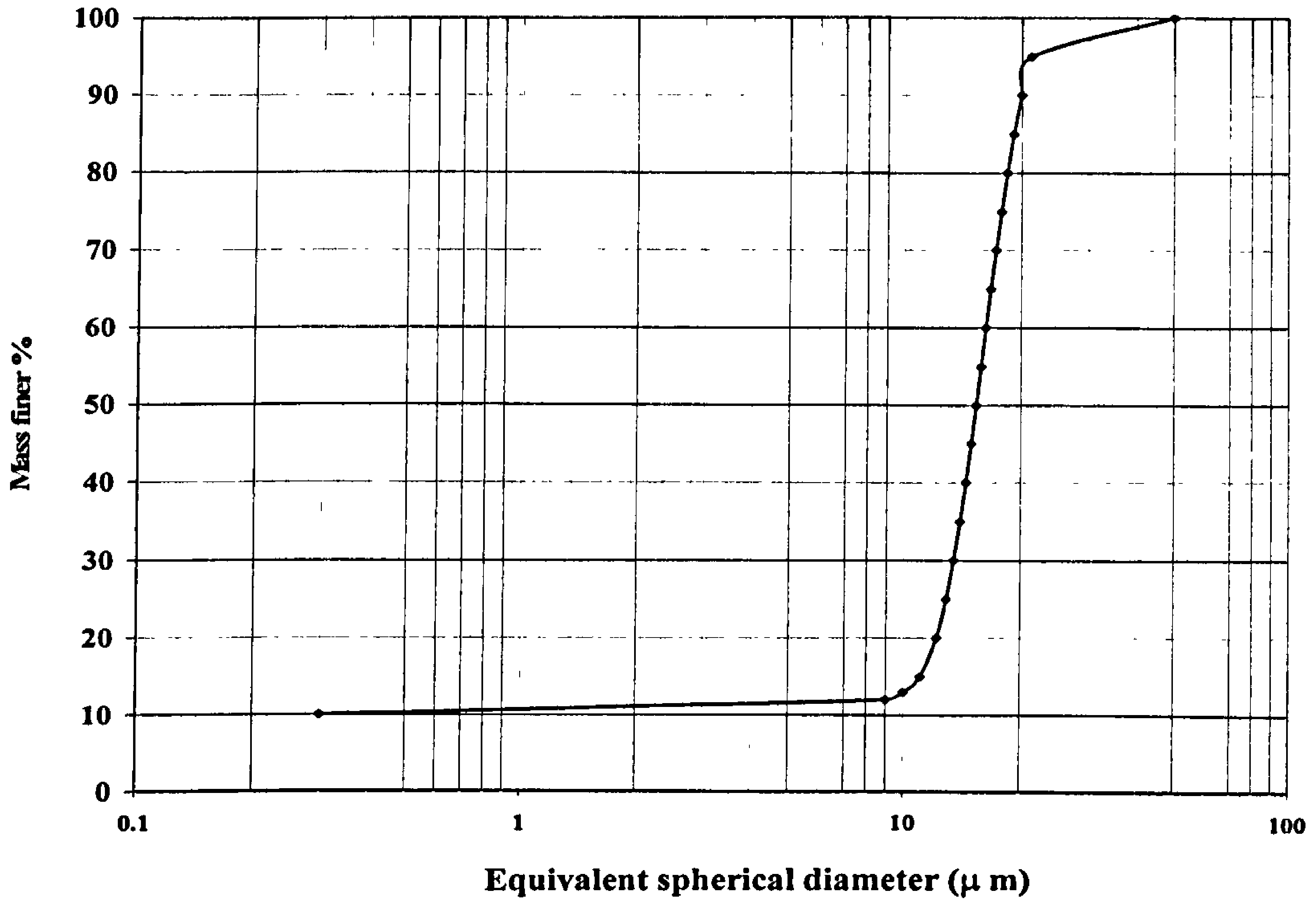


Figure 5.4 Particle size distribution of calcium hydroxide

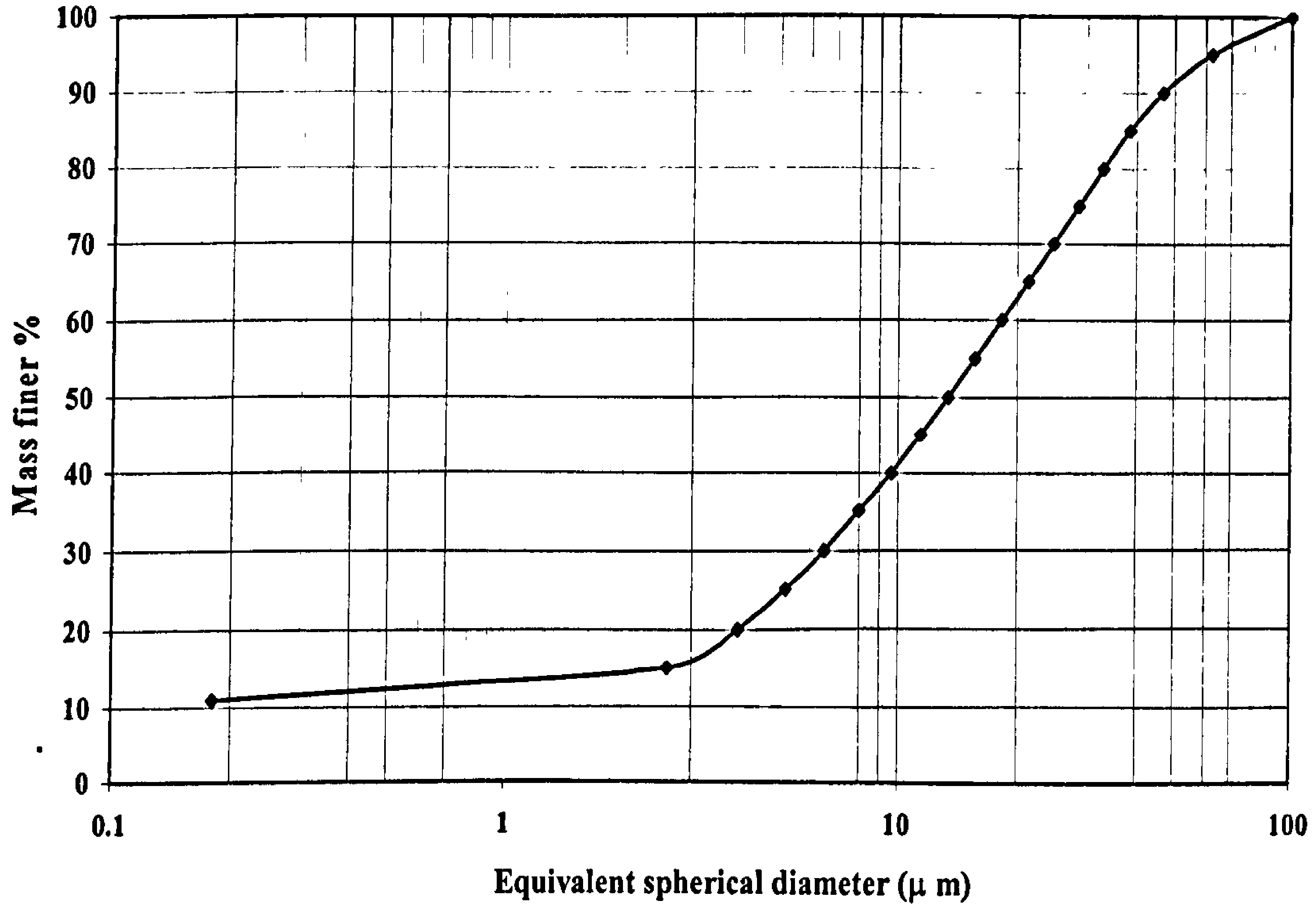


Figure 5.5 Particle size distribution of GGBS

Table 5.5 Chemical properties of hydrated lime, GGBS from the U.K and from Egypt

Composition	Lime*	GGBS**	Egyptian GGBS***
SiO_2	0.46 %	34.01 %	34.8 %
Al_2O_3	0.10	12.26	10.7
Fe_2O_3	0.06	0.46	1.2
TiO_3	0	0.52	0.6
Ca O	0	44.74	36.4
Mg O	0.83	7.33	1.9
Mn O	0	0.36	5.4
$Ca(OH)_2$	96.79	0	0
$Ca CO_3$	1.36	0	0
$Ca SO_4$	0.06	0	0
Fe O	0	0	0.75
Fe	0.60	0	0
S	0	0	0.85
Ba O	0	0	6.0
Mn	0	0	0
H_2O	0.34	0	0

* Data supplied by Buxton Chemical Company
** Data supplied by Frodingham cement co. ltd.
* ** Data supplied by Egyptian iron and steel company

5.7.2 HYDATED LIME

The hydrated lime (calcium hydroxide) used in this investigation was produced and supplied by Buxton Lime Industries Ltd. Derbyshire, under the trade name of "Limbux". It is produced in the form of a very fine powder by carefully hydrating quicklime (CaO). Hydrated lime was chosen because it is much safer to use than quick lime (calcium oxide), which can produce severe burns when in contact with skin.

The hydrated lime was supplied in 10 kg containers. It was stored in airtight plastic containers to prevent carbonation which would have affected its chemical composition and, consequently, its effectiveness as a stabilizing agent or activator for the GGBS. Physical properties and chemical composition are given in table 5.4 and 5.5 respectively, while the particle size distribution curve is shown in figure 5.4.

This chapter described the properties of the test materials used in this investigation including soils, GGBS and lime. In the next chapter the instrumentation and test procedures will be described in some detail, before describing the test results in chapter seven and eight.

CHAPTER SIX

TESTING PROCEDURES

6.1 INTRODUCTION

The objective of this chapter is to describe the principal methods of testing employed in this investigation. For practical reasons, standard tests have not been described in detail and are referenced where appropriate. Where standard procedures have been modified to suit research requirements, the modifications are described in some detail and the reasons for the modifications given. Non-standard procedures are described in some detail, and the reasons why these procedures were adopted in preference to standard methods, if such alternative methods were available, are described.

This chapter is sub-divided under three main headings: first, characterisation tests comprising organic matter content, specific gravity determination and grain size distribution. Second, engineering tests, comprising compaction tests (where the methods for obtaining the optimum moisture content and the maximum dry density of the clay-GGBS-lime mixtures are described), Atterberg limits (the methods for liquid and plastic limits testing), unconfined compressive strength tests together with sample preparation (i. e. mixing, compaction and curing), and methods for assessing the swelling characteristics are also described. Third, the techniques used to chemically analyse and characterise the specimens are described. These are X ray diffraction (XRD), differential thermal analysis (DTA), nuclear magnetic resonance (NMR) and scanning and electron microscopy (SEM). These are described to the level appropriate to the present research work.

6.2 CHARACTERISATION TESTS

6.2.1 ORGANIC MATTER CONTENT

The organic matter content test was carried out on River Aire soil only as it is natural soil and organic matter would be expected. The other materials were supplied to specified standards and did not contain organic materials. The peroxide oxidation method was used to determine the organic matter content as part of the pre-treatment of the soil before fine particle size analysis, to eliminate colloidal organic matter (Head, 1992). The method is as follows:

*100 g of soil, oven dried at 50° ± 2.5° C and passing a 2 mm sieve, was prepared

*The sample was weighed to 0.01g (m_1) and then placed in a wide-mouth conical flask.

*150 ml of hydrogen peroxide was added and the mix stirred gently with a glass rod.

*The soil was heated gently to a temperature of about 60° C, and then stirred to release any gas bubbles.

*The sample was left to allow the reaction to continue until gas no longer evolved at a rapid rate.

*The volume was reduced to about 50 ml by boiling the mixture, which also decomposed excess peroxide.

*The sample was then filtered through a Whatman No. 50 filter paper, using a Buchner funnel and vacuum flask and then washed thoroughly with distilled water.

*The soil was transferred to a weighed and dried glass-evaporating dish (mass m_2).

* The dish and its contents were dried in the oven at 105-110° C.

* The dish and its contents were weighed (m_3).

* The loss due to hydrogen peroxide treatment, the organic content, expressed as a percentage is given by

$$\text{Loss} = \frac{m_3 - m_2}{m_1} \times 100\,\%$$

6.2.2 SPECIFIC GRAVITY MEASUREMENTS

The relative densities of the raw materials, River Aire soil, montmorillonite, kaolinite, GGBS and lime, were measured. A[1] Quantachrome Corporation Ultrapycnometer–1000 was used with helium as a displacement medium. The instrument is fully automated and can measure the volume of solid objects of irregular shape whether in small pieces or in one piece. The test requires about 15-20 minutes to measure the specific gravity of a specimen. The Ultrapycnometer is designed to measure the volume and true density of solid objects. A simplified diagram of the instrument is shown in figure 6.1 while a photo of the equipment in figure 6.2.

The shaded area in figure 6.1 is the empty, sealed sample cell volume V_c. By opening the solenoid valves to the sample cell, the system is brought to ambient pressure, P_a, after being purged with helium. The state of the system is then defined as

$$P_a V_c = nRT_a \tag{1}$$

Where n is the number of moles of gas in the sample cell at P_a, R is the gas constant and T_a is ambient temperature in Kelvin.

When the solid sample of volume V_p is placed in the sample cell, equation 1 can be written as

$$P_a(V_c - V_p) = n_1 R T_1 \tag{2}$$

When pressurized to some pressure above ambient, the state is given by

$$P_2(V_c - V_p) = n_2 R T_a \tag{3}$$

---

[1] Ultrapycnometer was made by Micromeritics, 1 Micromeritics Drive, Norcross, GA 300093-1877

Where P_2 is the pressure above the ambient and n_2 is the number of moles of gas contained in the sample cell. When the solenoid valve opens to connect the added volume V_a to the cell, the pressure will fall to a lower value P_3 given by:

$$P_3(V_c-V_p+V_A)=n_2RT_a+n_ART_a \tag{4}$$

Where n_A is the number of moles of gas contained in the added volume when at ambient pressure.

Substituting in equation 4, P_aV_A can be used in place of n_ART_a, giving

$$P_3(V_c-V_p+V_A)=n_2RT_a+P_aV_A \tag{5}$$

From equations 3 and 5

$$P_3(V_c-V_p+V_A)=P_2(Vc-V_p)+P_aV_A \tag{6}$$

Or

$$(P_3-P_2)(V_c-V_p)=(P_a-P_3)V_A \tag{7}$$

Then,

$$V_c-\ V_p=\frac{(P_a-P_3)V_A}{P_3-P_2} \tag{8}$$

Adding and subtracting Pa and rearranging gives:

$$V_p=V_c-\frac{(P_a-P_3)V_A}{(P_3-P_a)-(P_2-P_a)}$$

$$=V_c+\frac{V_A}{1-\dfrac{P_2-P_a}{P_3-P_a}} \tag{9}$$

Assuming all pressures are in gauges

$$V_p=V_c+\frac{V_A}{1-\dfrac{P_2}{P_3}} \tag{10}$$

The instrument calibrates V_A by performing two pressurizations, once with the cell sample cell empty ($V_p=0$) and once with a calibration sphere V_{cal} in the sample cell. Equation (10) for these two conditions can be written as

$$V_p = 0 = V_c - V_A \frac{1}{\left(\frac{P'_2}{P'_3}\right) - 1} \tag{11}$$

and

$$V_p = V_{cal} = V_c - V_A \frac{1}{\left(\frac{P_2}{P_3}\right) - 1} \tag{12}$$

Combining equation (11) and (12) yields:

$$V_A = \frac{V_{cal}}{\frac{1}{\left(\frac{P'_2}{P'_3}\right) - 1} - \frac{1}{\left(\frac{P_2}{P_3}\right) - 1}} \tag{13}$$

The instrument calibrates V_c for a cell containing the sample holder and the appropriate calibration sphere of volume V_{cal}. Equation (10) is used, which can be written as:

$$V_p = V_{cal} = V_c + \frac{V_A}{1 - \frac{P_2}{P_3}} \tag{14}$$

The volume of cell can be found using equation (14), and then the specific gravity of the solids can be found using the volume of the solid particles and the accurate weight of the sample given by an accurate balance.

To run the specific gravity test, a typical amount of material, such as soil, lime or GGBS, depending on the grain size, are weighed and placed into the cell chamber. The air and moisture are then removed from the cell chamber automatically by the pycnometer, which performs a number of purge cycles of charging and discharging helium into and out of the chamber. Five runs are carried out for each sample and after each run the pressure change of the helium in the calibrated cell and calibrated expansion volume of the cell is used for measuring the sample volume. The average of these five runs is taken as the volume of the sample and the standard deviation

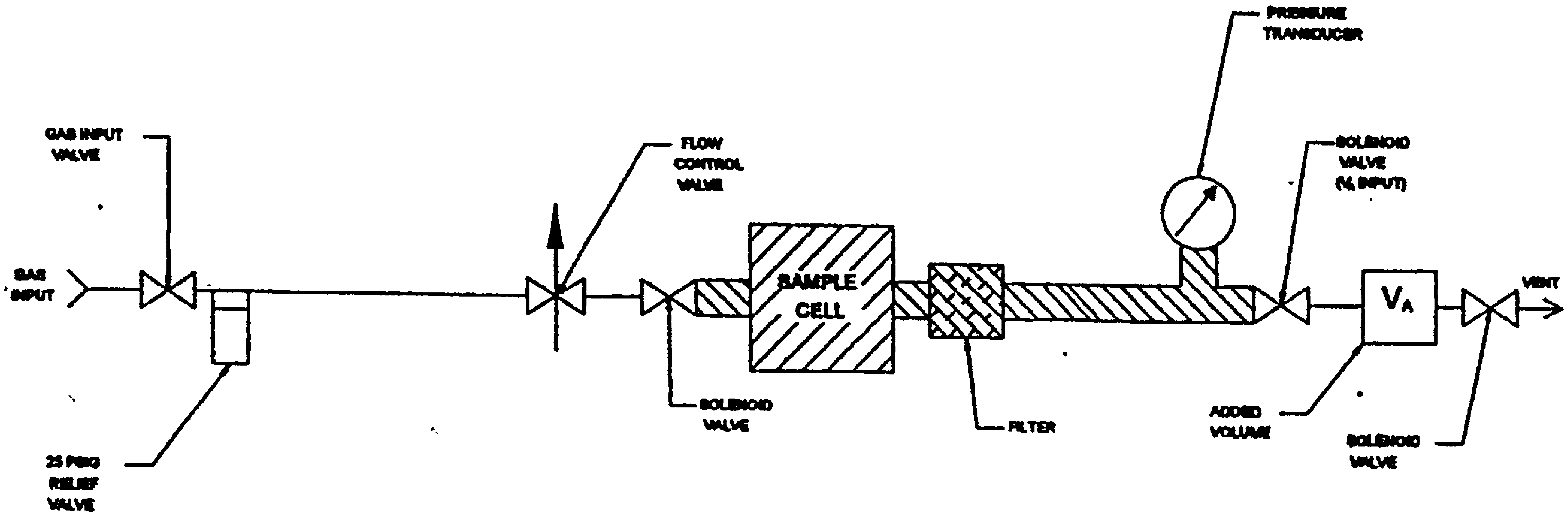


Figure 6.1 Ultrapycnometer flow diagram, user manual, 1997

should not exceed 0.01, and the pycnometer then gives the specific gravity of the sample.

Figure 6.2 Ultrapycnometer used for specific gravity measurements

6.2.3 GRAIN SIZE DISTRIBUTION

The equipment employed in determining the grain size distribution was the SediGraph 5100, made by Micromeritics[2]. The SediGraph 5100 Analyser, sieve analysis, and hydrometer measurements give particle size distributions on a mass (weight) basis. When the particles are irregular, close agreement between different techniques should not necessarily be anticipated. Sieve analysis must be very carefully performed with fine particles to ensure complete separation of particles without damaging the sieves. Even so the size determined is not necessarily the same as that measured by the SediGraph. The SediGraph 5100 Analyser is used in this

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[2] SediGraph 5100, was made by Micromeritics, 1 Micromeritics Drive, Norcross, GA 300093-1877

investigation as it is accurate, quick and easy to use compared to using the hydrometer (Micrometrics SediGraph 5100 user manual, 1990).

The minimum diameter that can be analysed using the normal sieve analyses is 63 μm, while the SediGraph Analyser determines the size of all particles under 300 μm to a minimum size of 1μm. Also, the sieve openings are usually square, elongated particles sometimes pass a given sieve in one orientation and at other times are retained (Micrometrics SediGraph 5100 user manual, 1990). Therefore, the SediGraph is considered to be more accurate than sieve analyses.

The SediGraph 5100 Analysis system consists of a particle size analyser and a computer. It is designed for completely automatic operation. However, a sliding transparent door on the front panel allows convenient observation of, and access to, the temperature-controlled analysis compartment. It contains an internal fixed position X-ray source/detector assembly and a vertical cell movement assembly. It incorporates a complete system for the circulation of the sedimentation liquid and particle sample mixture between the cell, the external mixing chamber, an external sedimentation liquid container and an external waste container, see figure 6.3.

6.2.3.1 Method of Analysis And The SediGraph System Operation

a) The sedimentation method of analysis

Generally, the sedimentation method derives its results from the rate that different size particles settle in a liquid due to the effect of gravity. Rate, as used here, is the distance that a particle settles in a certain period of time as described by Stoke's Law. The rate of settlement of large particles is faster than for smaller particles.

The falling rates for various particle sizes are calculated before analysis using Stoke's Law. "A vertical distance and an elapsed time are selected for each falling rate such that a particle falling at a certain rate will fall a certain distance in a selected time. Relative particle concentration is then measured at the selected distance and time" (Micrometrics SediGraph 5100 user manual, 1990).

Figure 6.3 SediGraph analyser 5100 connected to the computer

Firstly, a homogenous mixed suspension of particles is produced in a container by rapidly stirring the mixture. The relative concentration of particles are measured at the selected vertical distances from the top of the container and at the selected elapsed times after stopping the stirring.

The particle size associated with each concentration measurement is the size of the largest particle present at the height and time of the measurement. All particles larger than that size have higher falling rates, and will have fallen to a lower point in the container. Smaller particles are still present at equal concentration just above and below the specified point. Thus, the concentration measured at the specified point is the concentration of particles smaller than or equal to that size. This series of relative concentration of particles smaller than various sizes is the particle size distribution (Micrometrics SediGraph 5100 user manual, 1990).

b) SediGraph 5100 system operation

The material to be analysed is dispersed in a liquid (distilled water + sodium hexa-meta-phosphate) and then poured into the mixing chamber at the front of the analyzer. A magnetic stirrer located under the mixing chamber keeps the particles suspended until the analysis time. At the appropriate time the analyser transfers the suspension from the mixing chamber to the analysis cell. Particles are allowed to settle inside the analysis cell under gravity. The SediGraph 5100 determines the particle size distribution.

An X-ray beam is provided from a source inside the analyser, as shown in figure 6.4. A unit which detects X-rays is placed directly opposite the X-ray source. The analysis cell is placed in the path of the X-ray beam, between the X-ray source and the detector. Sedementing particles inside the cell cross the path of the X-ray beam. The particles in the path of the beam absorb X-rays. The amount of X-ray absorption is determined as a percentage of the X-ray absorption with the highest particle concentration for that sample. This percentage is related to the maximum particle size above that point in the cell.

The SediGraph 5100 system uses both particle falling rates and the amount of X-ray absorption for particle size analysis. Particle falling rates are used to determine the points in the cell beyond which certain size particles have fallen. X-ray absorption is used to determine the percentage of the total particle mass at different points in the cell (Micrometrics SediGraph 5100 user manual, 1990). A sample of kaolinite was analysed using both the hydrometer and the SediGraph Analyser and there were no major differences between the two results.

Each sample was analysed twice in the Sedigraph and the average of the two runs was considered to be the average particle size distribution of the specimen. The average curve did not deviate by more than 5 % from each single test.

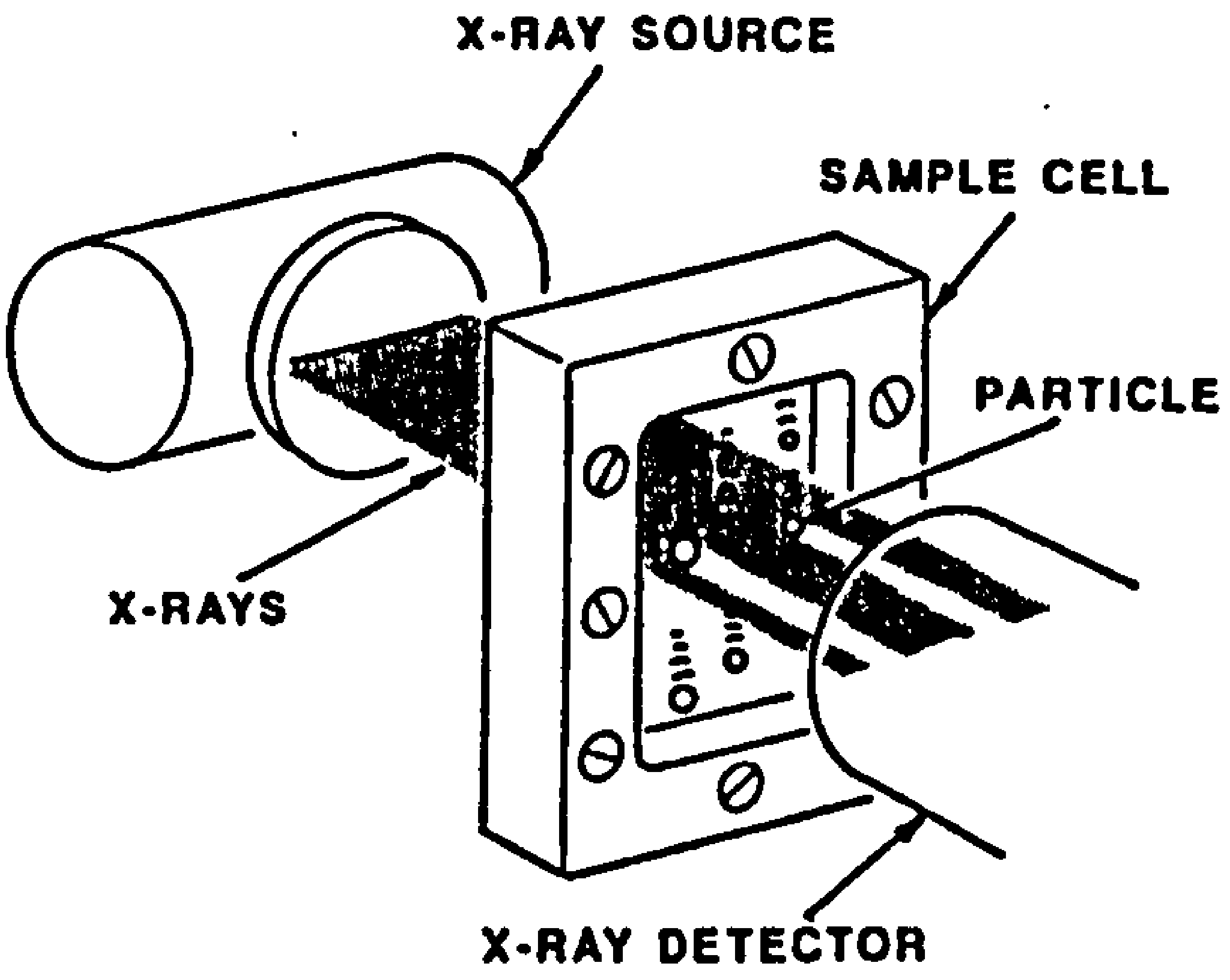

Figure 6.4 SediGraph 5100 Particle Size Analysis, (Micrometrics SediGraph 5100 user manual, 1990)

6.2.4 X Ray Fluorescence

Chemical analysis by X-ray fluorescence (XRF) tests were carried out to determine the percentages of the oxide contents in the raw materials. These materials include calcium montmorillonite (BB1), inactivated calcium montmorillonite, Speswhite kaolinite, River Aire soil, GGBS and lime. Percentage of sulphate content was only carried out on the River Aire soil as it is a natural material and sulphate is expected. Small amounts (about 40 gram) of fine powder passing through a 75 μm mesh of each materials were collected and labelled for XRF tests. The XRF tests were carried out in the School of Earth Sciences, University of Leeds.

6.3 PHYSICAL TESTS

6.3.1 COMPACTION (MODIFIED PROCTOR) TEST

The first phase of this study involved a detailed investigation of the compaction characteristics of the test soil containing different GGBS and lime contents, in order to obtain the optimum moisture contents and maximum dry densities. The optimum moisture contents were used in preparing samples for unconfined compressive strength tests and was maintained constant at the optimum value for each mixture, to give the maximum dry density and to maintain consistency of the results. Initial mixing of the dry materials and then of the dry materials with water was carried out using a Hobart mixer.

Test 3, BS 1377:1990 was adopted for the determination of the optimum moisture content and the maximum dry density. This test covers the determination of the mass of dry soil per cubic metre when the soil is compacted over a selected range of moisture contents, covering that giving the maximum. Air dried soil passing a 2 mm B. S. sieve was mixed thoroughly with the required amount of binder (GGBS + lime) for 2 minutes and mixing was continued for a further 2 minutes while the required amount of distilled water was added to the mixture. This mixing time (4 minutes) was found to be enough to obtain homogenous mixtures and it was kept constant during these investigations.

In this test, compaction is achieved by dynamic means using an automatic Proctor/CBR compaction machine. The test employs a 4.5 kg rammer falling through a height of 450 mm. A cylindrical metal mould with an internal diameter of 105± 0.5 mm and a height of 115.5 ±1 mm is employed. The mould is filled in five equal layers, with each layer being subjected to 27 blows. After weighing the mould together with compacted materials two small sub-samples are taken for moisture content determination. The average of the two is considered to be the sample moisture content. At least five dry density-moisture content values are obtained for each sample.

The mixtures investigated consisted of the test soil containing 0, 2, 4, 6, 8 and 10% of GGBS expressed as a percentage of the dry weight of the test soil. Additional tests were carried out to study whether the optimum moisture content and the maximum dry density were influenced by lime addition. The mixes investigated contained 10, 20 and 30% of lime expressed as a percentage by weight of the GGBS. The optimum moisture content and the maximum dry density obtained were used to calculate the weight of the materials required to produce a cylindrical specimen of known maximum dry density, knowing the volume of the cylindrical mould for UCS testing.

6.3.2 ATTERBERG LIMITS

6.3.2.1 Liquid Limit

Atterberg limit (consistency) tests were performed on the raw materials (River Air soil, montmorillonite and kaolinite) and the test soil. The tests were extended to the test soil mixed with 2, 4, 6, 8 and 10% by dry soil weight of GGBS. The Atterberg limits were also performed on a pure clay test soil, consisting of 55 % pure calcium montmorillonite and 45 % Speswhite kaolinite (see appendix 5.1) and pure clay test soil mixed with 2, 4, 6, 8 and 10% by dry weight of GGBS. The Atterberg limits were also measured for 2, 4 and 6 % binder (GGBS +lime) with 10, 20 and 30% by weight GGBS replacement by lime and pure clay test soil mixed with 2, 4, 6, 8 and 10% by dry soil weight of lime only. The Atterberg limit tests were carried out in accordance with BS 1377:1990, part 2 classification tests (test 4 (LL) and test 5 (PL)).

In the case of River Aire soil, the materials were first air dried for several days, crushed, and then sieved through a 425μm sieve as required by BS 1377: 1990. In all other cases as the materials passed a 425μm sieve, no sieving process was carried out. Normally, the Atterberg limit tests were performed in accordance with BS 1377: 1990 without any mellowing period (a period ranging from 1 to 3 days, during which the wet material is left to mix with the water much more effectively) being allowed, the test being done within twenty four hours of initial water addition. It should be noted that mellowing is not a universal procedure and is mainly used in the United Kingdom to study the effect of additives on the Atterberg limits of soils (Kinuthia, 1997).

The soil was first mixed with the predetermined amount of binder materials to obtain the combinations of mixtures described above. For each mixture a small amount of distilled water was mixed thoroughly with the dry soil mixture, in a porcelain dish, until a thick homogeneous paste was formed. All mixtures were then stored in sealed polythene bags and left to mature for 24 hours. The material paste was then mixed with more distilled water with spatulas for at least 10 minutes to give a cone penetration of about 15 mm. A small amount of this material, approximately 30 g, was set aside for the determination of the plastic limit later. Some of the remaining material was then used to fill a brass cup (56 mm in diameter and 41 mm deep) in accordance with BS 1377:1990. A metal cone (of angle 30 ± 1° and 35 mm in height) was allowed to penetrate into the material's surface for 5 seconds. After 5 seconds, the penetration of the cone was determined by a scale and dial pointer attached to the cone and then the penetration was recorded. The test was repeated and the penetration was recorded again, if the difference between the two values of penetration was less than 0.5 mm, the average value was taken as a representative value of the penetration (see figure, 6.5). If the second penetration is between 0.5 and 1 mm different from the first, a third test is carried out, and provided the overall range does not exceed 1 mm, the average of the three penetrations is recorded and the moisture content is measured. If the overall range exceeds 1 mm, the soil is removed from the cup and the test is repeated.

A small amount of material is then taken from the middle of the cup for moisture content determination. The moisture content was determined in accordance with BS 1377:1990, part 2: 3.2 using the oven drying method. After removal of material for the determination of the moisture content, the residual material in the cup was mixed with the rest of the sample and more water added. Further penetration tests, together with their corresponding moisture contents, were performed so as to obtain at least four set of points. The liquid limit was taken as the moisture content corresponding to a penetration of 20 mm, from a graph of penetration against moisture content (Head, 1992).

6.3.2.2 Plastic Limit

The material set aside after initial mixing for the liquid limit determination was gently rolled on a flat glass plate into cylinders and if they could reduced to 3 mm using a standard effort the moisture content is too high. The sample is then worked by hand to reduce the moisture content, the exercise repeated until the threads crumbed at a 3 mm diameter. The water content at this point is the plastic limit.

From the liquid limit and the plastic limit, the plasticity index of the material is obtained as the difference between the two limits (Head, 1992).

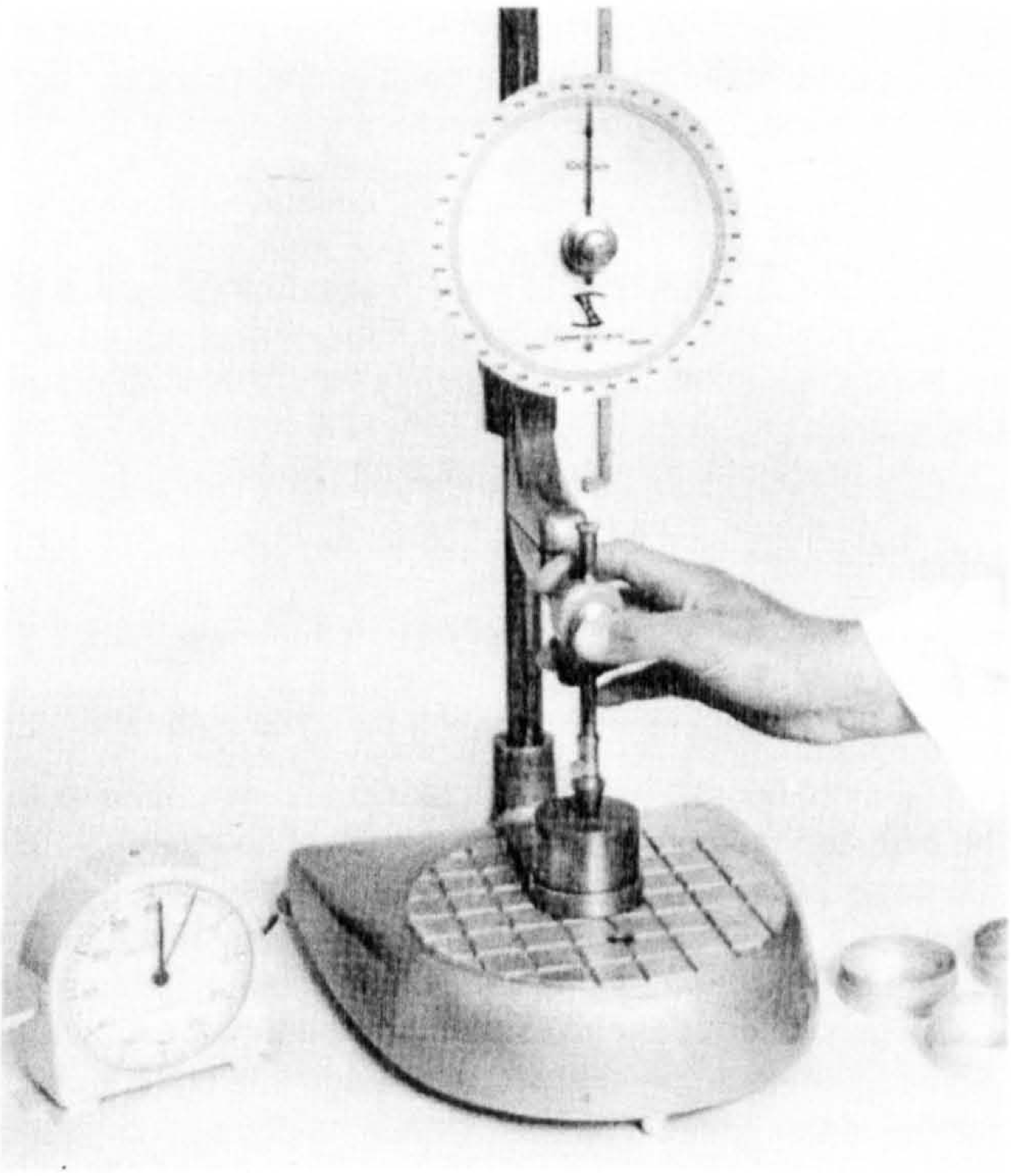

Figure 6.5 Cone penetrometer test immediately after penetration (Head, 1992)

6.3.3 pH VALUE DETERMINATION

pH determination tests were carried out on the pure clay test soil only and mixtures of the pure clay test soil and lime (2, 4, 6 and 9% lime content) and pure clay test soil with GGBS (2, 4, 6 and 9% GGBS content). The tests were also carried out on mixtures of pure clay test soil with GGBS and lime (0, 4, 6, 11, 16 and 27 % binder content) for both 20 % and 30 % GGBS replacement by lime. The pH of the pure lime solution was also determined.

Clay portions with the required amount of binder, weighing 20 grams, were placed in conical flasks, 100 ml of deionised water was added to all the flasks and each shaken manually for 30 seconds every 10 minutes for one hour. The pH of each of the solutions was measured using a Corning 240 digital pH meter. BS 1924 recommends a temperature of 25° C for pH measurements, to correct the pH measurements at room temperature to the standard temperature, the following equation is suggested,

$$pH_{25} = pH_T + 0.03\ (T-25)$$

Where pH_{25} is the pH at 25° C

pH_T = pH at the prevailing temperature (°C)

T = Prevailing temperature

The pH correction was applied to all pH values taken at the temperature of 20° C maintained in the test room.

6.3.4 SPECIMEN PREPARATION FOR UNCONFINED COMPRESSIVE STRENGTH TEST

6.3.4.1 Mixing

Mixing was performed using a Hobart variable speed mixer. The distilled water used was that required to give the optimum moisture content as obtained from the compaction tests. The optimum moisture contents were used in this investigation to obtain the maximum dry density for each individual mixture. Dry materials, enough to produce six compacted cylindrical test samples of dimension 38 mm diameter and 76 mm length, were thoroughly mixed in a variable speed Hobart 1/4 hp mixer at the lowest speed for 3 minutes before slowly adding the calculated amount of water. The mixing paddle, the bottom and the inside of the mixing bowl were scraped free of the materials and then hand mixing with palette knives was carried out to ensure a uniform dispersion and to produce a homogenous mixture.

6.3.4.2 Compaction

In order to compact the large number of specimens required and in order to produce flat ended specimens, the modified proctor method was not used. A predetermined amount of material, sufficient to produce one sample, was placed in a mould. This mould was specially designed for this purpose. The internal diameter of the mould tapered from 37.6 to 38.4mm top to top, in order to facilitate the sample extrusion from the mould, its wall thickness being 4 mm on average. Prior to filling, the inside of the moulds were lightly covered with mould oil to facilitate extrusion after compaction. This mould oil does not have any chemical interaction with the test materials. The amount of material placed in the mould was that required to achieve the previously determined maximum dry density for a height of 76 mm. The compaction was carried out using a C.B.R test machine operating at a speed of 0.9 mm/min and the material in the mould was compressed to achieve the required height. After compaction, the pressure was released and the compacted specimen was extruded using a manual hydraulic jack. The specimen was cleaned of releasing oil using tissue paper, weighed, measured, wrapped with cling film and placed in sealed double polyethylene bags to ensure minimum loss of moisture during the curing period. The specimens were then labelled and stored in the curing rooms.

6.3.4.3 Curing

Samples were separated into two batches, one was placed in the curing room which was maintained at a temperature of 20° C and a relative humidity ranging between 90-100 percent (CC1); the other half were cured at a temperature of 35° C and a relative humidity ranging between 50–60 percent (CC2). The curing times varied between 7 days and 12 months.

6.3.4.4 Unconfined Compressive Strength Of Specimens

At the end of the curing period, the specimens were taken out from the storage bags and cling film and any water trace on the specimens was removed. The specimens were then weighed to the nearest 0.01g. The end surfaces of specimens to be used for

UCS testing were carefully adjusted using fine sandpaper if required (without a major effect on the specimen height) to ensure a good contact with the testing rig platens. All the specimens were tested in a universal test machine (Hounsfield type H 20 K-W) with a loading rate of 1.2 mm/min, to allow the specimen to fail in about 5 minutes, until failure (see figure 6.6 and 6.7). A minimum of three specimens of each mixture were tested and the average result recorded. The accuracy and repeatability are described in detail in chapter seven. Representative samples were taken from the core of the failed specimens for free swelling tests, and for further analysis.

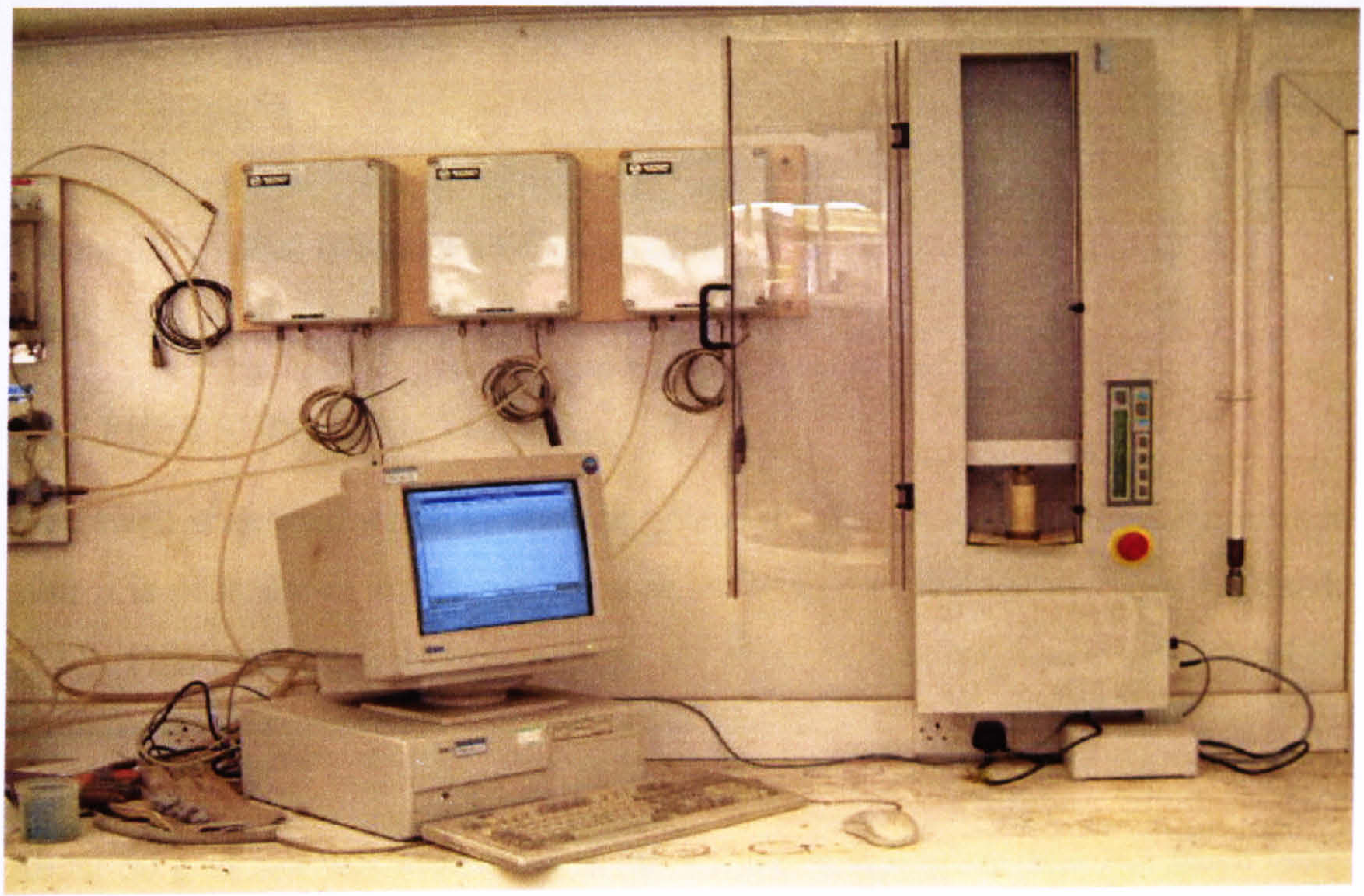

Figure 6.6 The universal test machine (Hounsfield type H 20 K-W) fitted with the computer

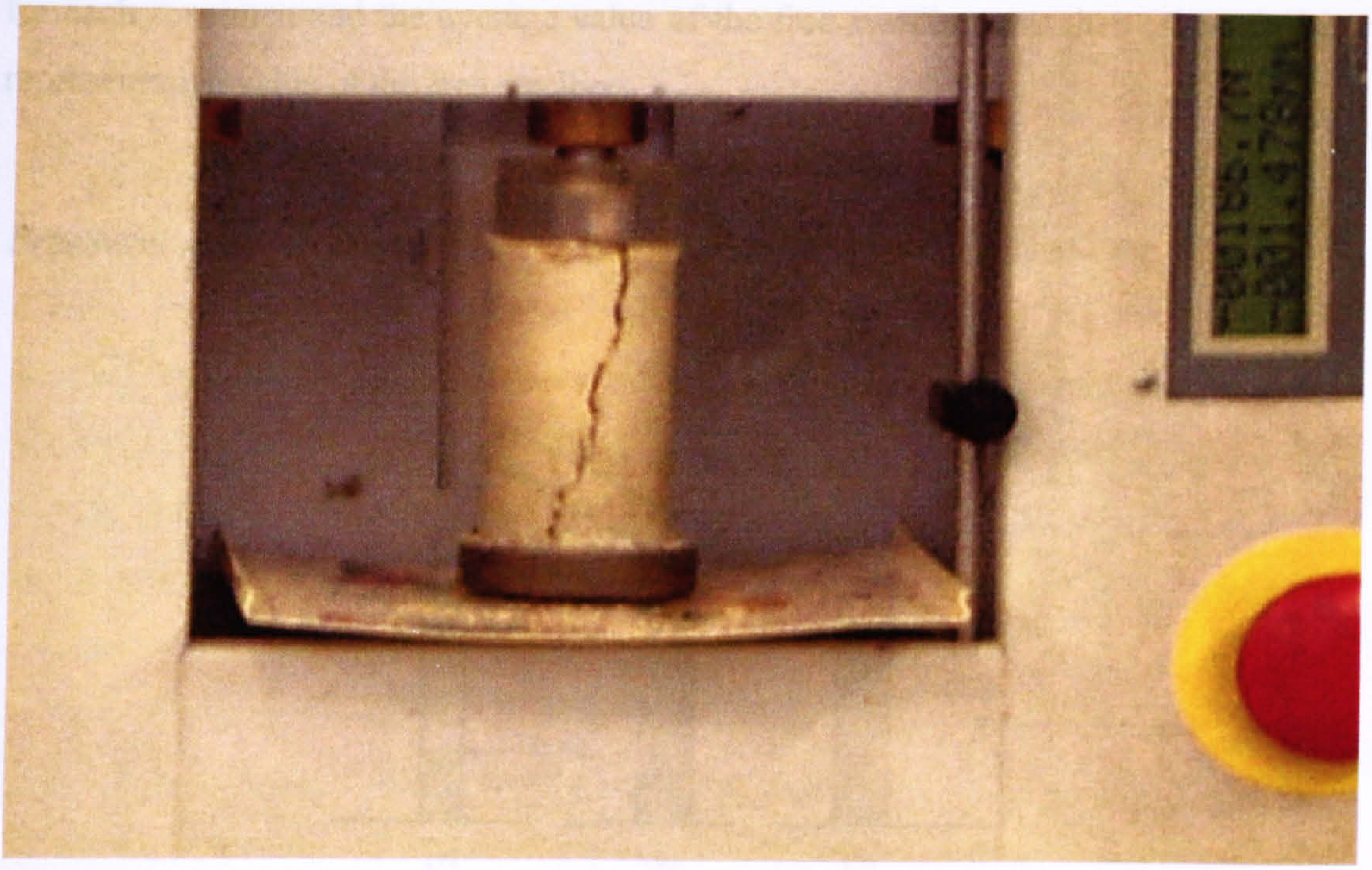

Figure 6.7 Typical failure of a UCS specimen

6.3.5 FREE SWELLING TESTS

Free swell is defined as the increase in volume of the soil from a loose dry powder after it is poured into water, expressed as percentage of the original volume (Head, 1992). Free swelling tests were carried out on all specimens that were subject to unconfined compressive strength test as described below.

About 50 g of soil was oven dried at 50° C, (this temperature does not much have effect on the properties of montmorillonite and kaolinite) ground and passed through a 425μm sieve. The soil powder was then placed loosely in a dry 25 ml cylinder up to the 10ml mark without any compaction or shaking down. 50 ml of distilled water was placed in a 50 mm diameter measuring cylinder. The dry soil powder is then poured slowly into the water. The water and soil were then left for at least half an hour to allow the main part of the solid to come to rest, (see figure 6.8). The volume of settled solids was then measured (Vml). Free swell was then calculated from the equation below (Gibbs and Holtz, 1956; Head, 1992). The test was repeated twice for each specimen and the average value of the free swelling was then taken as the representative value of the free swelling.

$$FreeSwell = \frac{V-10}{10} \times 100$$

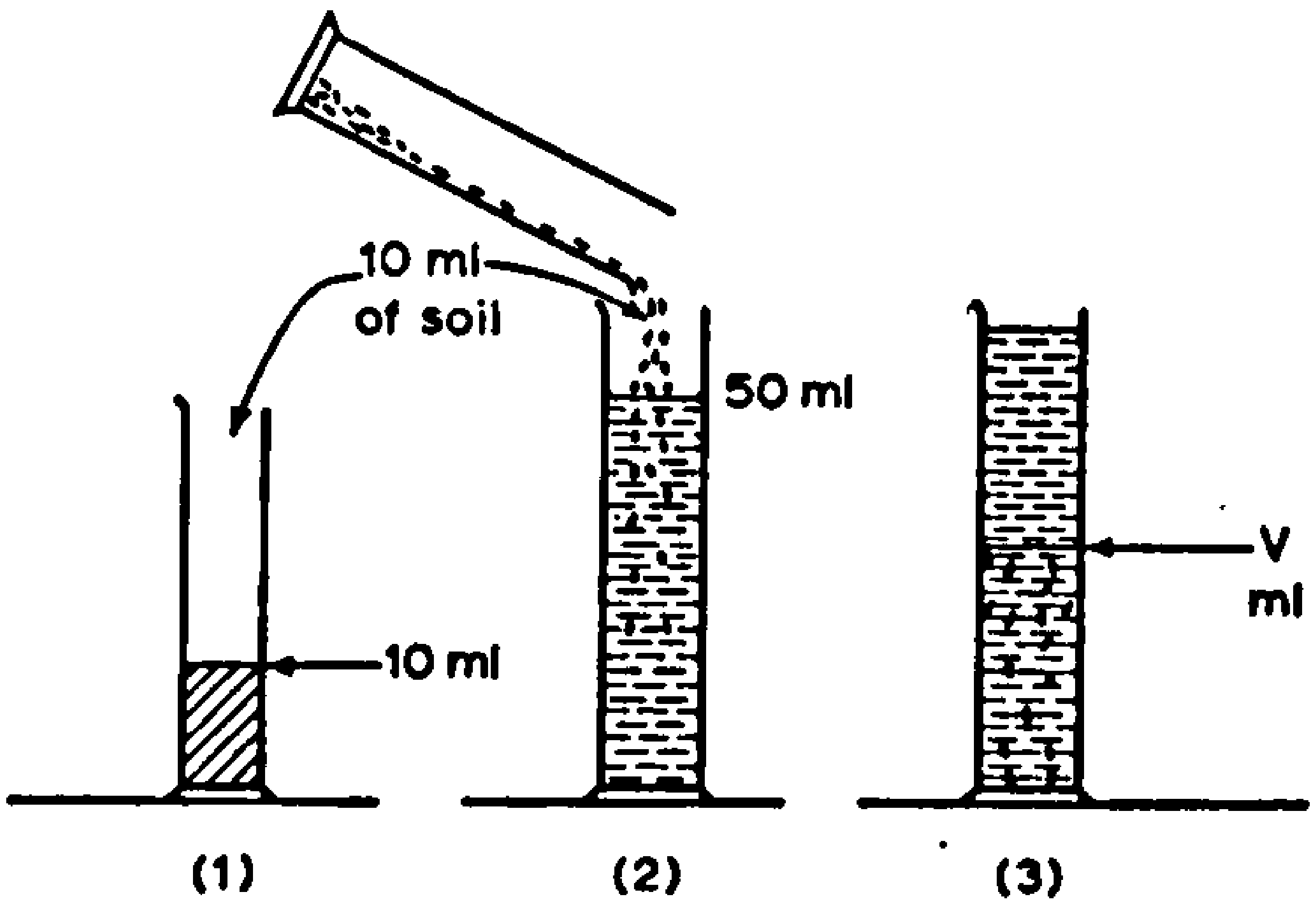

Figure 6.8 Free swell test (Head, 1992)

6.4 ANALYTICAL TECHNIQUES

6.4.1 INTRODUCTION

To understand the changes which may take place in a soil when it is stabilised with GGBS and GGBS activated by lime, it is essential to understand the mineral component of the raw materials, to identify and characterise the reaction products formed and to characterise the micro-structural changes which occur. Various analytical methods are now available for the identification and quantitative and micro-structural (morphological) studies of materials. In the current study, X- ray diffraction (XRD), differential thermal analysis (DTA), nuclear magnetic resonance (NMR) and scanning electron microscopy (SEM) were employed to provide some indication of the principal initial materials and reaction products. The following sections give a brief description of the principals, advantages, disadvantages and limitations of each technique in relation to this study at the level appropriate to its use in the current work.

6.4.2 X- RAY DIFFRACTION

One of the most widely used techniques in the identification of clay minerals is X ray diffraction. This technique has been used to identify the reaction products of lime-clay mixtures especially after the discovery of the apparent similarity of these reaction products to the hydrated cement minerals (Abdelkader, 1981). X-ray diffraction techniques have also been recently used to identify the reaction products of clay with GGBS activated by lime. The minerals of the raw materials or the reaction products can usually be identified from the diffraction lines, although it is not possible to determine the exact proportion of each mineral or reaction product in a mixture. Understanding and interpretation of the diffraction patterns are based on the fact that each crystalline material has its own characteristic atomic structure which diffracts X-ray in a characteristic pattern (Barr *et al*, 1995; Abdi, 1992). In the current work, X- ray diffraction is used solely to identify the crystalline phases in the reaction products.

Crystals can be considered to be made up of many identical unit cells, which are the smallest repeating units having the same structure (Yong and Warkentin, 1975). As the wavelength (λ) of the radiation and the distance between the atomic planes in a crystal are related, diffraction at a certain angle occurs when X-rays strike the atomic planes. The diffraction patterns are used to determine the spacing between the planes of atoms in the crystals. The spacing between the planes (d spacing) characterise the shape and size of the repeated unit cell of the crystal. Therefore, by characterising and measuring the positions of the reflections, crystals and their structures can be identified (Gillot, 1987; Abdi; 1992).

X-ray diffraction is described by Bragg's law (equation 6.15). Figure 6.9, represents a beam of X–rays reflected from parallel crystal planes spaces d units apart. Assuming two rays 1 and 2, ray 2 moves an extra length ABC. From ABC, AB = d sin θ, where d is the spacing of the crystal (d-spacing) and θ is the glancing angle. "For many glancing angles the path-length difference is not an integer number of wavelength, and the waves interfere largely destructively. However, when the path-length difference is an integer number of wavelengths (AB + BC = n λ), the reflected waves are in phase and interfere constructively. It follows that a bright reflection should be observed when the glancing angle satisfies Bragg's law" (Atkins, 1998):

$$n\lambda = 2\,d \sin\theta \text{ (Bragg's law)} \qquad (6.15)$$

Where n is an integer (1, 2, 3, etc.), and λ the X-ray wavelength. In modern work it is normal to absorb the n into d, to write Bragg's law as

$$\lambda = 2\,d \sin\theta \text{ (Bragg's law)} \qquad (6.16)$$

The main advantages in using X-ray diffraction technique over other procedures are that the test is non-destructive and only small amounts of the substances are required. The disadvantage is this technique is, however, that it is not enough to identify all the clay minerals as well as the reaction products as it is limited to identify the crystalline phases only. Also, it is not possible to identify some clay minerals that have structural similarities (Kinuthia, 1997).

In the current work, XRD analysis is limited to two specimens only, the pure clay test soil as a control sample and the control sample with 27% GGBS (20% lime), cured at 35° C after 6 months. After the required curing period, the specimens were taken out of the curing room, about 40 g of each specimen was taken out of its core. These small amounts of materials were left to dry in a vacuum oven, maintained at constant temperature of 50° C for 24 hours. The specimens then were ground in a mortar to pass through a 75 μm mesh before mounting in the aluminium sample holder of the X ray diffractometer.

The equipment used in X-ray diffraction analysis consisted of a Philips PW 1825 X ray generator with CuKα radiation of wavelength λ= 1.54179 Å, the scanning speed is 0.05 degree /second. Preparing the specimens for X-ray diffraction was carried out in the School of Civil Engineering, University of Leeds while, the X-ray diffraction tests were carried out in the School of Materials, Process and Environmental, University of Leeds. The results and the interpretations of the X-ray diffraction test results are presented in chapter eight (Analytical Test Results).

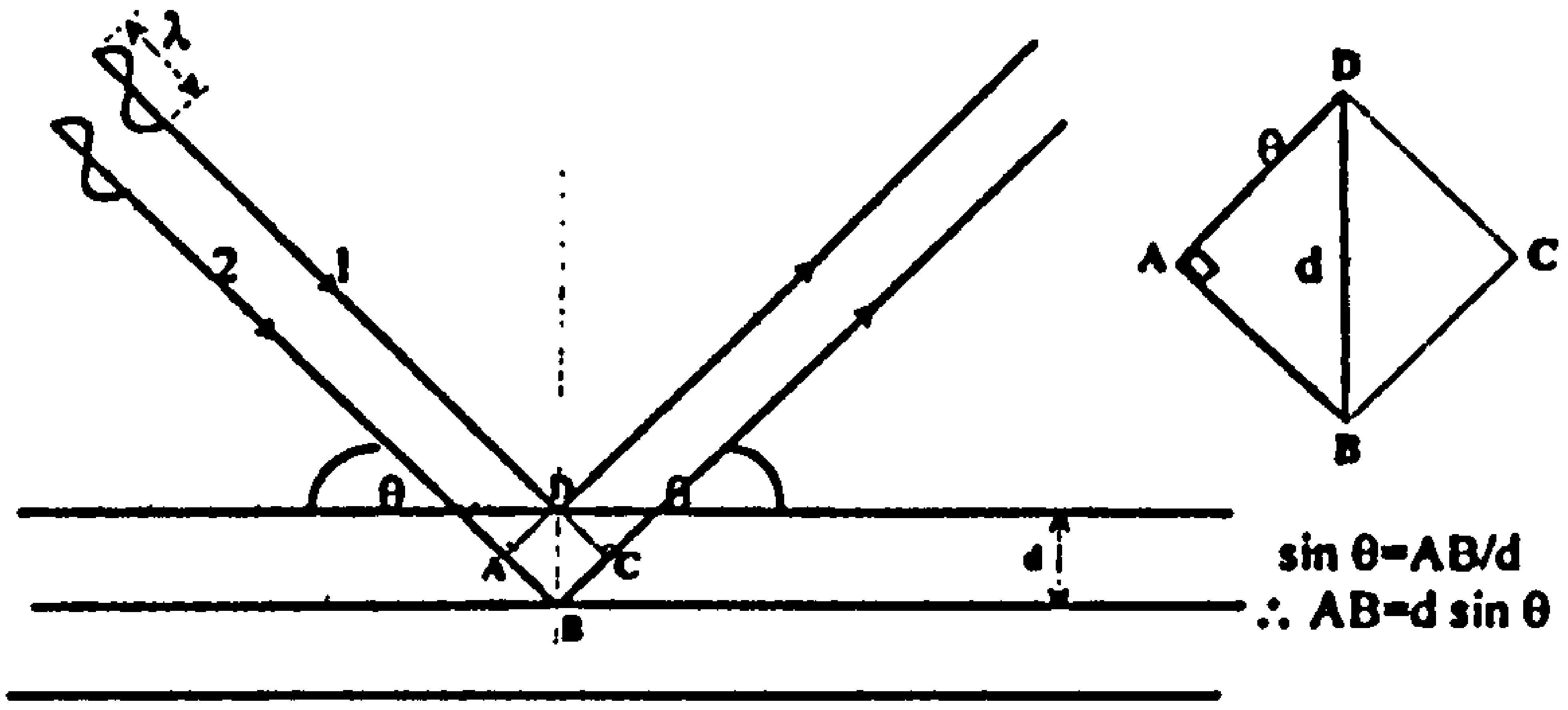

Figure 6.9 X-ray diffraction according to Bragg's law, Kinuthia

6.4.3 SCANNING ELECTRON MICROSCOPY

6.4.3.1 Introduction

Scanning electron microscopy (SEM) is a poweful tool and a common instrument in materials analysis and characterisation. It is prefered to the optical microscope when higher magnification or an increased depth of field is needed, or when some sort of elemental or compositional analysis is required. SEM is used to generate images of the surface and the subsurface of a specimen at magnifications in the range 20x-20000x. It can be used to examine the micro-structure of specimens and to determine particle crystallinity. SEM may also be used to characterise and identify particular phases and their shape and forms. It has the advantage of giving three dimensional images of superior depth of focus and resolution capabilities compared to optical micrographs (Ananta, 1999; Kinuthia, 1997).

6.4.3.2 The Nature Of Backscattered Scanning Electron Images

The modern SEM provides several different types of images, quantitative data relating to porosity and composition and the crystallographic structure of individual minerals. Electrons are emmitted from a cathode and accelerated by a voltage between the cathode and anode. A fine probe is then formed on the surface of the specimen by using a group of lenses. When a beam of primary electrons (PE) hits the surface of a specimen, they are scattered after multiple collisions with specimen atoms and they re-emerge from the specimen surface. Two groups of electrons are ejected, electrons having energy between the PE energy and (50 eV) which are the backscattered electrons (BSE). The other group of electrons, secondary electrons have energy (< 50 eV), and they leave the specimen from a thin surface layer. They can only provide information on surface topography. Electrons transmitted throught the thin specimens (BSE) give information on their internal structure (see figures 6.10 and 6.11). They give average atomic number information and crystallographic information on surfaces. They can be used for elemental and chemical analysis (Richardson, 2001).

Prior to the early 1980s, most SEM work in geology employed the secondary electron (SE) mode to examine fine surface textural details on sediment grains. This technique provides useful textural information relating to geotechnical properties. Its value is limited because the textural relationships between different minerals are not always clear. Using backscattered electrons to provide images has wider geological and mineralogical applications. The technique is very useful for study of clay comparing it with data that can be observed using conventional optical microscopy (Krinsley *et al.*, 1998).

6.4.3.3 Preparation Of The Specimens For Microscopy Study

A) Specimen Cutting and Stopping Hydration

After the desirable curing period, a small piece of undisturbed sample was cut so that it fitted inside a specific mould of 30 mm diameter. To stop hydration, the specimen was freeze dried by immersing in liquid nitrogen for a few minutes and then kept in a desiccator until resin impregnation. This method of drying should not have a major effect on the structure of the specimens.

B) Specimen Impregnation

Friable materials require impregnation with a suitable medium to impart the necessary mechanical strength for normal specimen preparation procedures to be applied (Reed, 1996). Also, the filling of pores and cavities is necessary to avoid entrapment of polishing materials. Epoxy resins are commonly used for impregnation. The viscosity of the resin is reduced by heating gently using a hair blower. The effectiveness of impregnation was improved by removing air under vacuum and utilising atmospheric pressure to force the medium into the pores. Specimens were then left for 2 days for complete setting, and taken out of moulds. The top side of specimen was then resurfaced, to ensure that the two sides of the specimen are flat and parallel, prior to grinding and polishing.

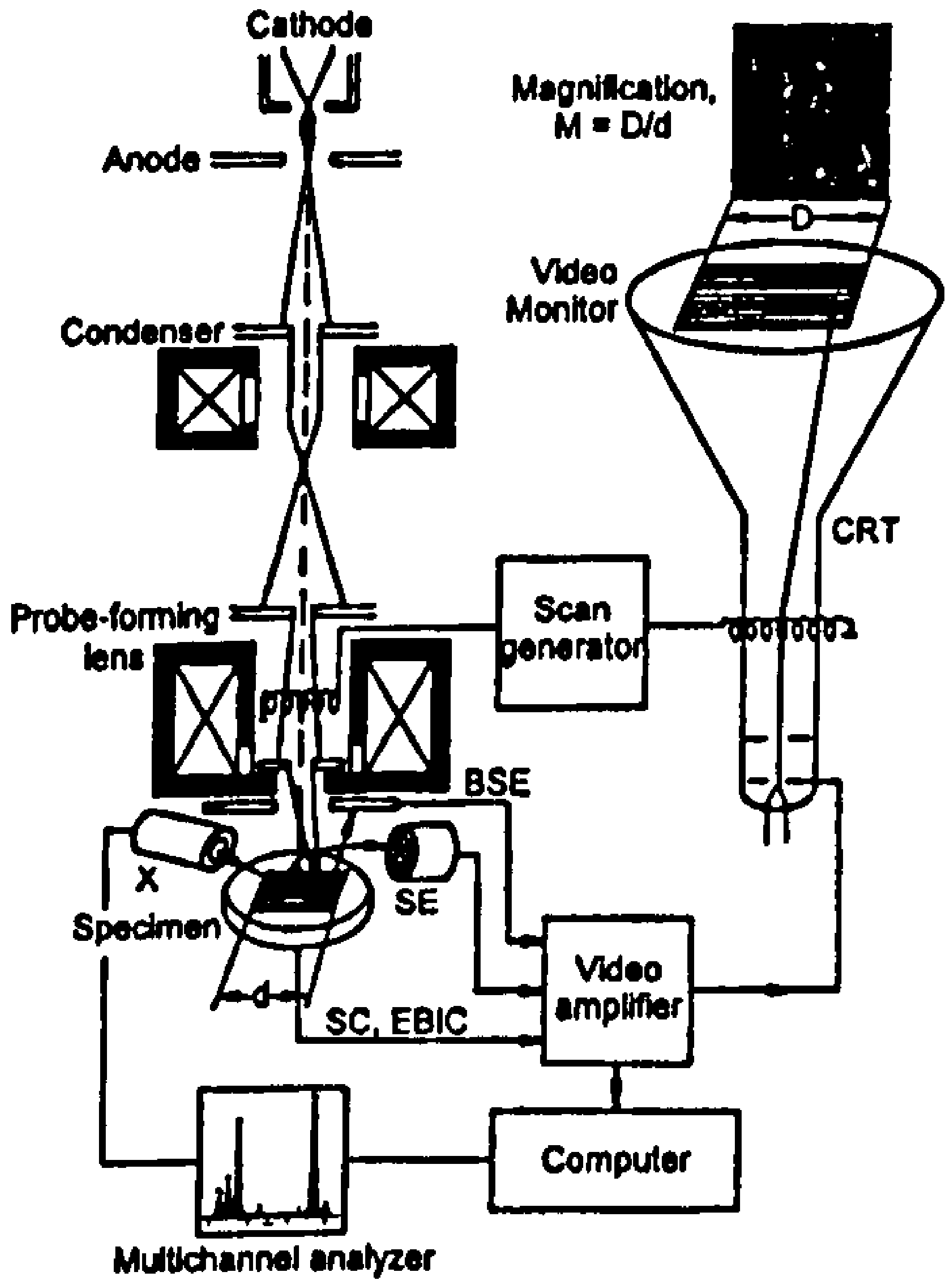

Figure 6.10 Schematic diagram of scanning electron microscope, (after Richardson 2001)

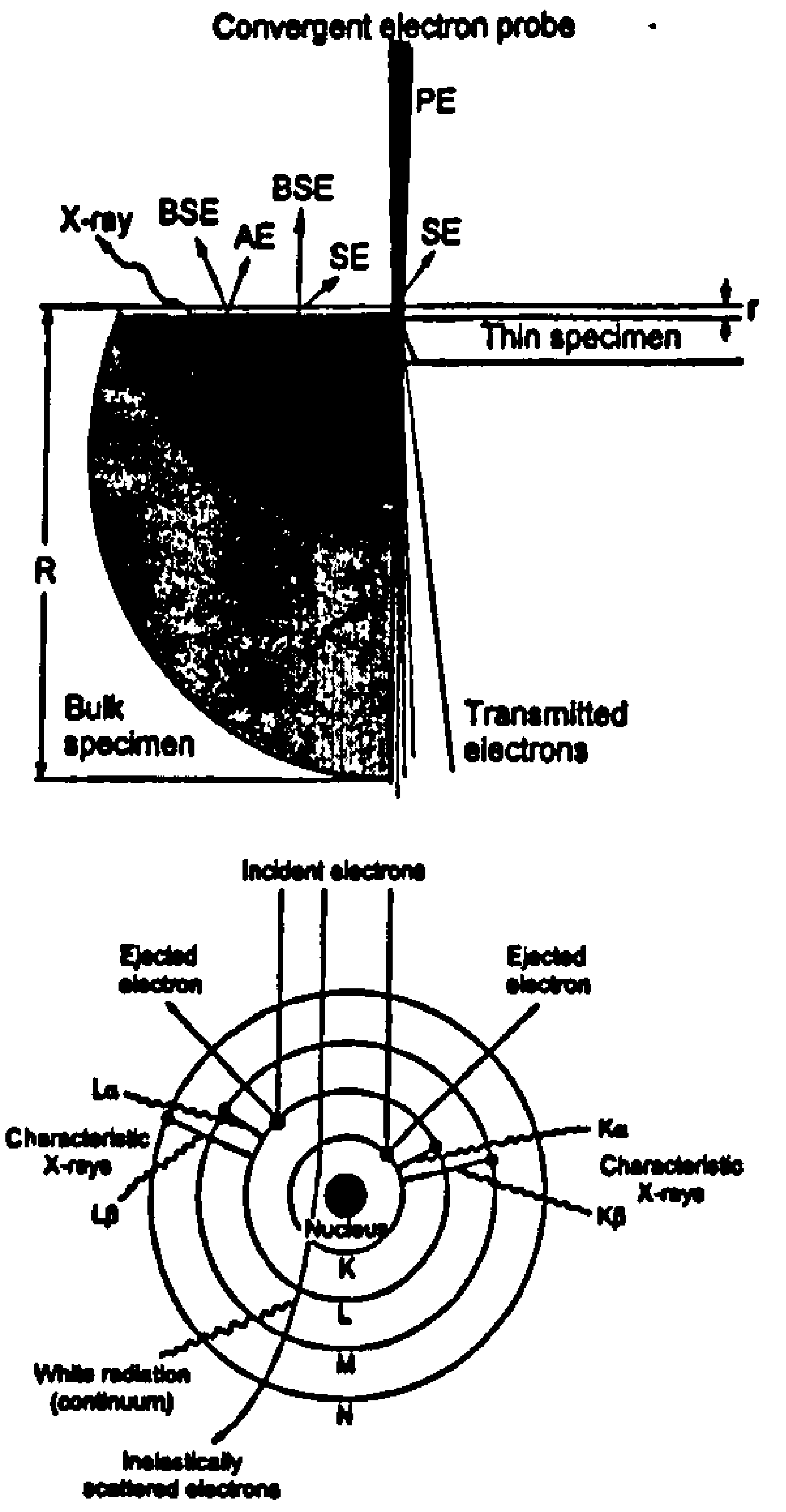

Figure 6.11 Signal resulting from interaction between a primary electron beam and a specimen, (after Richardson, 2001)

C) Grinding and Polishing Specimen Surfaces

For energy dispersive X-ray analysis and back scatter electron (BSE) imaging, it is desirable to avoid topographic effects: specimens should therefore be flat and well polished. The process of polishing consisted of two stages as follows:

1. grinding the surface of resin with various sizes of silicon carbide paper;
2. fine polishing the surface of resin with diamond paste to obtain a completely flat surface.

A hydrocarbon fluid was used as lubricant instead of water otherwise hydration would start again. Specimens were first ground using a rotational disk with silicon carbide paper using hydrocarbon fluid as a lubricant. Three sequential paper sizes, as shown in table 6.1, were used to achieve a good plane surface and between each grinding stage specimens were cleaned with hydrocarbon. The sequence of silicon carbide paper sizes employed and respective times of polishing are presented in table 6.2 (Castro Gomes, 1997).

Table 6.1 Sequential powder sizes and respective times employed for grinding

Time of grinding	Silicon carbon paper size
3- 5 min	30 μm with lubricant
3- 5 min	14 μm without lubricant
3- 5 min	10 μm without lubricant

Table 6.2 Sequential diamond paste sizes and respective times and rpm employed for polishing

Time & rpm for polishing	DP- Stick Diamond Paste Size
2-5 min & 80 rpm	6 μm
2-5 min & 60 rpm	3 μm
2-5 min & 60 rpm	1 μm

The progress of polishing was controlled by observing the reduction of scratches on the surfaces using an optical microscope. The polishing apparatus is shown in Figure 6.12. The specimen was cleaned with a hydrocarbon fluid between polishing stages in an ultrasonic bath.

D) Coating Specimens for the Electron Microscopy Study

Finally, specimens had to be coated with a conductive material for the electron microscopy study. The process consists of coating the polished surface with carbon. Under BSE detectors carbon is known to give better results compared with gold coating (Scrivener, 1986).

Figure 6.12 The grinding and polishing apparatus

The microscopy study was carried out in the Electron Microscopy Laboratory of the School of Materials, University of Leeds with a Camscan4 Electron Microscope, as shown in Figure 6.13. The examination was performed on specimens obtained from samples which were moist cured for 8 weeks.

Figure 6.13 Analysis of specimens using scanning electron microscopy

6.4.4 THERMAL ANALYSIS

The methods of thermal analysis are based on the relationship between substances and temperature, that is, on a study of the thermal response of substances and of the changes that take place in a substance depending on temperature (Todor, 1976). The two thermal analysis techniques used in soil mineralogical studies are thermogravimetric analysis (T G) and differential thermal analysis (DTA).

Thermogravimetry (TG) is an analytical technique in which the weight of a sample is continuously recorded, as a function of time, during heating or cooling under controlled conditions at a constant rate. This in turn gives information on the sample composition, its thermal stability, and its thermal decomposition. In thermogravimetric tests, the specimens are heated at a constant rate and the weight changes can be recorded in two ways as a percentage of weight loss against temperature. One method is to produce the derivative thermogravimetric (DTG) curve which is the relationship between the rate of weight change (first derivative) and the temperature (Abdi, 1992).

The pattern and amounts of weight loss can be used to both identify the compound and determine the quantity present. However, care must be taken in the interpretation and analysis of these thermograms as a number of factors influence the temperature range over which weight loss occurs and the rate of weight loss. These factors are sample size, sample packing density, heating rate, particle size, furnace atmosphere and sample crystallinity. Thus, in order to ensure consistency of results these factors must, whenever possible, be standardised (Todor, 1976).

The second approach is differential thermal analysis (DTA) which is a method of investigating phase transformation by recording time and temperature obtained during the uniform heating of a solid substance. The method consists of heating a sample and a thermally inert reference material while recording the temperature in the furnace and the temperature difference between the sample and the reference material. A differential thermocouple is used to detect the difference in temperature between the sample and the inert material with a device having one thermocouple placed in the sample and the other in the reference material, both being simultaneously heated at a constant rate. Assuming the temperature flow to be equivalent in the furnace, in the sample and in the inert material, the temperature difference between them should be equal to zero and the instrument would record a base line as a function of time and temperature if no changes occurred in the test sample. If a decomposition reaction takes place in the sample, the temperature gradient against the reference material will be modified and a temperature variation will be recorded deviating from the initial base line (Todor, 1976).

Differential thermal analysis has been employed extensively for the qualitative analysis of clay minerals. The major endotherm and exotherm peaks associated with the clay minerals occur in two temperature ranges 50-200° C, and 450-700° C. These are attributed to the loss of absorbed water and the loss of hydroxyl ions. The size of the peaks in the low temperature region reflects the amount of absorbed water and more than one peak may occur if water bound with different energies is present. Well-crystallised kaolinites generally have a small peak or none at all in this region, whereas the smectite group of minerals often have a complex peak system depending on the degree of hydration and the nature of exchange ions (Keattch, 1969; Todor, 1976).

In the present work, a 706 Stanton thermal analyser was employed, see figure 6.14. The instrument comprises a furnace, a balance and a recorder. The samples were analysed in a dry CO_2-free nitrogen atmosphere to avoid any carbonation. The nitrogen gas was passed over a drying agent (silica gel), and then over "carbosorb" before entering the furnace. To ensure the consistency of the peak temperature, a constant amount of about 16-18 mg of finely powdered sample was used, and the heating rate and flow of nitrogen were kept constant at respectively 20° C/min and 58 cm^3/min.

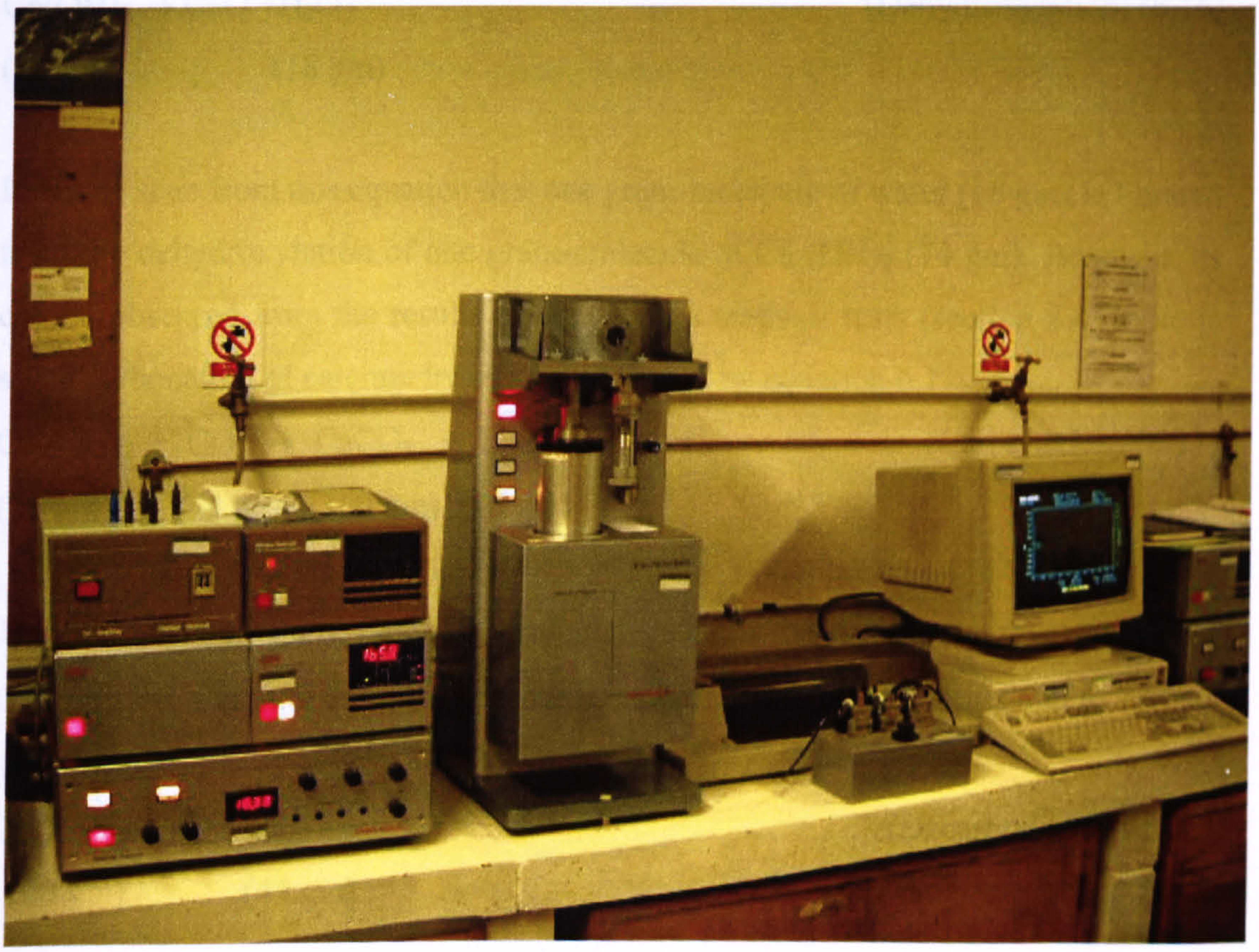

Figure 6.14 A 706 Stanton thermal analyser

To achieve consistency in the results, the sample was normally left in the furnace atmosphere for about 1 hour to reach a constant weight and hence achieve equilibrium with its immediate environment. This gave the initial weight. The sample was then heated at a rate of 20° C/min. and both the weight loss (T G) and rate of energy loss (DTA) were continuously recorded until 1000° C. The weight at 1000° C was taken as the final weight. The total weight loss was taken as the difference between the initial and final weight.

6.4.5.1 Determination Of Lime Consumption

The total amount of residual calcium hydroxide (i.e. unreacted) present in the hydrated mixtures was calculated from the weight loss due to dehydroxylation of $Ca(OH)_2$ and the decarbonation of $CaCO_3$. The dehydroxylation of $Ca(OH)_2$ occurs at a temperature range of 400-500°C, i.e. water is released at this temperature. This can be illustrated in the equation below.

$Ca(OH)_2 \rightarrow CaO + H_2O\uparrow$ (Dehydroxylation) (6.16)

(74 gm) (18 gm)

It can be seen from this equation that one gram-molecule of water (18 gm) is librated from the dehydroxylation of one gram-molecule of Ca $(OH)_2$ (74 gm). However, as can be observed from the results of the thermal analysis tests (section 8.4) there is some carbonation of calcium hydroxide as shown by equation 6.17.

$Ca(OH)_2 + CO_2 \rightarrow CaCO_3 + H_2O$ (Carbonation) (6.17)

(74 gm) (44 gm) (100 gm) (18 gm)

Therefore the amount of Ca $(OH)_2$ calculated should be modified to take account of this reaction. Calcium carbonate decomposes in the temperature range 600 to 780° C as follows:

$CaCO_3 \rightarrow CaO + CO_2\uparrow$ (Decarbonation) (6.18)

(100 gm) (44 gm)

To determine the amount of calcium carbonate in the sample, the weight of carbon dioxide produced is converted to the equivalent weight of calcium hydroxide: $CO_2 \rightarrow CaCO_3 \rightarrow Ca(OH)_2$. Equation 6.18 indicates that one gram-molecule of CO_2 (44 gm) is obtained from the decomposition of one gram-molecule of $CaCO_3$ (100 gm) and one gram-molecule of $CaCO_3$ is obtained from the carbonation of one gram-molecule of $Ca(OH)_2$ (74 gm) which therefore corresponds to one gram-molecule of CO_2 (44 gm). Therefore, the total amount of calcium hydroxide, including that which has undergone carbonation, can be calculated as follows (Cabrera and Lynsdale, 1989):

$$W\ Ca\ (OH)_2 = \frac{74}{18} \times M_1 + \frac{74}{44} \times M_2$$

Where:

$W_{Ca\,(OH)}$ = The total amount of calcium hydroxide in the sample

M_1 = Percentage of weight loss due to dehydroxylation of $Ca\ (OH)_2$ in mg.

M_2 = Percentage of weight loss due to decomposition of $CaCO_3$ in mg.

6.4.5 NUCLEAR MAGNETIC RESONANCE (N M R)

6.4.5.1 Introduction

Nuclear magnetic resonance (NMR) as a spectroscopic technique has a history of about 40 years. NMR spectroscopy has developed into an indispensable tool for chemists and physicists. It has also been employed as one of the most powerful tools for cement and concrete research. There are many elements that have isotopes with nuclear spin; if these elements are taken into account the analytical power of N M R spectroscopy is greatly increased and NMR becomes one of the most important techniques for the characterisation of inorganic compounds in the solid state (Justnes *et al.*, 1990; Engelhardt; Mitchel, 1988).

6.4.5.2 The Technique

NMR in its simplest form is the study of the properties of molecules containing magnetic nuclei by placing samples in a strong magnetic field absorbs radiowaves at frequency which can be empirically correlated with structure. Frequencies are expressed as shifts in parts per million (ppm) from a reference compound (tetramethylsilane). Solids give very broad orientationally dependent lines, but these can be sharpened by spinning very rapidly at the "magic angle". An NMR spectrometer consists of a magnet that produces a uniform, intense field and the appropriate source of radiofrequency electromagnetic radiation (figure 6.15).

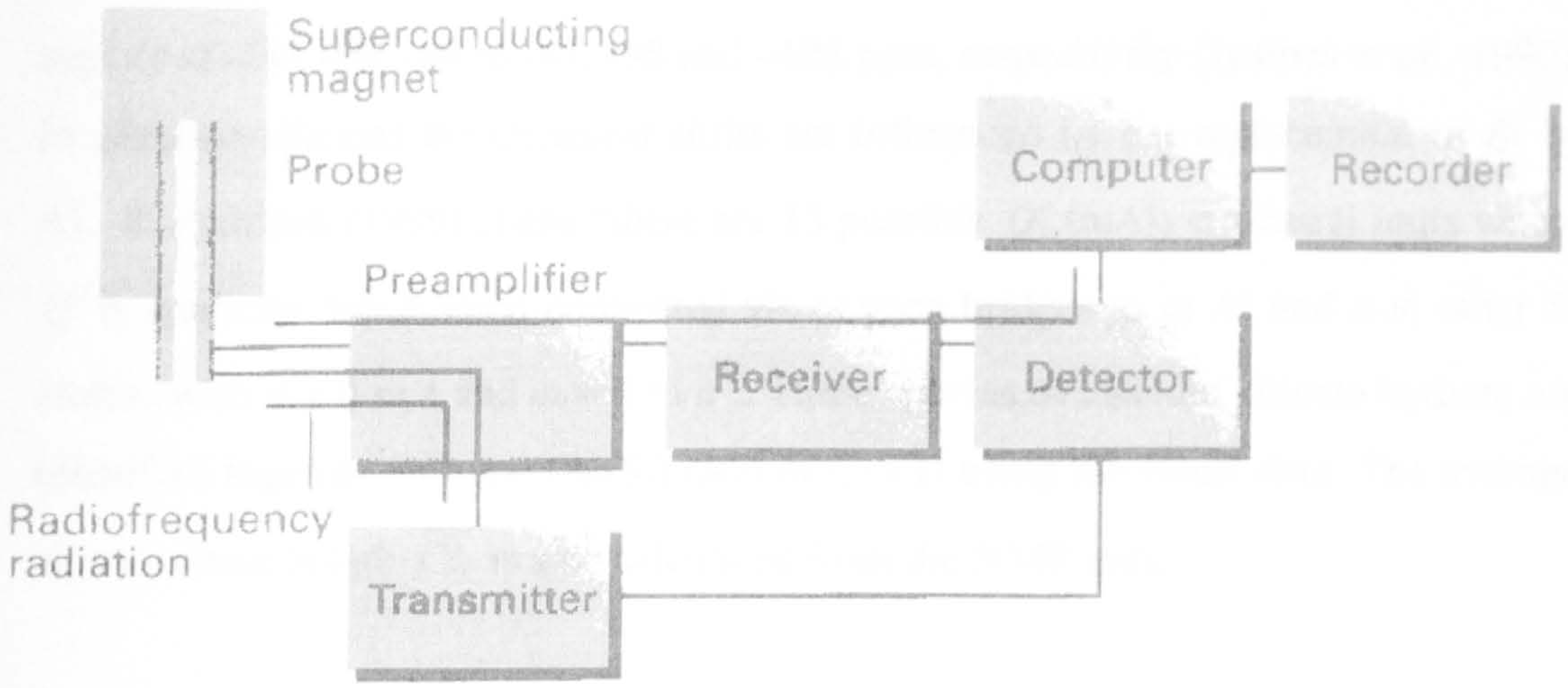

Figure 6.15 The layout of a typical NMR spectrometer (Atkins, 1998).

6.4.5.3 Specimen Preparation For NMR Testing

NMR tests were carried out on specimens of the pure clay test soil with different combinations of GGBS and lime (calcium hydroxide) and also on the control sample (pure clay test soil only without any binder). Small pieces were taken out of the middle of the test specimen. The specimens used for the NMR test, at their natural moisture content, were ground to a fine powder and packed into 7 mm zirconia rotors. Solid-state ^{29}Si MAS NMR spectra were acquired using a Bruker MSL-300 spectrometer (magnetic field 7.0 tesla, sample spinning speed 4.5 kHz, pulse length 45°, repetition time between scans 5 s, decoupling field 35 kHz). The ^{29}Si chemical shifts are quoted relative to tetramethylsilane (TMS) at 0 ppm. The specimens were tested using a solid state silicon NMR. The tests were carried out in the School of Chemistry, University of Leeds, figure 6.16.

6.4.5.4 Interpretation Of The NMR Data

Several silicates with known molecular structures have been investigated by solid state high resolution Si NMR and the following rules for interpretation of Si spectra found. Isolated SiO_4 tetrahedra (Q°) and SiO_4 tetrahedra sharing one (Q^1), two (Q^2), three (Q^3) and four (Q^4) oxygen with neighbouring SiO_4 tetrahedra lead to signals at –70, -80, -84 to -87, -98 and –108 ppm, respectively (Justnes *et al.*, 1990). In alumino-silicates the chemical shifts are influenced by the replacement of Si by Al. Richardson (1999) states "there are 15 possible Q^n (mAl) structural units where Q is a silicate tetrahedron connected via oxygen bridges to m Al and n-m other Si atoms, with $n = 0$ to 4 and $m = 0$ to n". These species of calcium silicate hydrate are identified together with the (Al/Si) ratio of C-S-H using the NMR data. The average silicate chain length $\overline{CL}$ is also calculated from the NMR data.

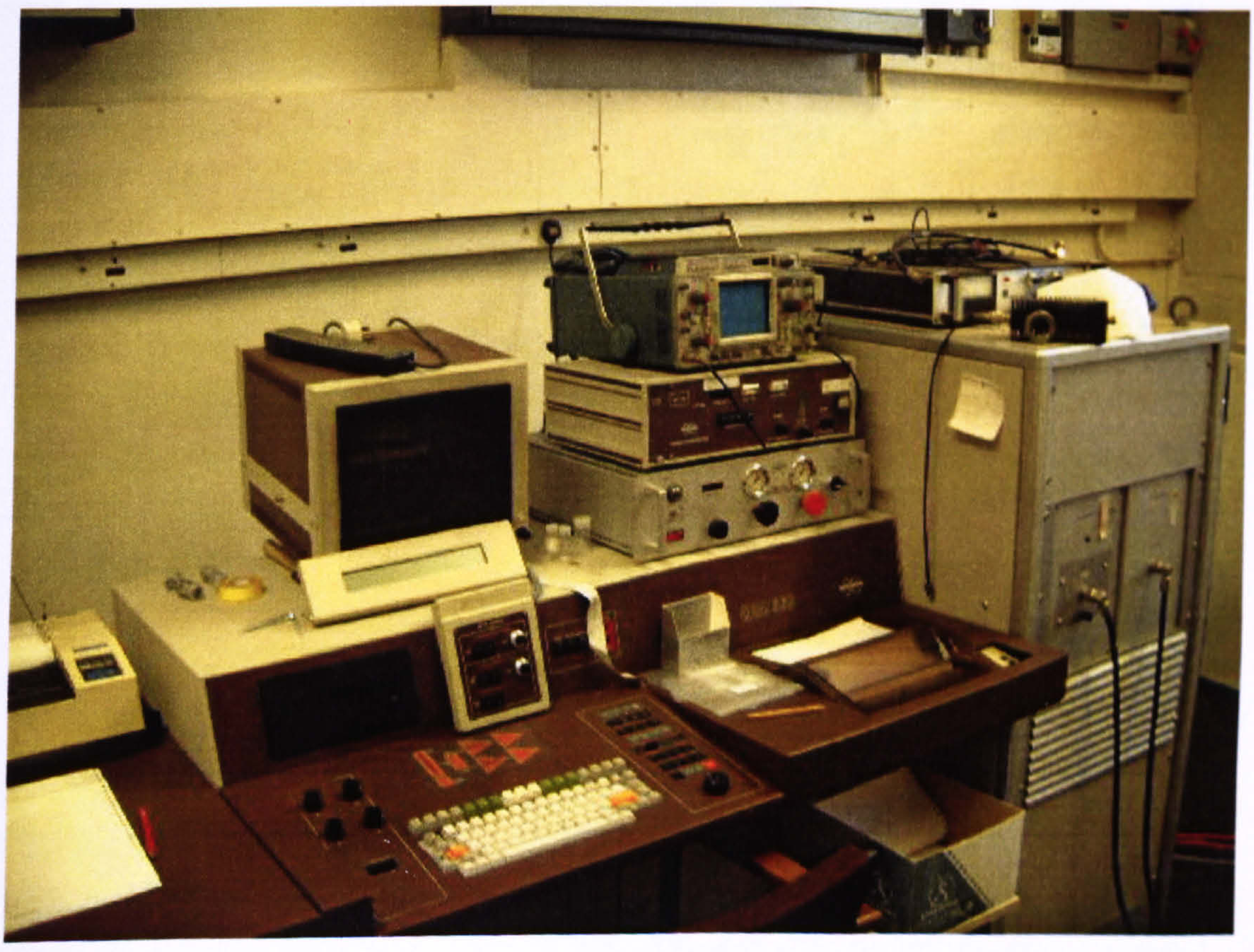

Figure 6.16 The NMR equipment

6.5 SUMMARY

This chapter describes the testing procedures using in this investigation. The tests included characterisation tests (organic matter content, specific gravity, grain size distribution and pH determination), and engineering tests (compaction, liquid and plastic limit, unconfined compressive and free swelling). Analytical techniques, X ray diffraction, scanning electron microscopy, thermal analysis and nuclear magnetic resonance, are also described.

In chapter 7 and 8 the results of these techniques and their accuracy and reliability are presented and discussed. The interpretation of the analytical test results are also described in detail and further discussion of the physical and analytical test results, including the identification of the reaction products, are presented in chapter nine.

CHAPTER SEVEN

ENGINEERING TEST RESULTS

7.1 INTRODUCTION

The effect of GGBS with and without lime on the engineering properties of the test soil, (compaction, plasticity characteristics, unconfined compressive strength (UCS) and swelling characteristics), are presented in this chapter. The UCS and investigation of swelling characteristics were conducted on specimens that were cured under two controlled conditions (see section 6.3.4.3 in chapter 6), for varying periods (i .e. 7 and 28 days and 3, 6, 9 and 12 months). The compaction tests were carried out immediately after adding the binders. The plasticity characteristic tests were carried out after 1 and 3 days after mixing. The compaction, plasticity, UCS and free swelling tests were carried out on the test soil composed of 80% River Aire soil (mainly kaolinite) and 20% calcium montmorillonite. Plasticity characteristic tests only were also carried out on a second test soil (pure clay test soil) composed of only the clay portion of the main test soil. The details of these mixtures are given in sections 5.5 and 5.6 and in appendix 1.

This chapter presents and analyses the results obtained and suggests possible explanation for the effect of GGBS, with and without lime, on the engineering behaviour of the test soil. The proposed explanations are evaluated using the analytical results in chapter eight and there is further discussion in chapter nine.

7.2 DRY DENSITY- MOISTURE CONTENT RELATIONSHIP

7.2.1 INTRODUCTION

Modified Proctor tests were used to establish the dry density-moisture content relationship for the test soil and to provide data for the preparation of specimens for unconfined compressive strength tests. Modified Proctor compaction tests were carried out on the test soil with various amounts of GGBS added to investigate the effect of adding GGBS on optimum moisture content and maximum dry density. Modified Proctor tests were also carried out on the test soil with 2, 4, and 6% total binder (GGBS +lime) by dry weight of soil. GGBS replacement by lime was 10, 20 and 30% of the total binder percent (2, 4 and 6 % of dry weight of soil). The reason for this are explained in section 7.4.

7.2.2 EFFECT OF GGBS ALONE ON THE COMPACTION CHARACTERISTICS OF THE TEST SOIL

Dry density of the test soil increased normally with an increase in the moisture content up to a maximum value of 1.78 Mg/m^3 at optimum moisture content of 19.3%, then the dry density decreased with further increases in the moisture content, see figure 7.2 and table 7.1. The general shape of the curve is normal for this type of soil. Changes were observed in the optimum moisture content (OMC) and the maximum dry density, (MDD), with an increase in GGBS content compared to the test soil only curve. The OMC increased from 19.3% to 22% while the MDD decreased slightly from 1.78 Mg/m^3 to 1.69 Mg/m^3 with an increase in the GGBS content from 0% to 10%, see figure 7.1, table 7.1 and section 9.2.1 in chapter nine. Elsekelly (1987) studied the compaction characteristics of a test soil, similar to that used above, treated with lime. He observed a decrease in the maximum dry density with an increase in the lime content. As the compaction was carried out immediately after mixing, there is no chemical reaction expected in this early stage of hydration. He suggested that the reduction in maximum dry density with increasing lime content might be a result of the replacement of soil particles by lime particles in a given volume; they partially filled the voids between the soil particles and prevented them from coming into a closer state of packing. In the current study, the same principle could be acting due to the difference in particle density between soil and GGBS. The increase in the OMC could be due to the increase in the specific surface area that has to be lubricated, soil-GGBS mixtures can be expected to require more moisture to achieve their maximum dry density than untreated soils, see section 9.2.1 for detail.

One of each set of three tests was repeated twice and the average value of the two values obtained was taken to be a representative of the true value unless the two values deviated by more than 2% from the mean. If so, a third test was carried out and the two closest values selected and checked as for the initial values.

7.2.3 EFFECT OF GGBS AND LIME ON THE COMPACTION CHARACTERISTICS OF THE TEST SOIL

The effect of the partial replacement of GGBS by lime on the compaction characteristics has also been studied. Compaction curves are shown in figures 7.2, 7.3 and 7.4. The effect of GGBS and lime on the MDD is shown in figures 7.5 and table 7.1. A further decrease was observed in the MDD with an increase in lime/GGBS ratio for a constant total binder content. For the same lime/GGBS ratio, the MDD decreased with an increase in the total binder content. For example, the MDD decreased from 1.71 Mg/m^3 to 1.65 Mg/m^3 with an increase in the lime/GGBS ratio from 0% to 30% at 6% of the total binder. The increase of lime content in the test mixture, it is thought, improved the workability and also caused flocculation of the test soil particles. Therefore, the clay particles became bigger and more voids were formed, thus reduced the dry density. Some investigators suggest the formation of cementitious products immediately after mixing clay with lime that cause resistance to compaction and reduced the density (Wild *et al.*, 1993 b, see section 2.9).

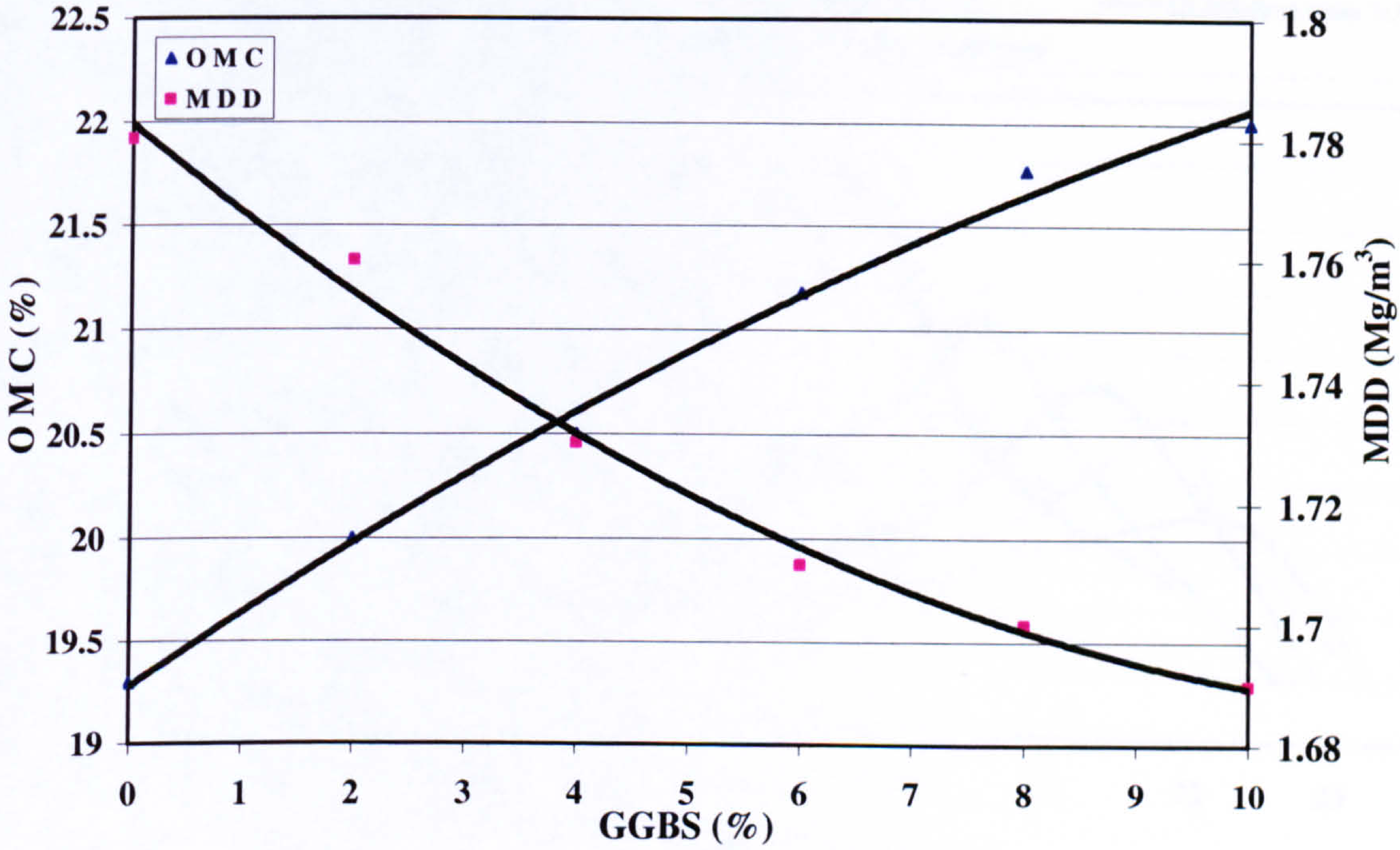

Figure 7.1 Effect of GGBS only on the maximum dry density and optimum moisture content of the test soil

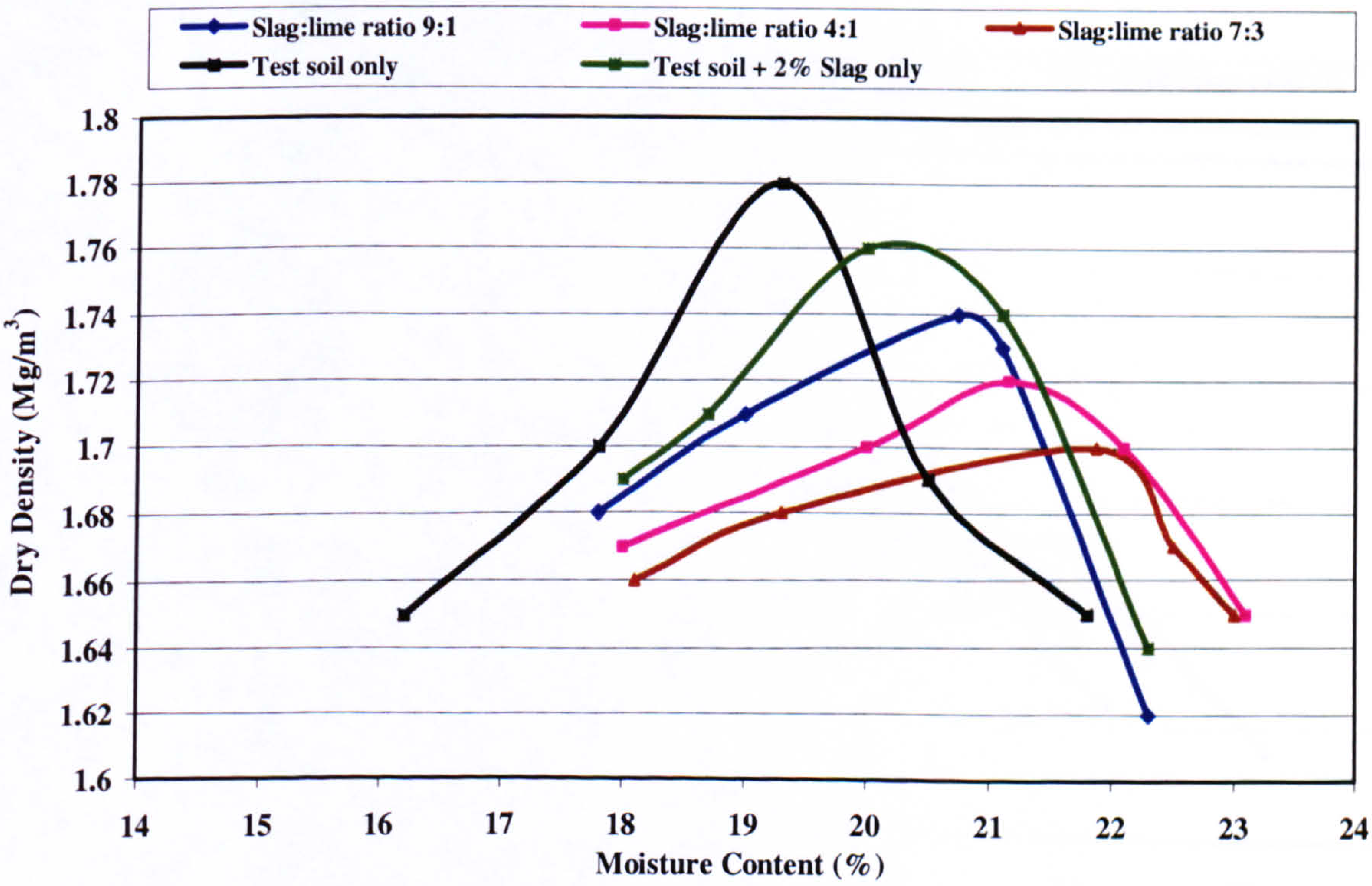

Figure 7.2 Compaction curves for test soil only, test soil + 2% GGBS and test soil + 2% binder (GGBS+ lime)

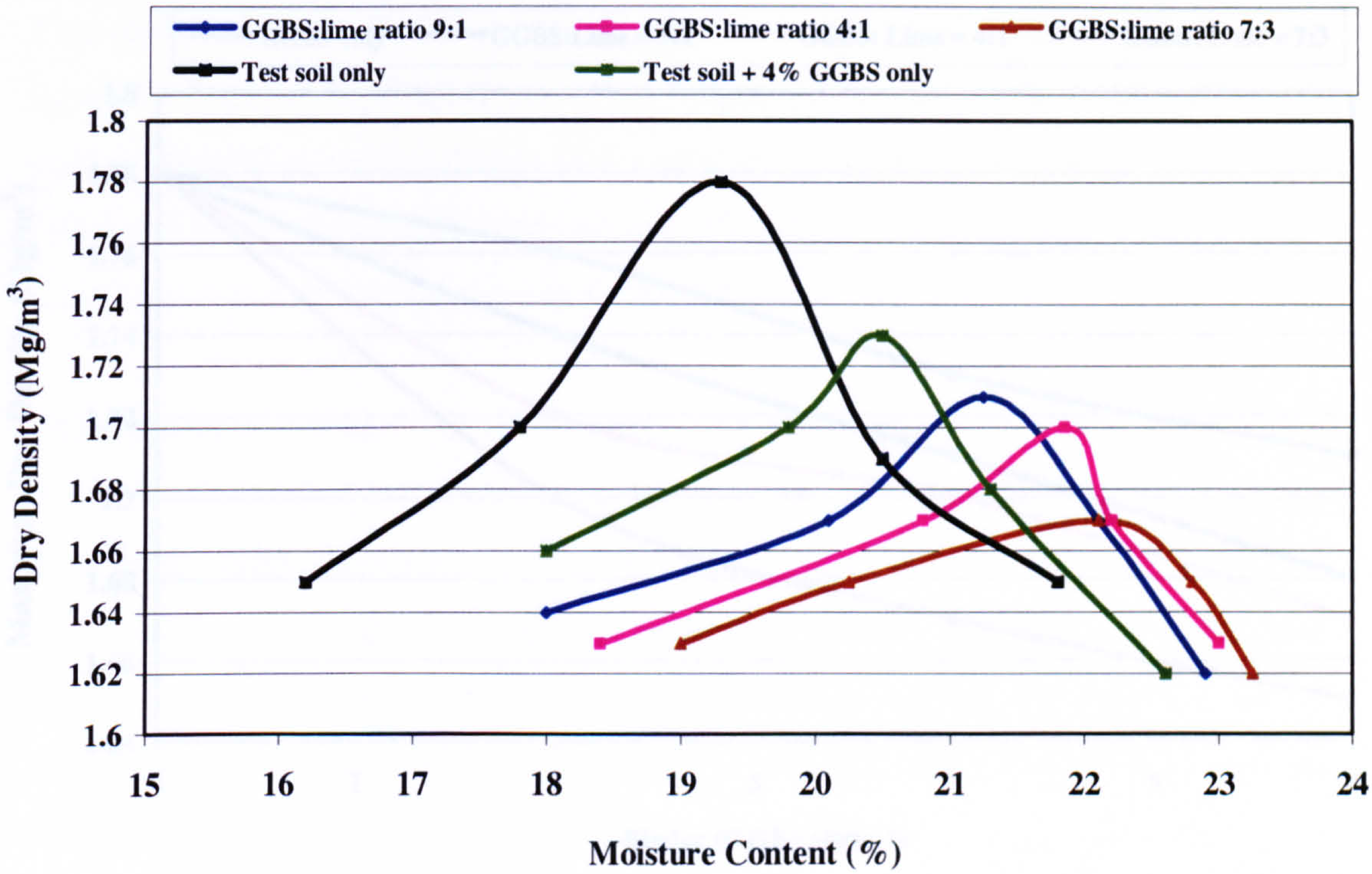

Figure 7.3 Compaction curves for test soil only, test soil + 4 % GGBS and test soil + 4 % binder (GGBS+ lime)

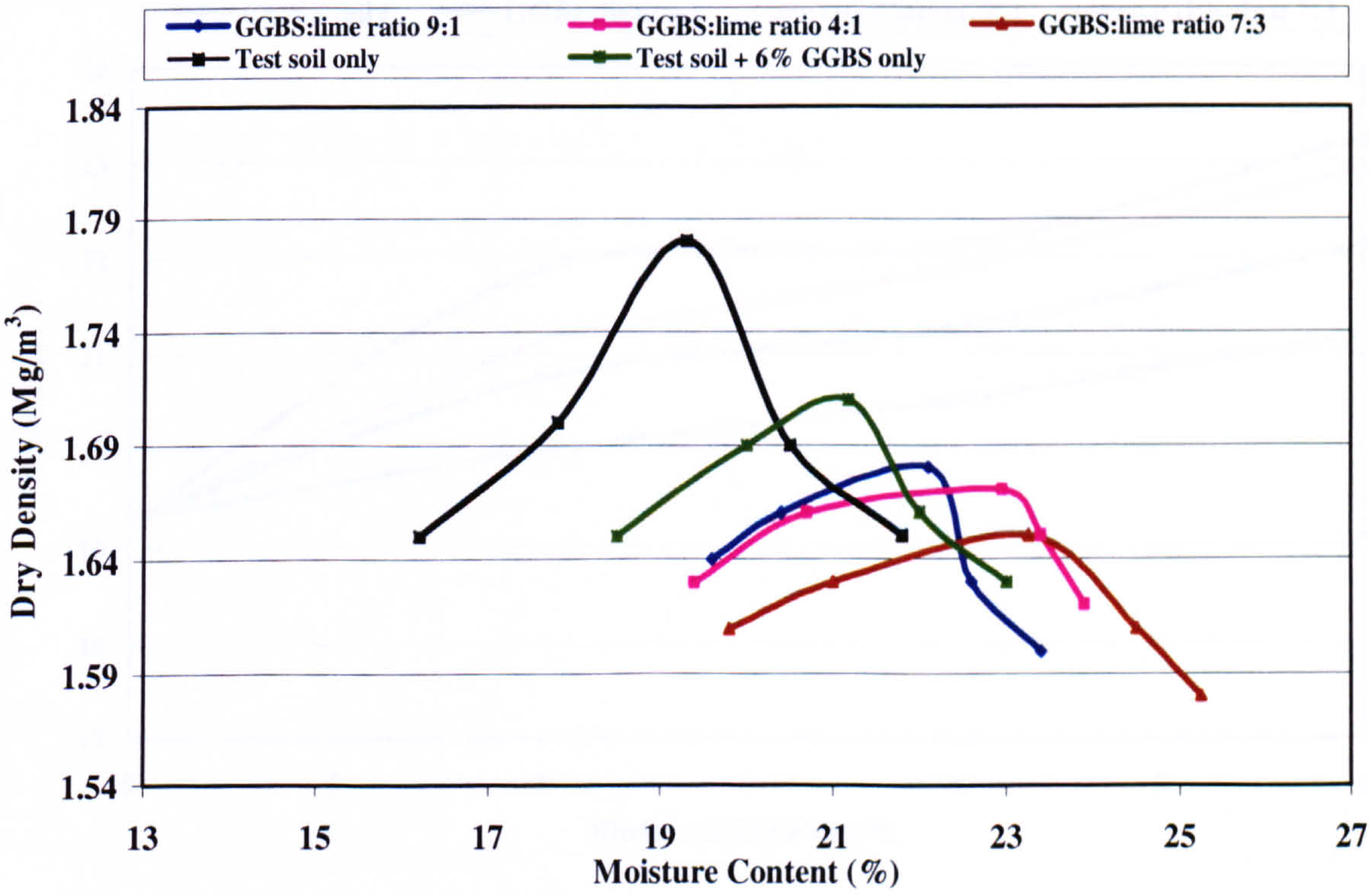

Figure 7.4 Compaction curves for test soil only, test soil + 6 % GGBS and test soil + 6 % binder (GGBS+ lime)

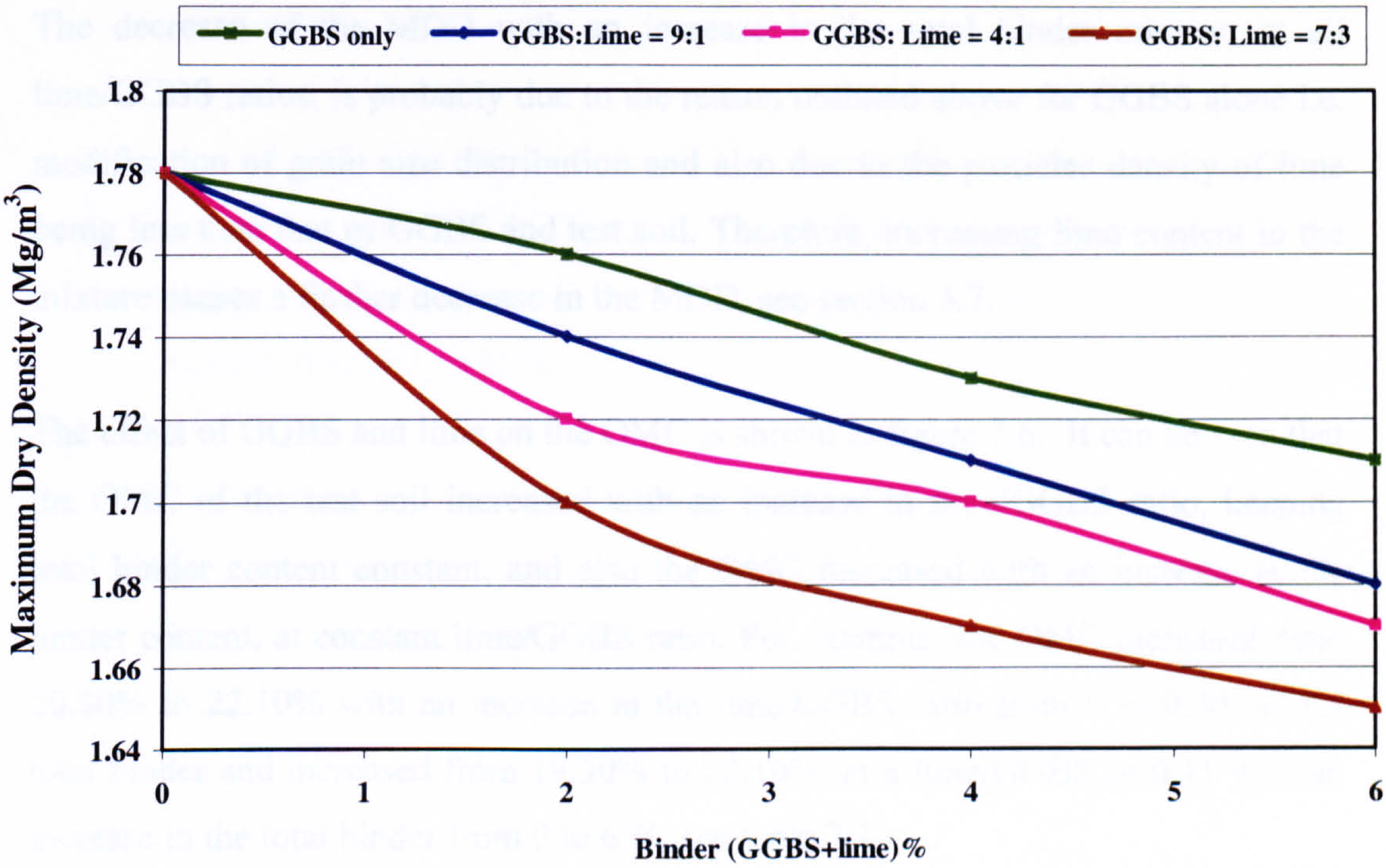

Figure 7.5 Effect of the total binder (GGBS and lime) (%) on the maximum dry density of the test soil

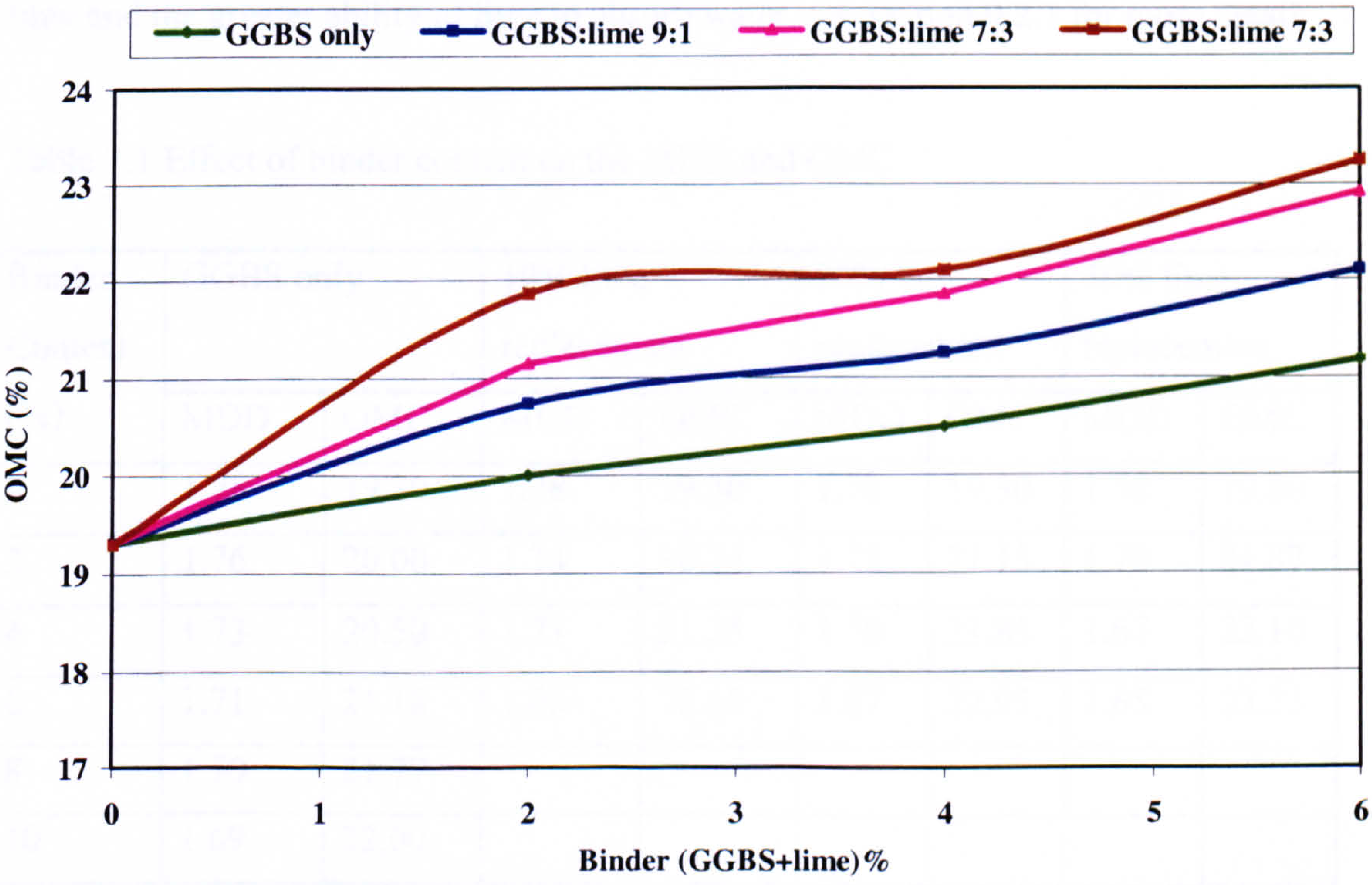

Figure 7.6 Effect of the total binder (GGBS and lime) (%) on the optimum moisture content of the test soil

The decrease of the MDD with an increase in the total binder content, at all lime/GGBS ratios, is probably due to the reason outlined above for GGBS alone i.e. modification of grain size distribution and also due to the particles density of lime being less than that of GGBS and test soil. Therefore, increasing lime content in the mixture causes a further decrease in the MDD, see section 5.7.

The effect of GGBS and lime on the OMC is shown in figure 7.6. It can be seen that the OMC of the test soil increased with an increase in lime/GGBS ratio, keeping total binder content constant, and also the OMC increased with an increase in the binder content, at constant lime/GGBS ratio. For example, the OMC increased from 20.50% to 22.10% with an increase in the lime/GGBS ratio from 0 to 0.30, at 4% total binder and increased from 19.30% to 22.10%, at a lime/GGBS of 0.11 with an increase in the total binder from 0 to 6 %, see table 7.2.

The increase in the OMC with an increase in the lime/GGBS ratio at constant binder content is probably due to the water needed for the hydration of GGBS activated by lime and the greater ability of lime to absorb water, see section 9.2.1 for more detail.

Table 7.1 Effect of binder content on the MDD and OMC

Binder Content (%)	GGBS only		10% lime replacement		20% lime replacement		30% lime replacement	
	MDD	OMC	MDD	OMC	MDD	OMC	MDD	OMC
0	1.78	19.30	1.78	19.30	1.78	19.30	1.78	19.30
2	1.76	20.00	1.74	20.74	1.72	21.15	1.70	21.87
4	1.73	20.50	1.71	21.25	1.70	21.85	1.67	22.10
6	1.71	21.18	1.68	22.10	1.67	22.95	1.65	23.25
8	1.70	21.77						
10	1.69	22.00						

7.2.4 SUMMARY

1. The addition of GGBS alone to the test soil slightly decreased the maximum dry density from 1.78 Mg/m^3 to 1.69 Mg/m^3, while it increased the optimum moisture content from 19.3 % to 22 %, with an increase in the GGBS content from 0% to 10%.

2. The addition of GGBS and lime to the test soil further increased the optimum moisture content with an increase in the total binder at a constant lime/GGBS ratio. The OMC also increased with an increase in the lime/GGBS ratio at a constant total binder percentage. The addition of the GGBS and lime further decreased the MDD of the test soil.

3. The rate of increase in the OMC increased with an increase in the total binder percentage at a constant lime/GGBS ratio. Also, the rate of decrease in the MDD increased with an increase in the total binder percentage.

7.3 PLASTICITY CHARACTERISTICS OF GGBS-SOIL AND GGBS-LIME-SOIL MIXTURES

7.3.1 INTRODUCTION

This section presents data and associated discussion concerning the effects of GGBS with and without lime and lime only on the plasticity properties of the clay test soil. The plasticity characteristics of soils are often expressed in terms of the liquid limit (LL), plastic limit (PL) and plasticity index (PI) as first proposed by Atterberg (1911) and as described in the British Standard (B S 1377: 2:1990).

These limits are expressed as the percentage of moisture by dry weight of soil, commonly referred to as the moisture content. Atterberg limits are used as index properties for the classification of soils. Certain behavioural characteristics have been associated with soils having Atterberg limits that fall within certain ranges.

However, only general indications should be drawn from these limits, since consistency limits of clays are affected by their composition and chemical environment (Abdelkader, 1985). Also, the limits may be directly related to engineering properties which may be more affected by stress history.

7.3.2 EFFECT OF GGBS ON THE PLASTICITY CHARACTERISTICS OF THE TEST SOIL

To determine the effects of GGBS with and without lime, and the effects of lime only on the plasticity characteristics of the pure clay test soil, Atterberg limit tests were first carried out on the test soil, after 1 and 3 days after mixing, see figures 7.7 and 7.8 and table 7.2. It can be seen, after 1 day after mixing, that the liquid limit decreased insignificantly from 55% to 53%, while the plastic limit increased slightly from 30% to 32% with an increase in the GGBS content up to 10% by dry weight of soil, which results in a significant decrease in the plasticity index from 25% to 21%. No significant changes were found comparing the 3 days with 1 day data. The results showed small changes as the test soil was not pure clay and contained some inert materials (silt and non clay minerals), see section 5.6. Therefore, the plasticity characteristic tests were carried out on samples of pure clay test soil with and without GGBS and/or lime. The first stage was to study the effect of GGBS only on the pure clay test soil, the percentage of GGBS varying from 0 to 10% by dry weight. The results of this study are illustrated in figure 7.10 and table 7.3.

Table 7.2 Effect of GGBS alone on the Atterberg limits of the test soil

GGBS (%)		1 day		3 days		
	LL	PL	PI	LL	PL	PI
0	55	30	25	55	30	25
2	55	32	23	56	33	23
4	54	32	22	55	35	20
6	53	31	22	55	35	20
8	54	32	22	54	36	18
10	53	32	21	53	34	19

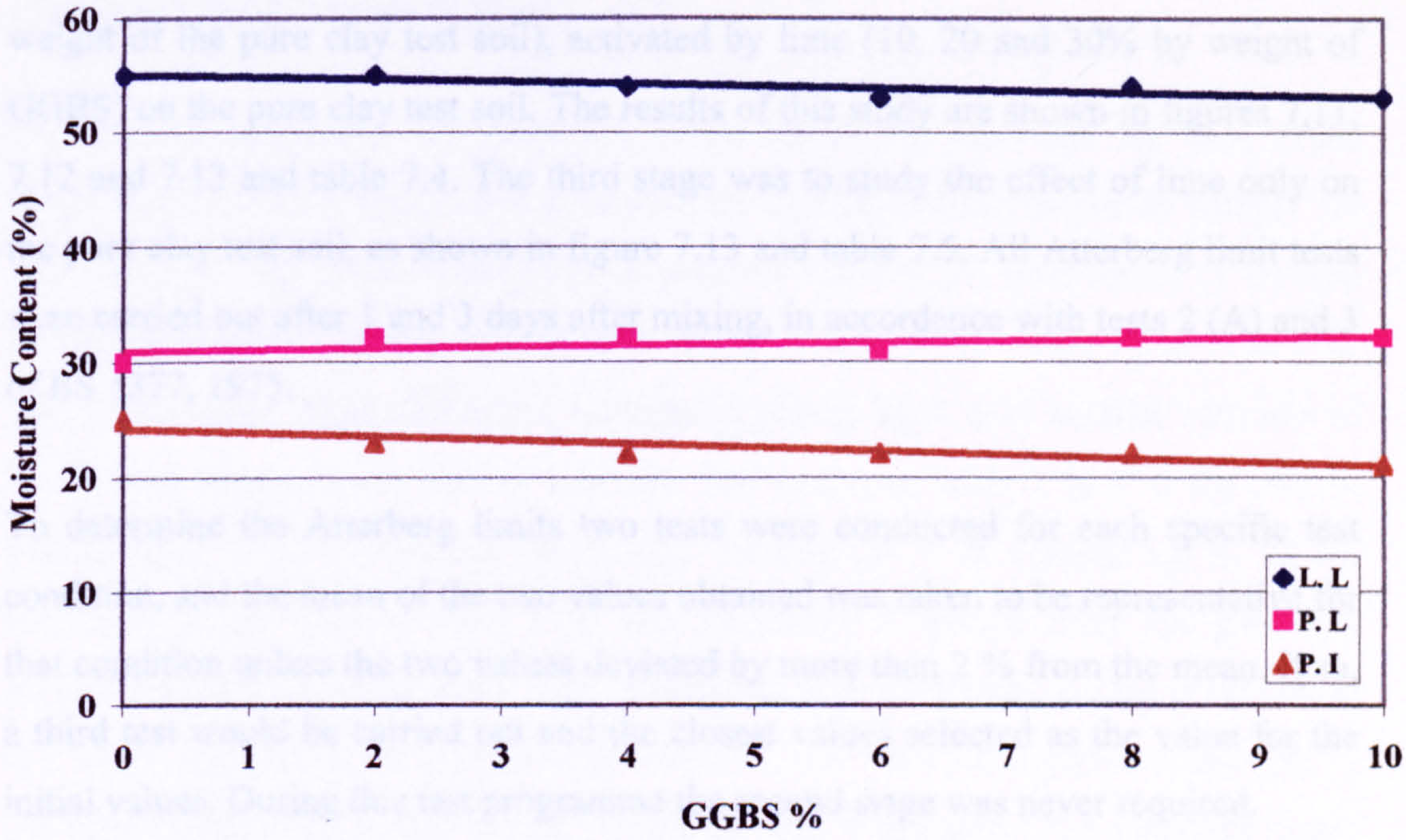

Figure 7.7 Atterberg limits for the test soil and GGBS after 1 days

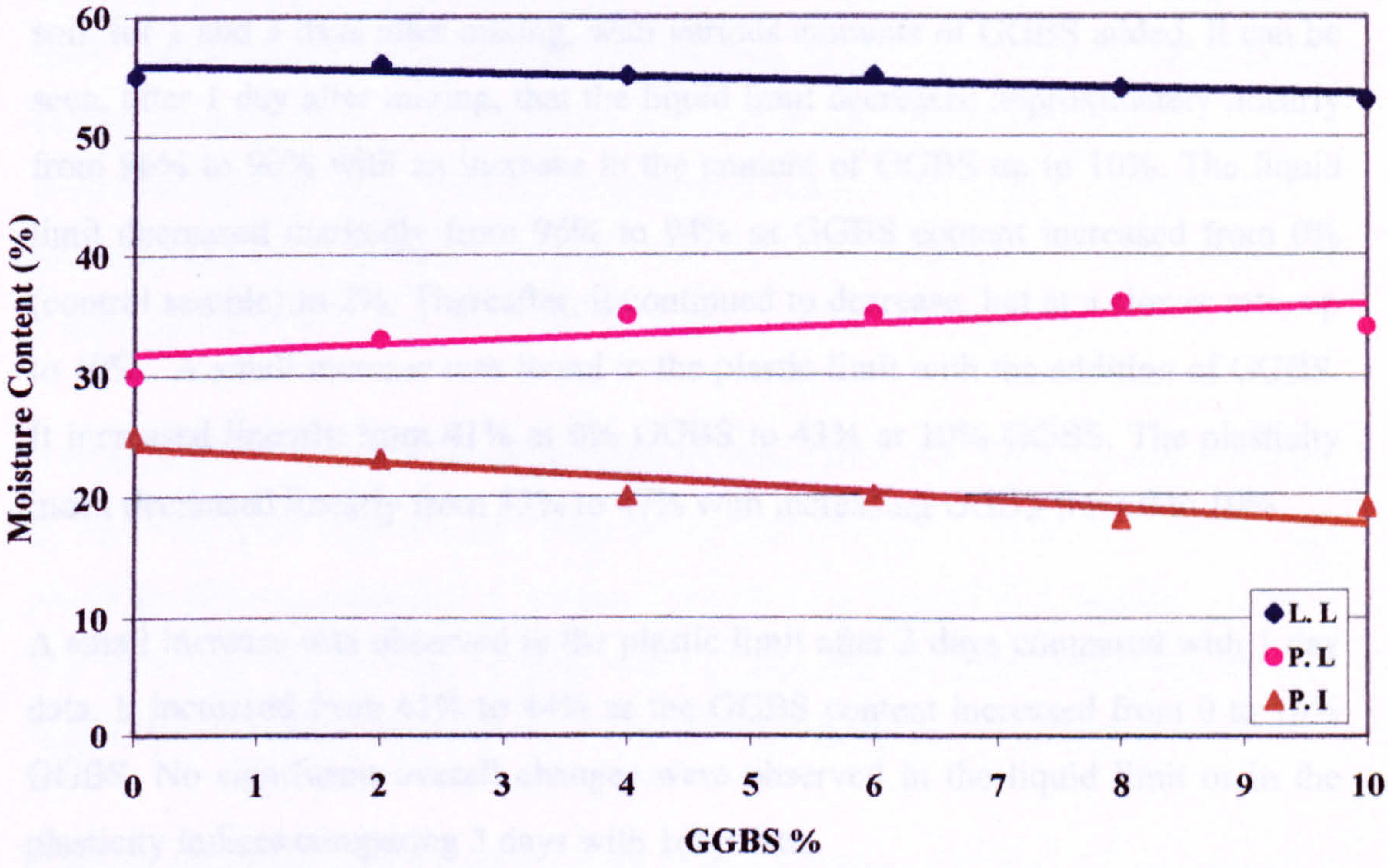

Figure 7.8 Atterberg limits for the test soil and GGBS after 3 days

The second stage was to study the effect of GGBS (2, 4, 6, 8, and 10 %, by dry weight of the pure clay test soil), activated by lime (10, 20 and 30% by weight of GGBS) on the pure clay test soil. The results of this study are shown in figures 7.11, 7.12 and 7.13 and table 7.4. The third stage was to study the effect of lime only on the pure clay test soil, as shown in figure 7.13 and table 7.5. All Atterberg limit tests were carried out after 1 and 3 days after mixing, in accordance with tests 2 (A) and 3 of BS 1377, 1975.

To determine the Atterberg limits two tests were conducted for each specific test condition, and the mean of the two values obtained was taken to be representative for that condition unless the two values deviated by more than 2 % from the mean. If so, a third test would be carried out and the closest values selected as the value for the initial values. During this test programme the second stage was never required.

The liquid and plastic limits and the plasticity index of the pure clay test soil (control sample) are 96% and 41% and 55% respectively. Figure 7.10 and table 7.3 illustrate the changes in the liquid and plastic limits and the plasticity index of pure clay test soil, for 1 and 3 days after mixing, with various amounts of GGBS added. It can be seen, after 1 day after mixing, that the liquid limit decreased approximately linearly from 96% to 90% with an increase in the amount of GGBS up to 10%. The liquid limit decreased markedly from 96% to 94% as GGBS content increased from 0% (control sample) to 2%. Thereafter, it continued to decrease, but at a slower rate, up to 10%. A small increase was found in the plastic limit with the addition of GGBS. It increased linearly from 41% at 0% GGBS to 43% at 10% GGBS. The plasticity index decreased linearly from 55% to 47% with increasing GGBS from 0 to 10%.

A small increase was observed in the plastic limit after 3 days compared with 1 day data. It increased from 41% to 44% as the GGBS content increased from 0 to 10% GGBS. No significant overall changes were observed in the liquid limit or in the plasticity indices comparing 3 days with 1day data.

7.3.3 EFFECT OF GGBS AND LIME ON THE PLASTICITY CHARACTERISTICS OF SOIL

Figure 7.11 and table 7.4 show the effect of GGBS activated by lime (10% replacement), after 1 and 3 days after mixing. The liquid limit increased sharply for 1 day after mixing , from 96% to a peak value of 110% at 2% total binder (T B) then decreased relatively slowly to 87% with further additions of binder up to 10% T B. The results show a marked initial increase in plastic limit with smaller amounts of binder and then a lower rate of increase. For example, with 4 % T B the rise in plastic limit is 13% whereas the further increase of T B content up to 10 %, increased plastic limit by only 2 %. This behaviour in plastic limit was generally similar for the immediate and 3 days curing, but with increasing curing period there is a slight increase in the plastic limit values. There were no significant changes in the liquid limit after 3 days curing compared with 1 day data. The plasticity index after 3 days curing is virtually the same as after 1 day curing.

Figures 7.12 and 7.13 and table 7.4 show the effect of GGBS activated by 20% and 30% replacement by lime respectively for 1 and 3 days after mixing. These figures show similar trends to 7.11 except that the peak of the liquid limit which generally occurs around 2 % of T B, decreases with increasing lime/GGBS ratio for 1 and 3 days after mixing. Also, there are increases in plastic limit values and decreases in plasticity index values with increasing lime/GGBS ratio. Increased curing from 1 to 3 days results in increases in PL and decreases in PI which are small but clear.

It was observed that only 2% of GGBS with lime (10%, 20% or 30%) was sufficient to move the pure clay test soil from being classified as an inorganic clay of high plasticity to being classified as an inorganic silt (see figure 7.9, Casagrande, 1947). The pure clay test soil has become more friable and the particle size has become bigger which indicate significant change in the plasticity characteristics

It can be observed from the plastic limit changes, figures 7.11 to 7.13, that there is a change of shape of the plastic limit data around 6 % of additives up to which there is a drastic increase in plastic limit and beyond which the rise is very slow or remains constant. Many investigators have observed that when lime is added to clay soil, the plastic limit increases with increasing lime content up to a certain point, thereafter further increase in the lime content produces small or no change in the plastic limit, see section 2.9.2. The increase in plastic limit can be explained as the initial addition of GGBS and lime results in either a cation exchange or a crowding of additional cations onto the clay. Clay particles then become electrically attracted to one another causing flocculation, or aggregation. The clay particles then act as aggregates and behave as silt that has a low plasticity. When clay particles get saturated with calcium ions, no further cation exchange occurs on the addition of extra lime and the plastic limit becomes constant and the remaining calcium ions are left for the pozzolanic reaction.

Comparing the effect of GGBS on the test soil and the pure clay test soil data, it can be observed that approximately the same trends were obtained, a decrease in the liquid limit, an increase in the plastic limit and a decrease in the plasticity index. The plasticity characteristics study was extended to cover the effects of lime on the Atterberg limits of the pure clay test soil.

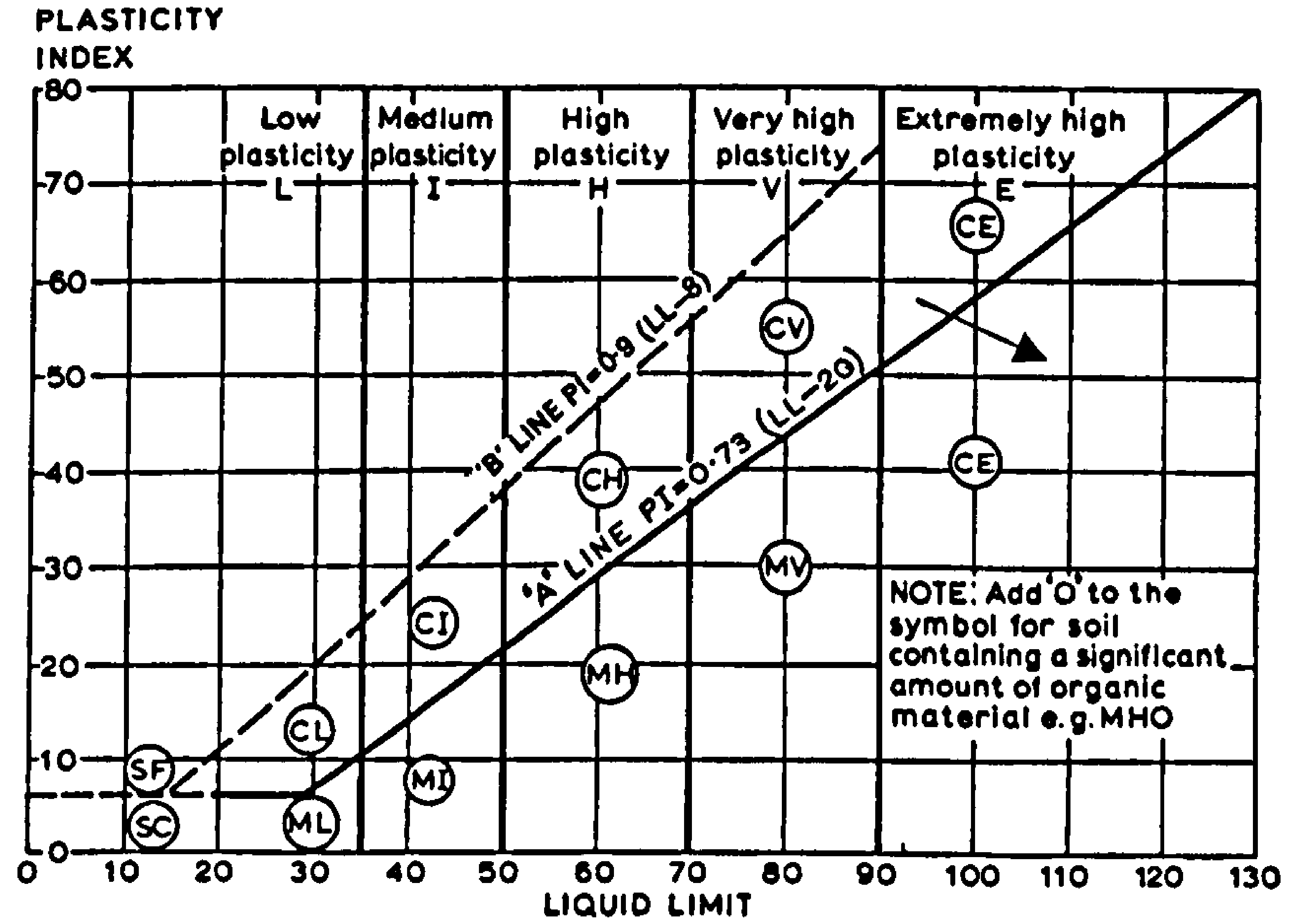

Figure 7.9 Effect of GGBS alone on the change in plasticity of the pure clay test soil in Casagrande chart

7.3.4 EFFECT OF LIME ONLY ON THE PLASTICITY CHARACTERISTICS OF SOIL

Figure 7.14 and table 7.5 show the effect of lime addition only on the plasticity characteristics of the pure clay test soil for 1 and 3 days after mixing. The liquid limit increased gradually after 1 day after mixing, from 96% to a peak value of 100% at 4% lime, and then decreased gradually to 94% at 10% lime. Similar results were obtained from the work carried out on two samples of soil obtained from Damanhour, and Elfayoum, in Egypt (Abdelkader and Hamdani, 1985) see section 2.9.2. Dumbleton (1962) in his study on London clay, observed an increase in liquid limit at low lime content and a gradual decrease in liquid limit with further increase in lime content. Also, Daniels (1971) studied the effect of lime on pure montmorillonite and kaolinite clay minerals. He found an increase in liquid limit to a peak value at low lime content, and then gradual decrease with a further increase in lime content in montmorillonite and kaolinite.

The plastic limit increased sharply from 41% to 61% at 4% lime. Thereafter, it remained more or less constant beyond this lime content up to 10% lime. The initial increase in plastic limit could be the result of flocculation of the clay particles on the addition of lime. According to Abdelkader and Hamdani (1985) the first reaction of soil stabilisation with lime is adsorption of calcium cations to the clay surface. The increased adsorption of the calcium cations to the clay surface is assumed to cause better bonding between particles leading to flocculation into larger lumps of soil, higher viscosity and hence high plastic limit. The flocculated nature of the soil-lime mixture is considered to be responsible for the reduction of the liquid limit by creating relatively weak bonding forces between the various "flocs" and therefore small amounts of water are required to lubricate them to the stage where the required movement would occur with the liquid limit shearing action.

Table 7.3 Atterberg limits for test soil and GGBS

GGBS (%)	1 Day			3 Days		
	LL (%)	PL (%)	PI (%)	LL (%)	PL (%)	PI (%)
0	96	41	55	96	41	55
2	94	41	53	93	42	51
4	93	42	51	93	43	50
6	91	42	49	92	44	48
8	91	42	49	92	44	48
10	90	43	47	91	44	47

Table 7.4 Atterberg limits for test soil, GGBS and lime

Binder % * (Lime) %	1 Day			3 Days		
	LL (%)	PL (%)	PI (%)	LL (%)	PL (%)	PI (%)
0 (10)	96	41	55	96	41	55
2 (10)	110	52	58	111	42	51
4 (10)	109	53	56	109	43	50
6 (10)	96	55	41	96	44	48
8 (10)	90	55	35	90	44	48
10 (10)	87	55	32	88	44	47
0 (20)	96	41	55	96	41	55
2 (20)	105	54	51	105	54	49
4 (20)	96	55	41	96	55	36
6 (20)	94	56	38	95	56	36
8 (20)	93	58	35	94	58	34
10 (20)	89	58	31	91	58	29
0 (30)	96	41	55	96	41	55
2 (30)	98	54	44	96	56	40
4 (30)	91	56	35	92	59	33
6 (30)	89	57	32	89	60	29
8 (30)	88	57	31	86	61	25
10 (30)	88	57	31	85	61	24

* Binder expressed as % total binder by dry soil weight, lime expressed as % of binder

Table 7.5 Atterberg limits for test soil and lime

Lime (%)	1 Day			3 Days		
	LL (%)	PL (%)	PI (%)	LL (%)	PL (%)	PI (%)
0	96	41	55	96	41	55
2	99	53	46	96	55	41
4	100	61	39	93	64	29
6	98	61	37	92	64	28
8	95	62	33	90	64	26
10	94	62	32	87	64	23

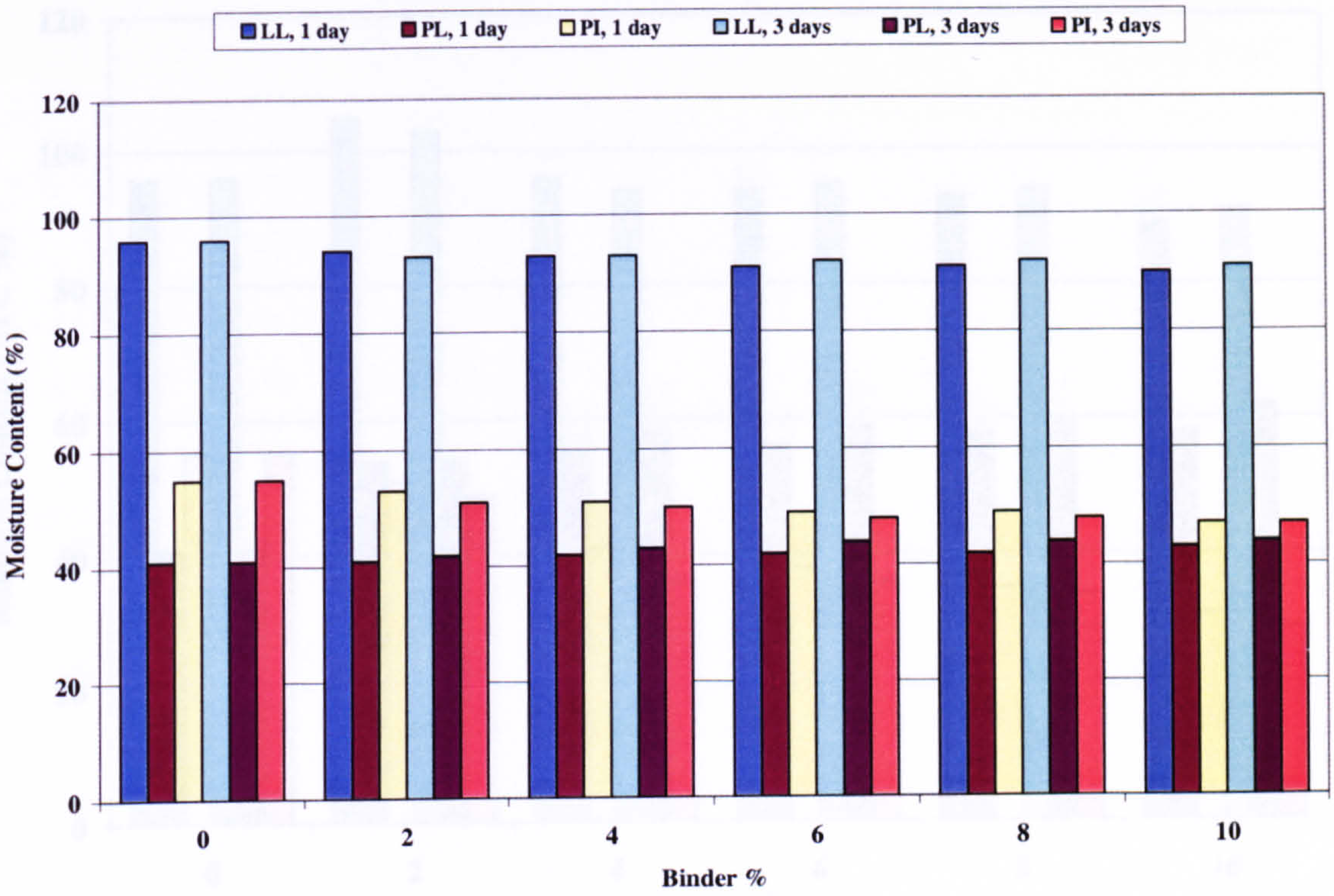

Figure 7.10 Atterberg limits for pure clay test soil and GGBS only after 1 and 3 days

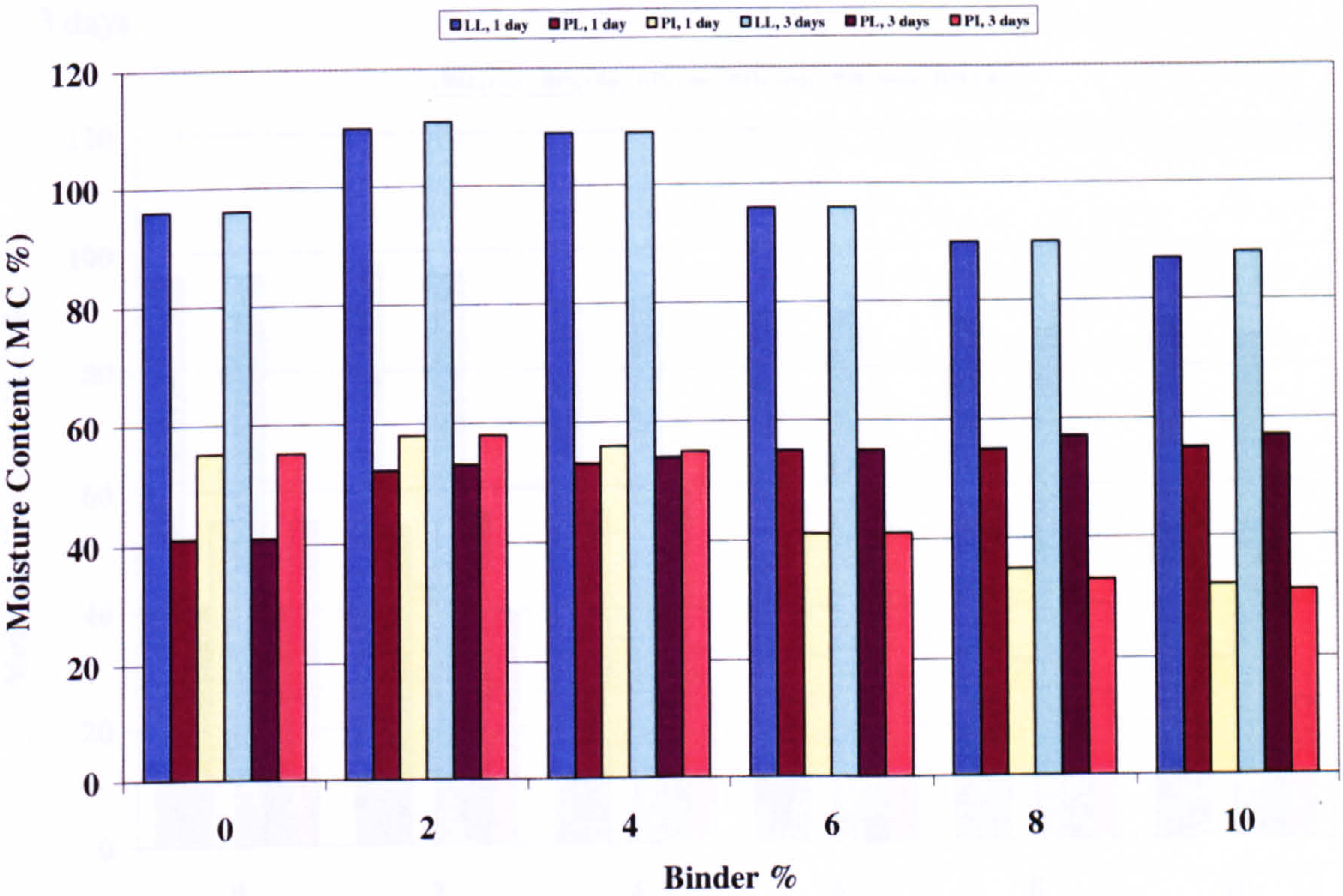

Figure 7.11 Atterberg limits for pure clay test soil and binder (10 % lime) after 1 and 3 days

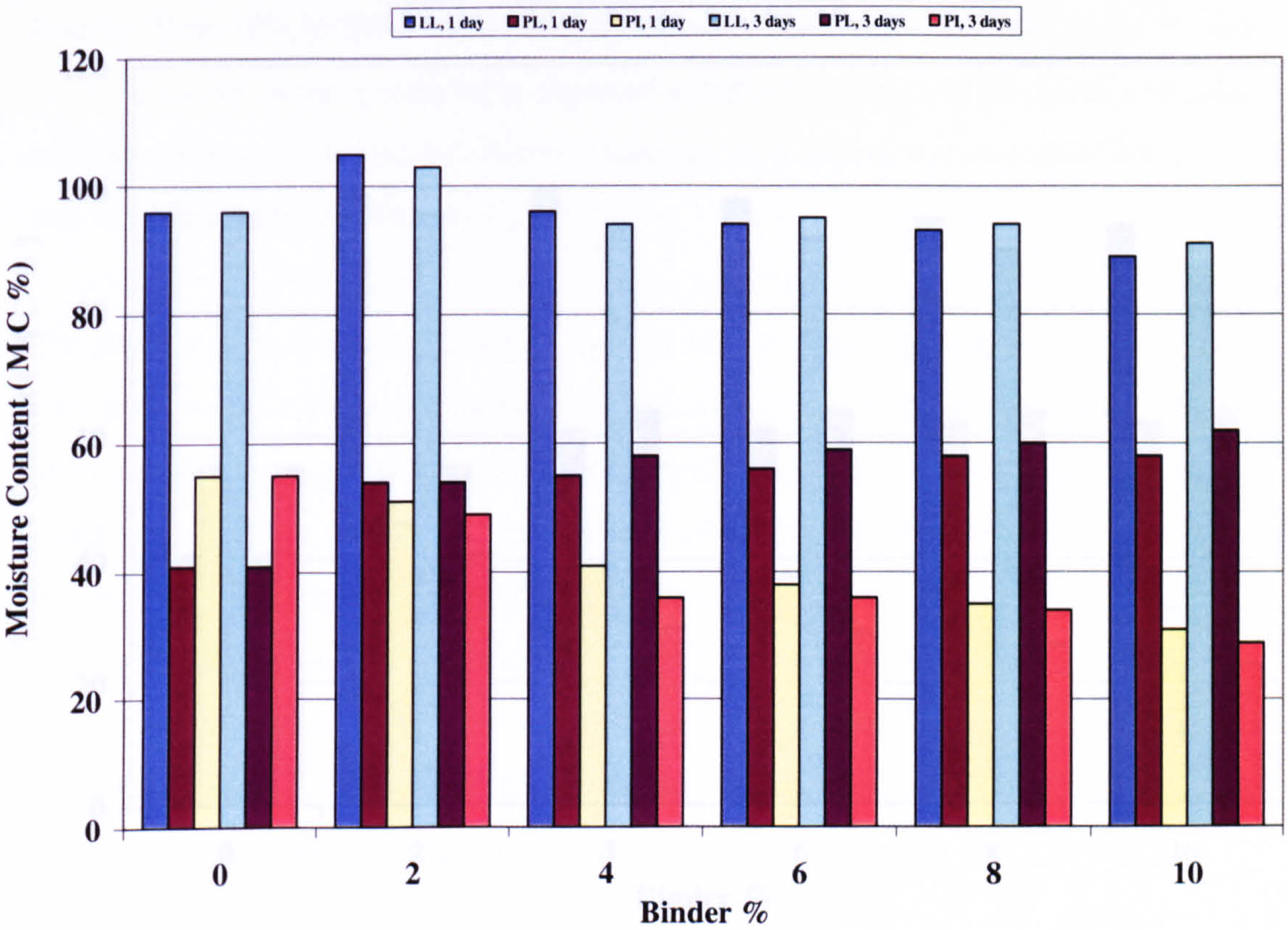

Figure 7.12 Atterberg limits for pure clay test soil and binder (20 % lime) after 1 and 3 days

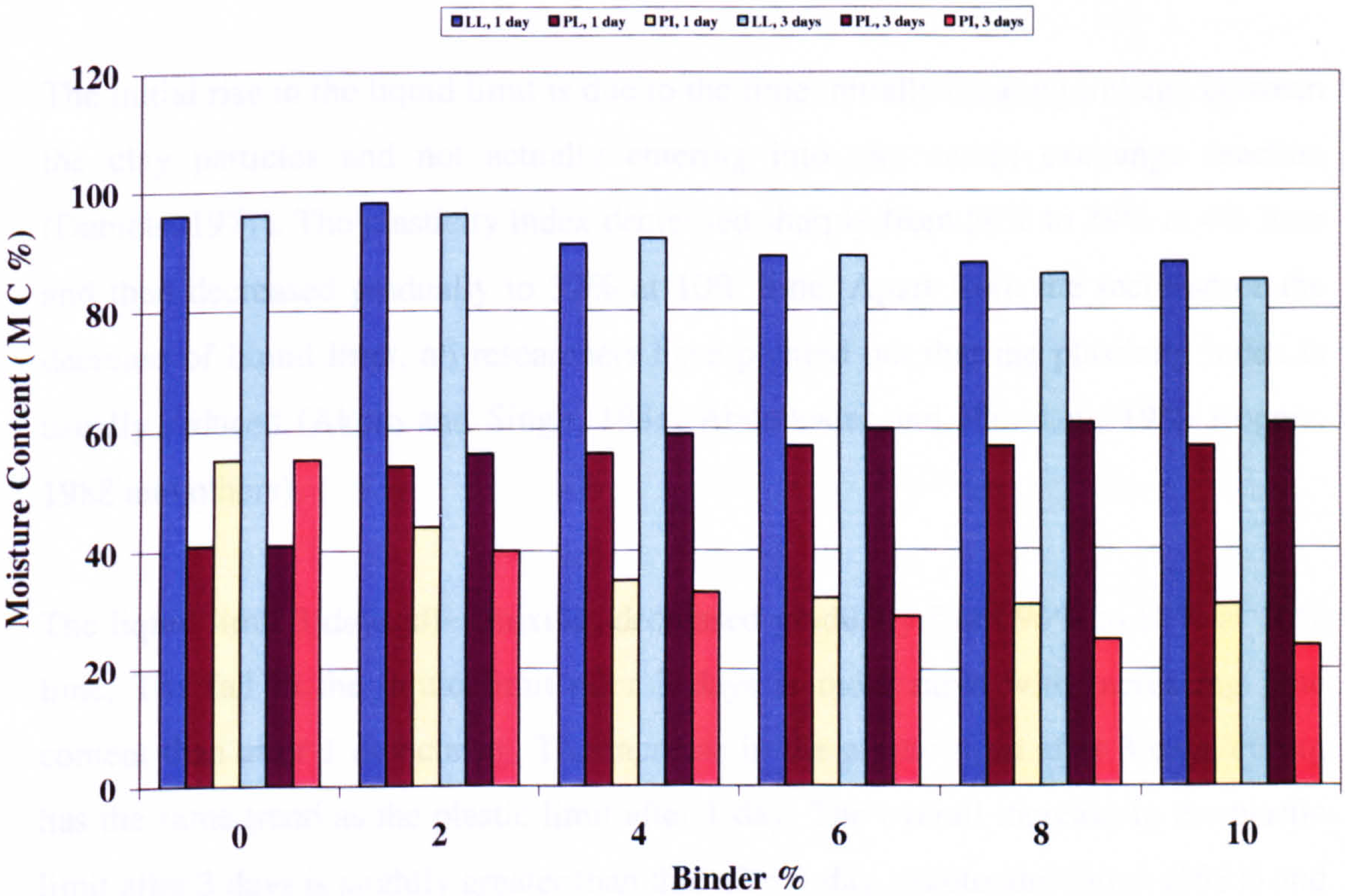

Figure 7.13 Atterberg limits for pure clay test soil and binder (30% lime) after 1 and 3 days

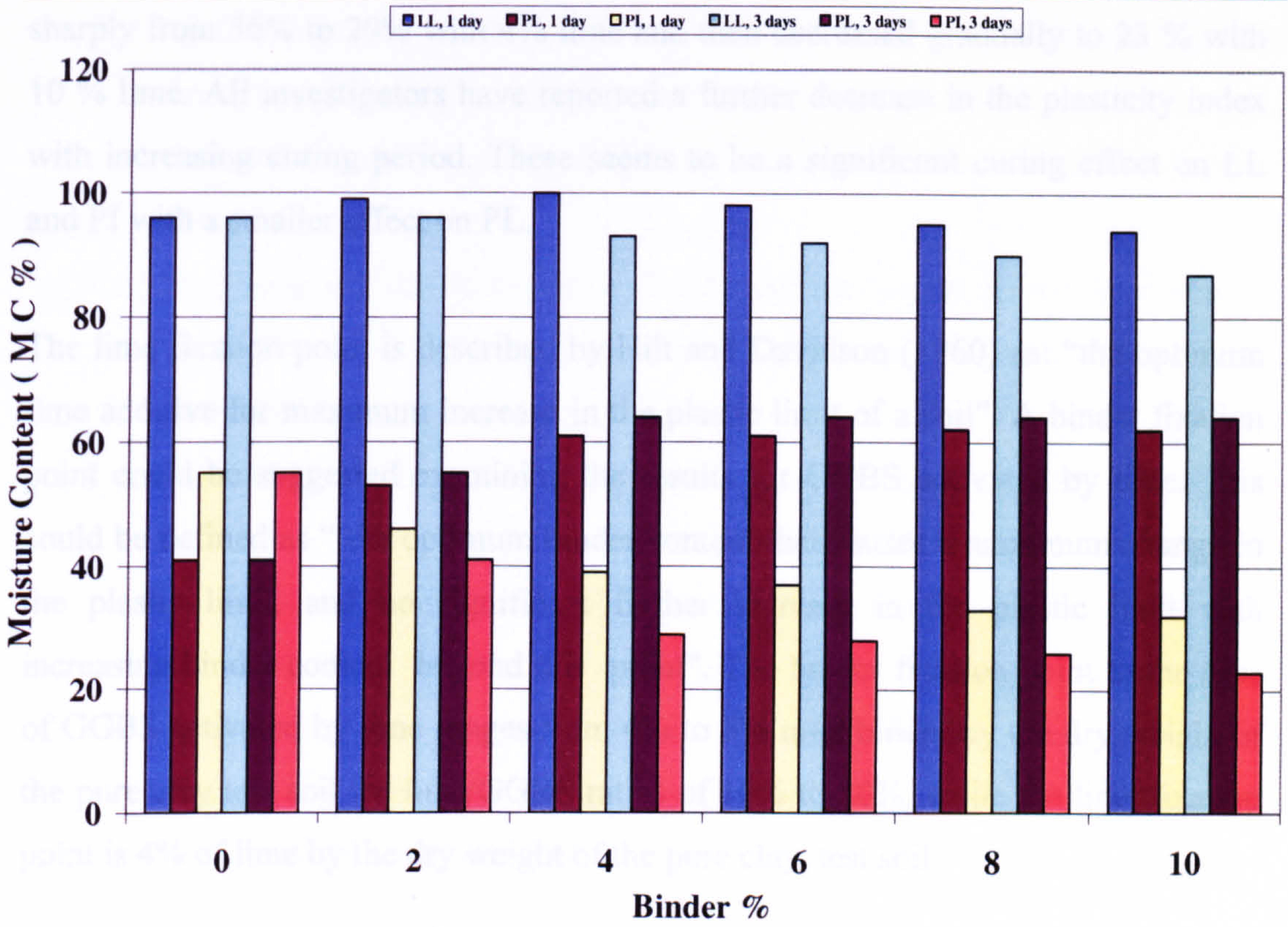

Figure 7.14 Atterberg limits for pure clay test soil and lime only after 1 and 3 days

The initial rise in the liquid limit is due to the lime initially creating linkage between the clay particles and not actually entering into any cation exchange reaction (Daniels, 1971). The plasticity index decreased sharply from 56% to 39% at 4% lime and then decreased gradually to 32% at 10% lime. Apart from the increase or the decrease of liquid limit, all researchers have pointed out that the plasticity index is usually reduced (Akoto and Singh, 1981; Abdelkader and Hamdani, 1985 Rogers; 1988 and others).

The liquid limit 3 days after mixing, decreased gradually from 96% to 87% at 10% lime. The fall in the liquid limit after 3 days is more rapid with increasing lime content than after 1 day curing. The increase in the plastic limit after 3 days curing has the same trend as the plastic limit after 1 day. The overall increase in the plastic limit after 3 days is slightly greater than that after 1 day. Akoto and Singh (1981) and Abdelkader (1981) reported a further decrease in the liquid limit and a further increase in the plastic limit with time. The plasticity index after 3 days decreased sharply from 56% to 29% with 4% lime and then decreased gradually to 23 % with 10 % lime. All investigators have reported a further decrease in the plasticity index with increasing curing period. These seems to be a significant curing effect on LL and PI with a smaller effect on PL.

The lime fixation point is described by Hilt and Davidson (1960) as: "the optimum lime additive for maximum increase in the plastic limit of a soil". A binder fixation point could be suggested examining the results for GGBS activated by lime. This could be defined as " the optimum binder content that causes a maximum change in the plastic limit, and no significant further increase in the plastic limit with increasing binder content beyond this point". The binder fixation point in the case of GGBS activated by lime ranges from 4% to 6% total binder by the dry weight of the pure clay test soil for lime/GGBS ratios of 10% to 30%, while the lime fixation point is 4% of lime by the dry weight of the pure clay test soil.

7.3.5 SUMMARY

1. The addition of GGBS from 0% to 10% to the test soil slightly decreased the liquid limit, slightly increased the plastic limit and slightly decreased the plasticity index. No significant overall change was observed in the Atterberg limit comparing 3 day data with 1 day data.
2. The addition of GGBS from 0% to 10% to the pure clay test soil decreased the liquid limit from 96% to 90%, increased slightly the plastic limit and decreased significantly the plasticity index from 55% to 47% GGBS. Small further increases in the plastic limit values while no significant overall changes in the liquid lime and the plasticity indices were observed comparing 3 days with 1 day data.
3. The addition of GGBS and lime (10, 20, or 30%) of binder to the pure clay test soil increased the liquid limit sharply from 96% to 110%, thereafter it decreased gradually to 87% with further increases in the binder contents. The plastic limit increased with an increase in binder content up to 6%. Thereafter no further increase was observed in the plastic limit with a further increase in binder content. The plasticity index decreased

dramatically with an increase in the binder content up to 6% and then it decreased gradually with increasing binder content. No significant overall changes were observed in Atterberg limits comparing 3 day with 1 day data.

4. Increasing the curing period to 3 days caused further increases in plastic limit values and further decreases in the plasticity index, with increases in the binder content.
5. The addition of lime to the pure clay test soil gradually increased the liquid limit from 96% to a peak value of 100% with 4% lime, and then it decreased gradually to 94% with 10% lime. The plastic limit increased with increasing lime content up to 61% with 4% lime and thereafter no significant further increase was observed with further increases in lime content. The plasticity index decreased from 55% to 39% with increasing lime content from 0% to 4%.

7.4 EFFECT OF GGBS AND LIME ON THE pH OF THE PURE CLAY TEST SOIL

The pH of the pure clay test soil with different percentages of GGBS and lime were measured to determine the minimum binder content required for the stabilisation of the test soil which is enough to cause the silica and aluminium from clay to dissolve.

The addition of small amounts of GGBS alone (of the order of 4 %) to the test soil significantly increased the pH of the test soil from 7.93 (the pH of the pure clay test soil) to 9.36, see figure 7.15. Further addition of GGBS resulted in a relatively small increase in the pH up to 9.39 at 9% GGBS. The addition of only 2% GGBS with 20% lime of the total binder to the test soil sharply increased the pH to 12.24, a further increase in the binder content to 4% resulted in an increase in the pH to 12.39, approximately equal to the pH of the pure lime solution. Further increases in the binder content to 27% increased the pH of the test soil very slightly to 12.41. The effect of GGBS with 30% lime on the test soil is very similar to that when the lime content is 20%. The addition of 6% lime alone to the test soil increased the pH sharply to 12.34, approximately the pH of a pure lime solution. Further increases in the lime content resulted in a very small increase in the pH.

It can be seen that the addition of 6% GGBS activated by 20% or 30% lime or 6% lime only, to the test soil increased the pH to approximately the same level. As the pH of the test soil, GGBS and lime and the pH of the test soil and lime only both at 6% binder, were approximately equal to that of pure lime solution, 6% of binder would be the initial consumption of binder required for full modification of the test soil used. This initial consumption of binder was also confirmed by plastic limit tests where the increase in the plastic limit produced a binder fixation point between 4% and 6% of binder depending on the lime/GGBS ratio, figures 7.11 to 7.13.

It was also noted that most of the changes in the pH (≈ 98%) had occurred at 4% binder content (GGBS and lime or lime only). Thus 6% binder was selected as the minimum amount of lime or binder required for modification and minimal stabilisation of this particular type of soil. Therefore, all investigations of the soil-GGBS-lime system including the UCS and free swelling characteristics are carried out at a minimum binder content of 6%.

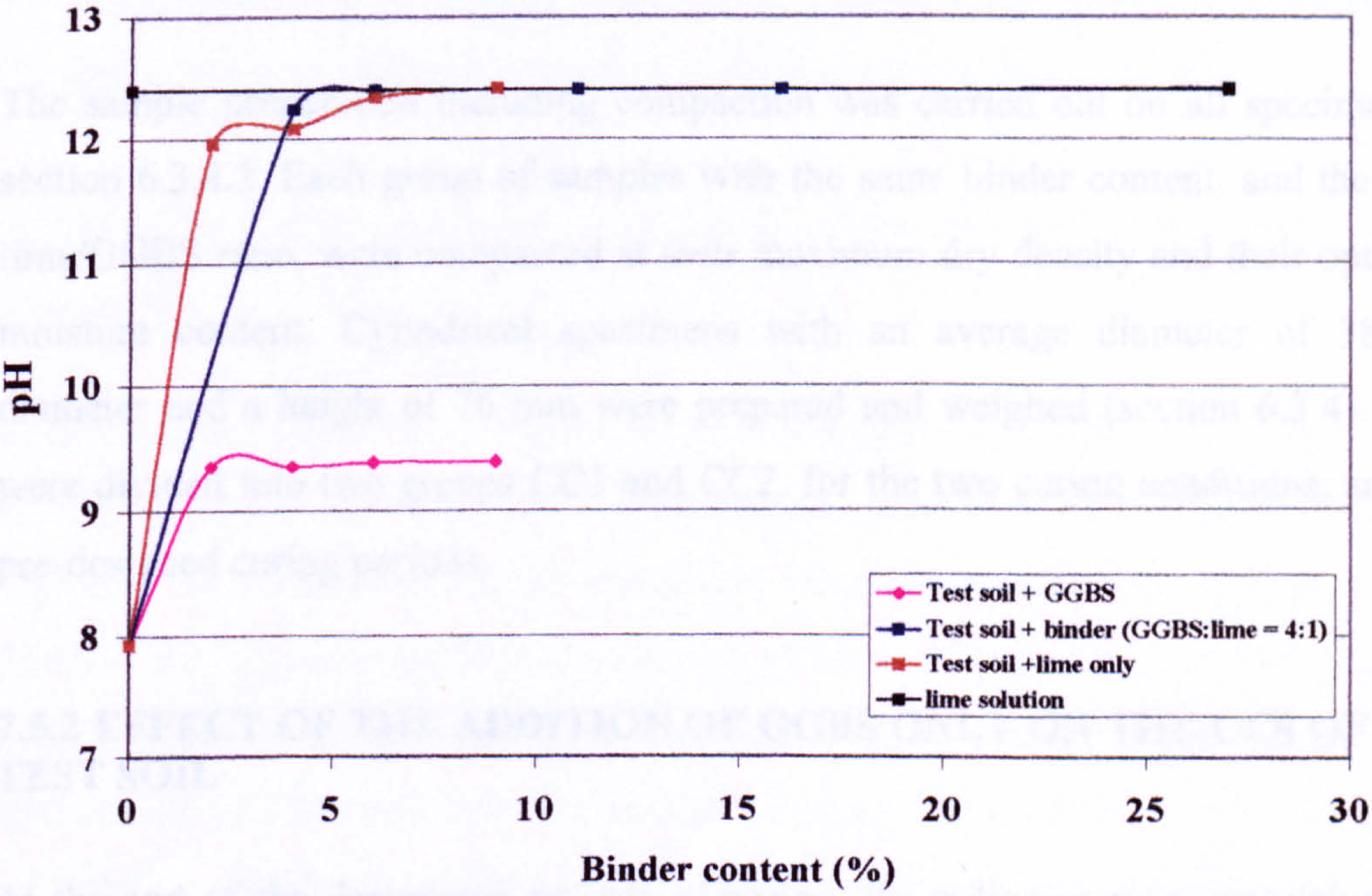

Figure 7.15 Effect of binder content on the pH value of the test soil at 22°C

7.5 UNCONFINED COMPRESSIVE STRENGTH OF GGBS–SOIL AND GGBS–LIME–SOIL MIXTURES

7.5.1 INTRODUCTION

The effects of GGBS with and without lime on the strength characteristics of the test soil were studied. The UCS test was used to assess the strength development of the test soil with varying GGBS content, lime/GGBS ratio, curing conditions and curing periods. The UCS, as described in the British standard (B S 1377-7:1990), was used because it is the simplest and most common test used for monitoring the strength characteristics of cohesive soils. The UCS tests were carried out in two different phases. The first phase was to investigate the effect of the addition of GGBS only on the UCS of the test soil on samples which were cured for different curing periods and conditions. The second phase was to study the effect of GGBS activated by lime on the UCS of the same test soil. Six curing periods were employed for the first phase, 7 and 28 days and 3, 6, 9 and 12 months. Two curing periods were employed for the second phase of tests, 7 and 28 days. The curing conditions are described in section 6.3.4.3 in chapter 6.

The sample preparation including compaction was carried out on all specimens in section 6.3.4.2. Each group of samples with the same binder content, and the same lime/GGBS ratio, were compacted at their maximum dry density and their optimum moisture content. Cylindrical specimens with an average diameter of 38 mm diameter and a height of 76 mm were prepared and weighed (section 6.3.4). They were divided into two groups CC1 and CC2, for the two curing conditions, and the pre-designed curing periods.

7.5.2 EFFECT OF THE ADDITION OF GGBS ONLY ON THE UCS OF THE TEST SOIL

At the end of the designated periods of curing, the cylinders were reweighed and were tested in the universal test machine, see section 6.3.4. In the current work care was exercised to avoid moisture loss during curing, as compressive strength is thought to be sensitive to any significant changes in the moisture content. It is therefore important to test each group of specimens at approximately the same curing conditions. The moisture content of the specimens during curing periods varied. Therefore, any sample with a loss in its moisture content during curing of greater than 5 % of the initial moisture content in the first 3 months or 15% after 12 months of curing was discarded. These variations in the initial moisture content were the minimum losses obtainable, and should have a relatively small effect on the compressive strength and were therefore considered acceptable. Wild *et al.,* (1998) in their study on the UCS of kaolinite and Kimmeridge clay, found that the moisture content varied within ± 5%, of the initial moisture content of the samples after 28 days, and therefore any loss or gain in moisture by the sample during curing was negligible.

The method used in the determination of the unconfined compressive strength results was recommended by the American Society for Testing and Materials (1988). In this procedure each test is repeated three times and each data set analysed separately and the mean of the three values obtained for each specific test condition was taken to be representative, unless one of the individual values deviated by more than 10% from the mean. When this took place, the deviant value was disregarded and the mean of the remaining two results was taken as a representative value.

7.5.2.1 Effect of GGBS only on the UCS of the test soil with varying curing conditions

The effects of GGBS only on the UCS of the test soil treated with 0 to 10% of GGBS and cured under CC1 and CC2 conditions are presented in figures 7.16, 7.17, 7.18 and 7.19 and table 7.6. All specimens were cured from 7 days to 12 months. The test soil alone showed small increases of strength with curing from 882 kN/m^2 to 920 kN/m^2 after 12 months curing. With a short curing period (28 days) the addition of GGBS increased the strength slightly (up to 4% binder) and it then remained a relatively constant under both curing conditions CC1 and CC2 (see figures 7.16 and 7.18). Longer curing periods resulted in greater increases in strength, especially at higher GGBS levels and at the higher temperature (figures 7.17 and 7.19). The UCS increased from 882 kN/m^2 for the test soil alone to 1108 kN/m^{2}for the test soil with 10% GGBS, cured for 12 months, under curing conditions CC1, about 25% increase from the initial value of the strength.

7.5.2.2 Effect of curing periods on the UCS of the test soil under different curing conditions

The effects of curing periods on the UCS of the test soil only, cured under both CC1 and CC2 conditions, for curing periods vary from 7 days to 12 months are presented in figures 7.20, 7.21, 7.22 and 7.23. No significant increase in the UCS of the test soil alone was found when cured under either curing condition, except a very small increase probably due to the effect of drying especially at higher temperature. General and significant increases were observed in the UCS of the test soil with the addition of GGBS, cured under either CC1 or CC2 conditions.

The UCS increases gradually with an increase in GGBS, with increasing curing period and increases in the curing temperature. The rate of increase in the UCS increases with increases in GGBS (see figures 7.20 to 7.23). For example, the addition of 2% of GGBS to the test soil slightly increased the UCS from 887 kN/m^2 after 7 days to 961 kN/m^2 after 12 months under CC1condition, while the addition of 6% of GGBS increased the UCS to 1066 kN/m^2 under the same curing condition and period. The addition of 4% GGBS increased the UCS of the test soil to 1026 kN/m^2 after 12 months, under curing conditions CC1, while the UCS increased to 1301 kN/m^2 with the same GGBS content and the same curing period under CC2.

Table 7.6 Unconfined compressive strength of the test soil and GGBS

GGBS (%)	Curing Period	UCS (kN/m^2)	
		20°C	35°C
0	7 days	882	920
2		887	940
4		899	1100
6		883	978
8		854	945
10		842	892
0	28 days	890	935
2		892	955
4		948	1185
6		941	1275
8		935	1210
10		931	1105
0	3 months	904	945
2		925	985
4		979	1210
6		984	1392
8		955	1255
10		950	1232
0	6 months	912	955
2		940	1045
4		990	1240
6		995	1410
8		1006	1464
10		1012	1425
0	9 months	916	962
2		955	1066
4		1010	1285
6		1031	1438
8		1074	1492
10		1089	1509
0	12 months	920	974
2		961	1088
4		1026	1301
6		1066	1444
8		1085	1522
10		1108	1555

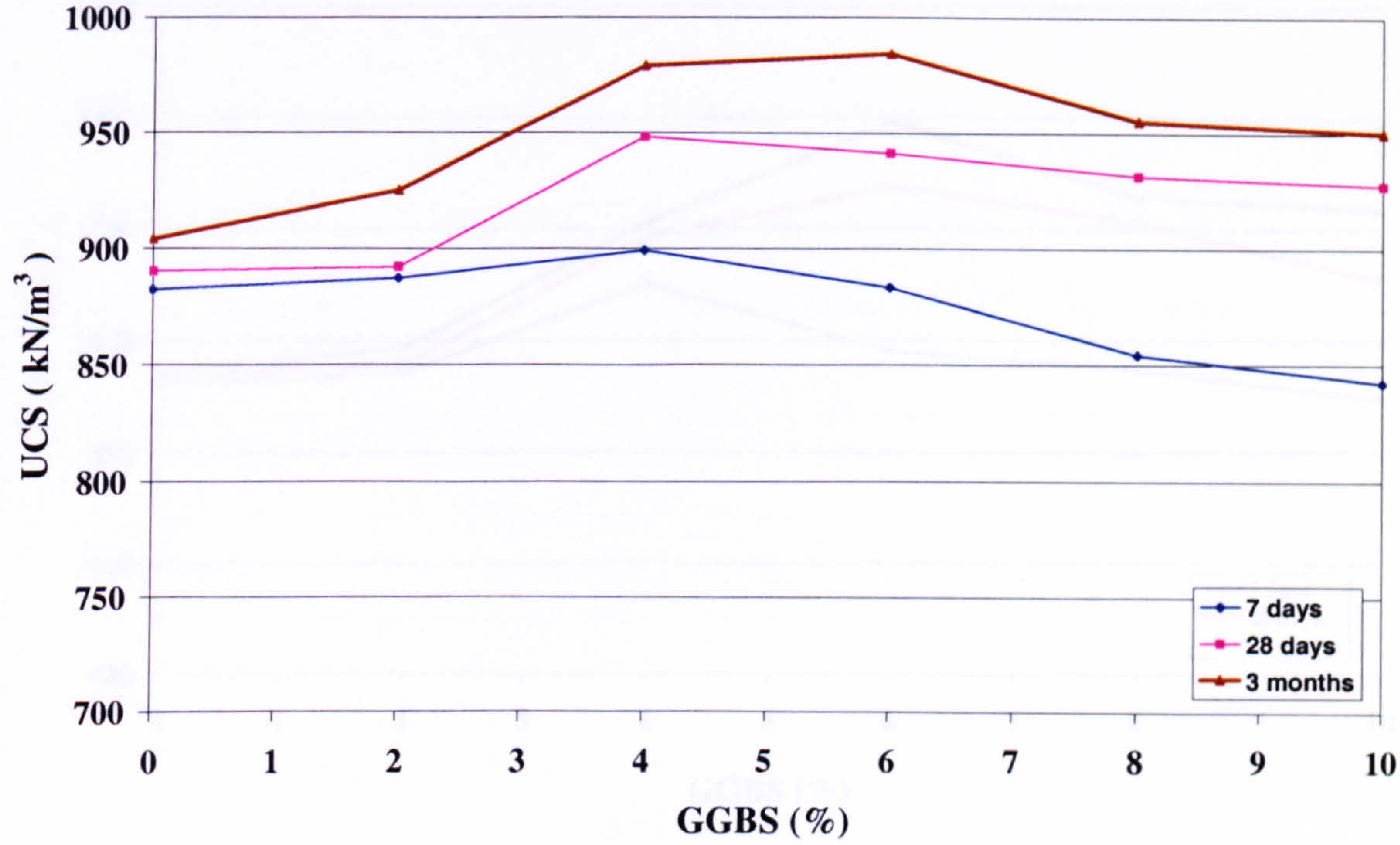

Figure 7.16 Effect of GGBS (%) on the UCS of the test soil cured under CC1 conditions after 7, 28 days and 3 months

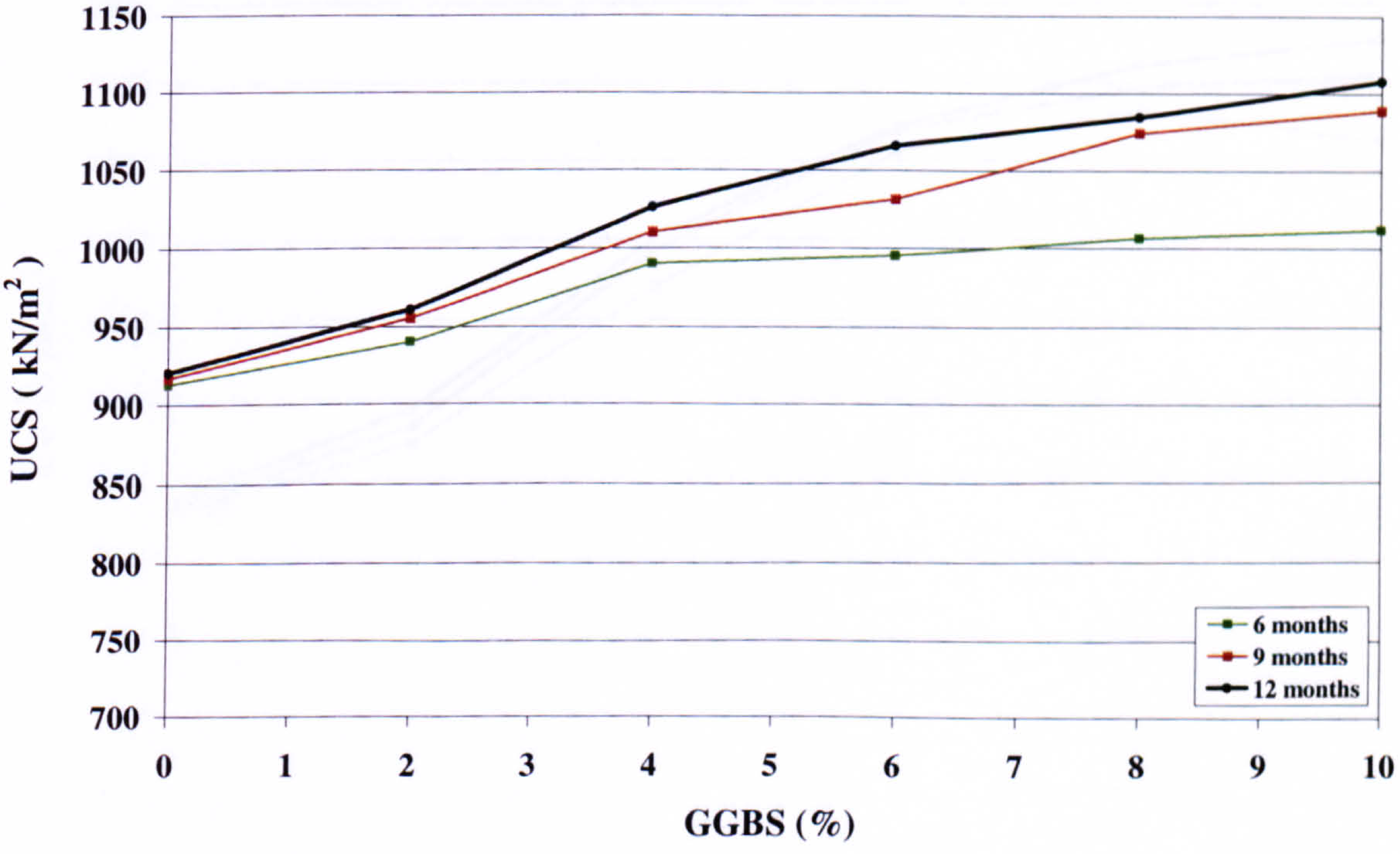

Figure 7.17 Effect of GGBS (%) on the UCS of the test soil cured under CC1 conditions after 6, 9 and 12 months

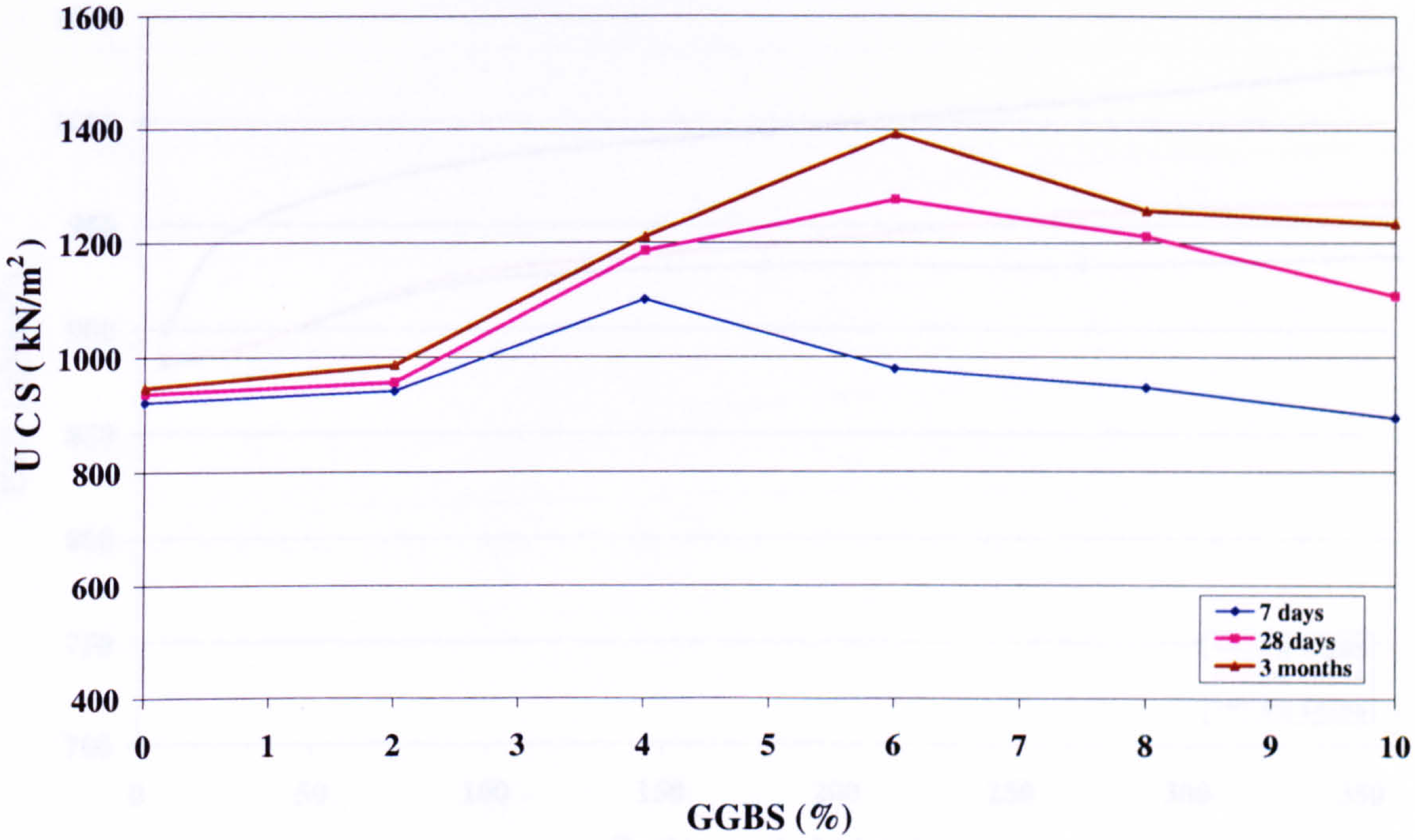

Figure 7.18 Effect of GGBS (%) on the UCS of the test soil cured under CC2 conditions after 7, 28 days and 3 months

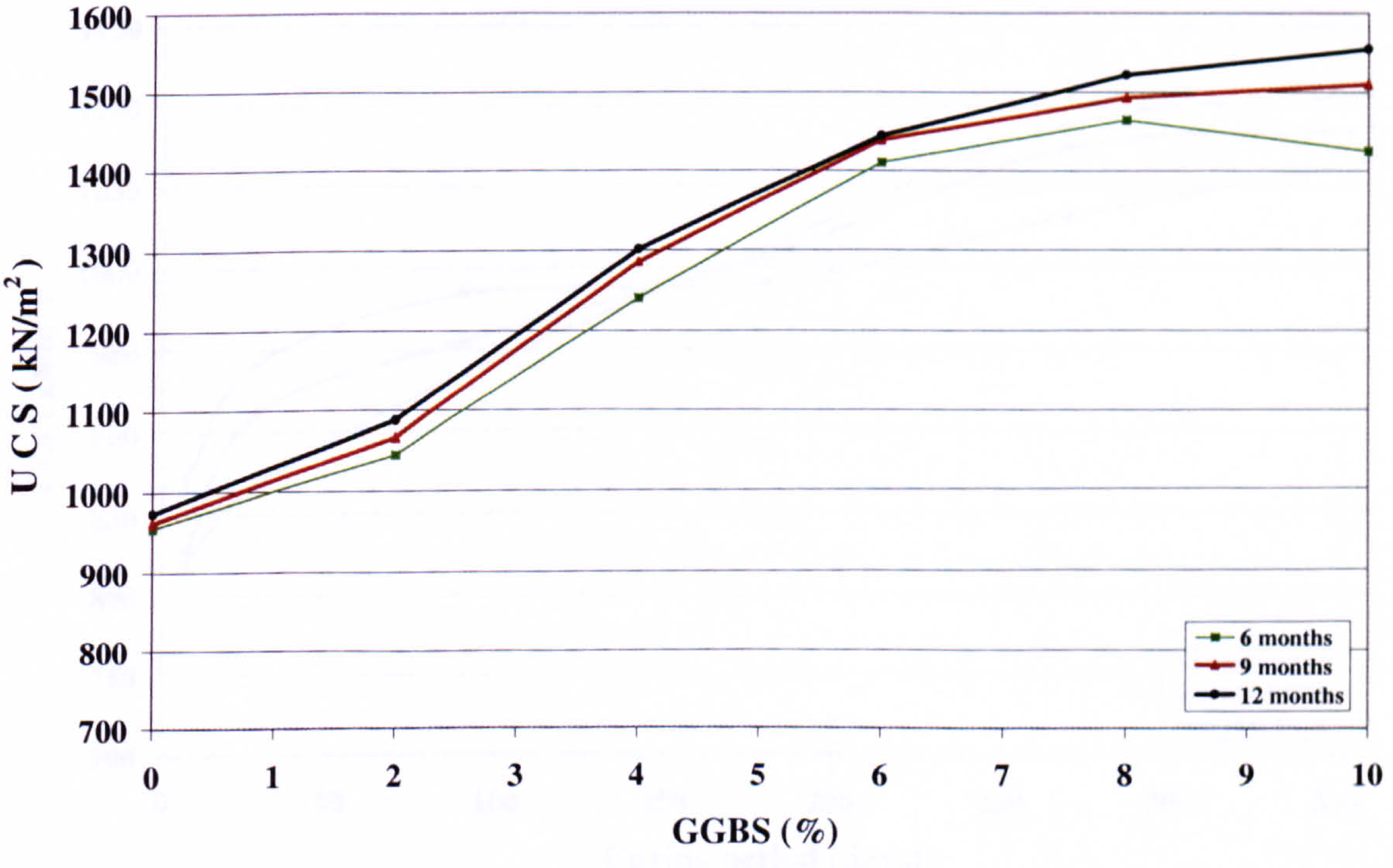

Figure 7.19 Effect of GGBS (%) on the UCS of the test soil cured under CC2 conditions after 6, 9 and 12 months.

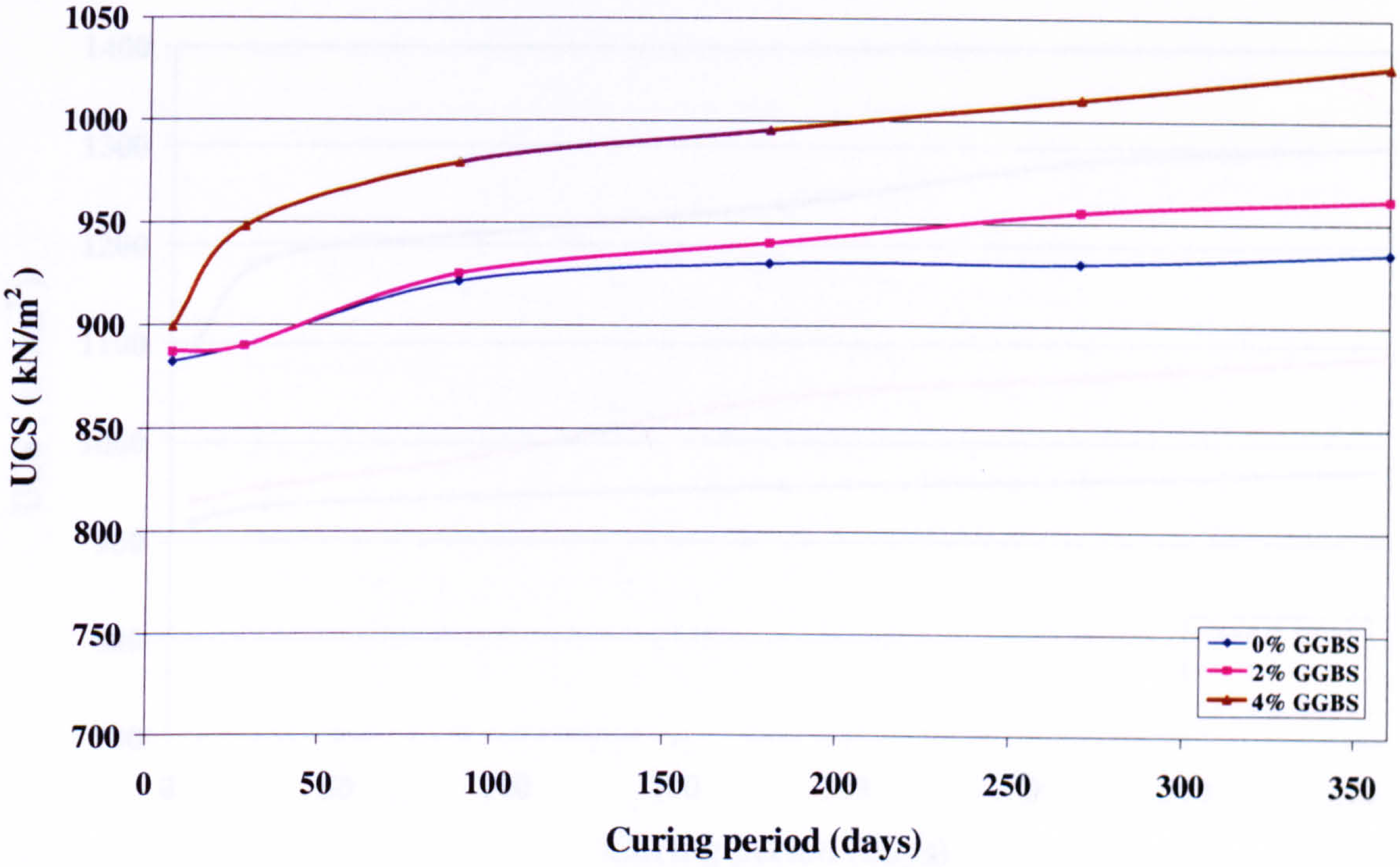

Figure 7.20 Effect of curing period on the UCS of the test soil mixed with 0, 2 and 4% GGBS cured under CC1 conditions

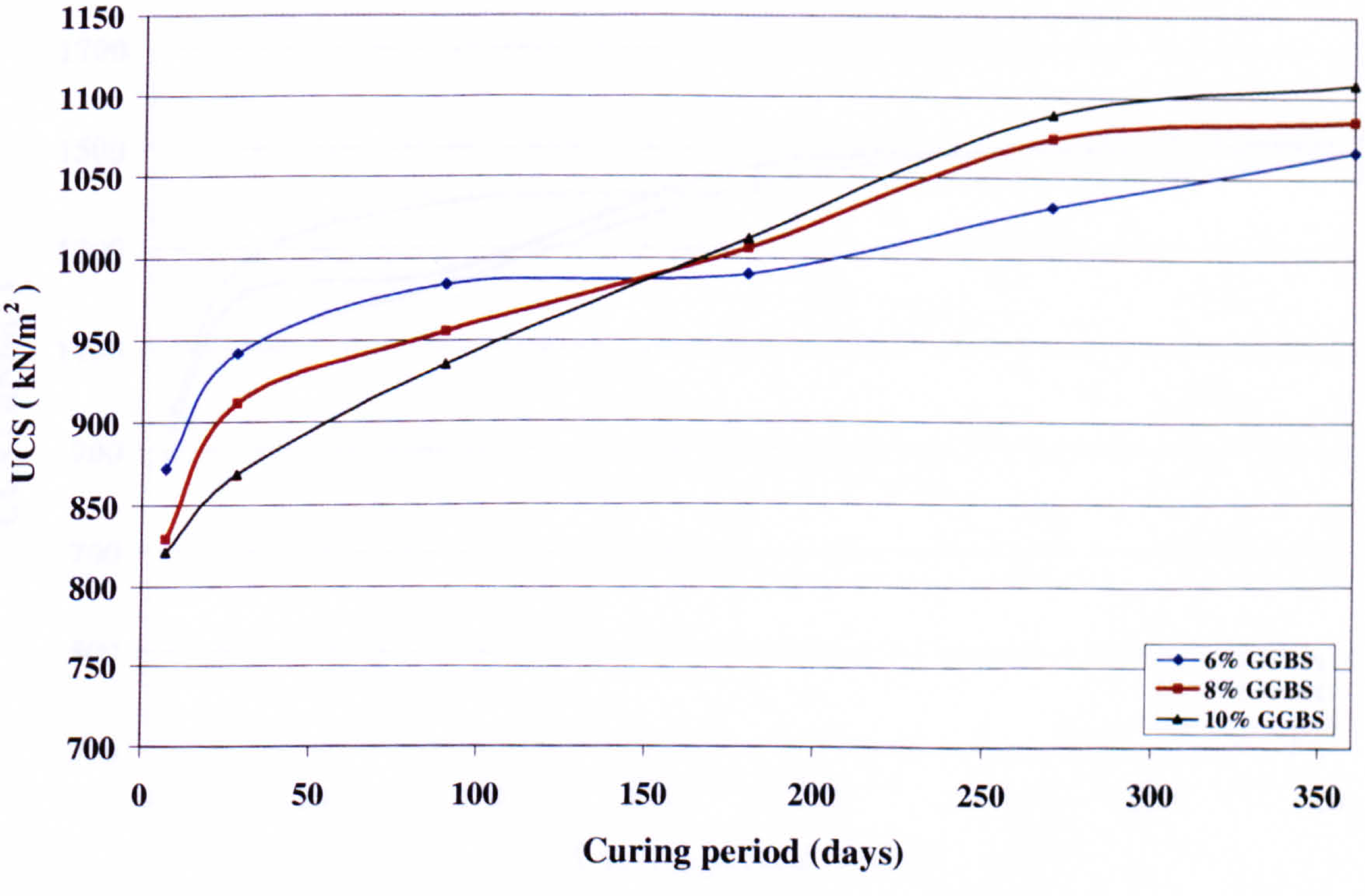

Figure 7.21 Effect of curing period on the UCS of the test soil mixed with 6, 8 and 10 %GGBS cured under CC1 conditions

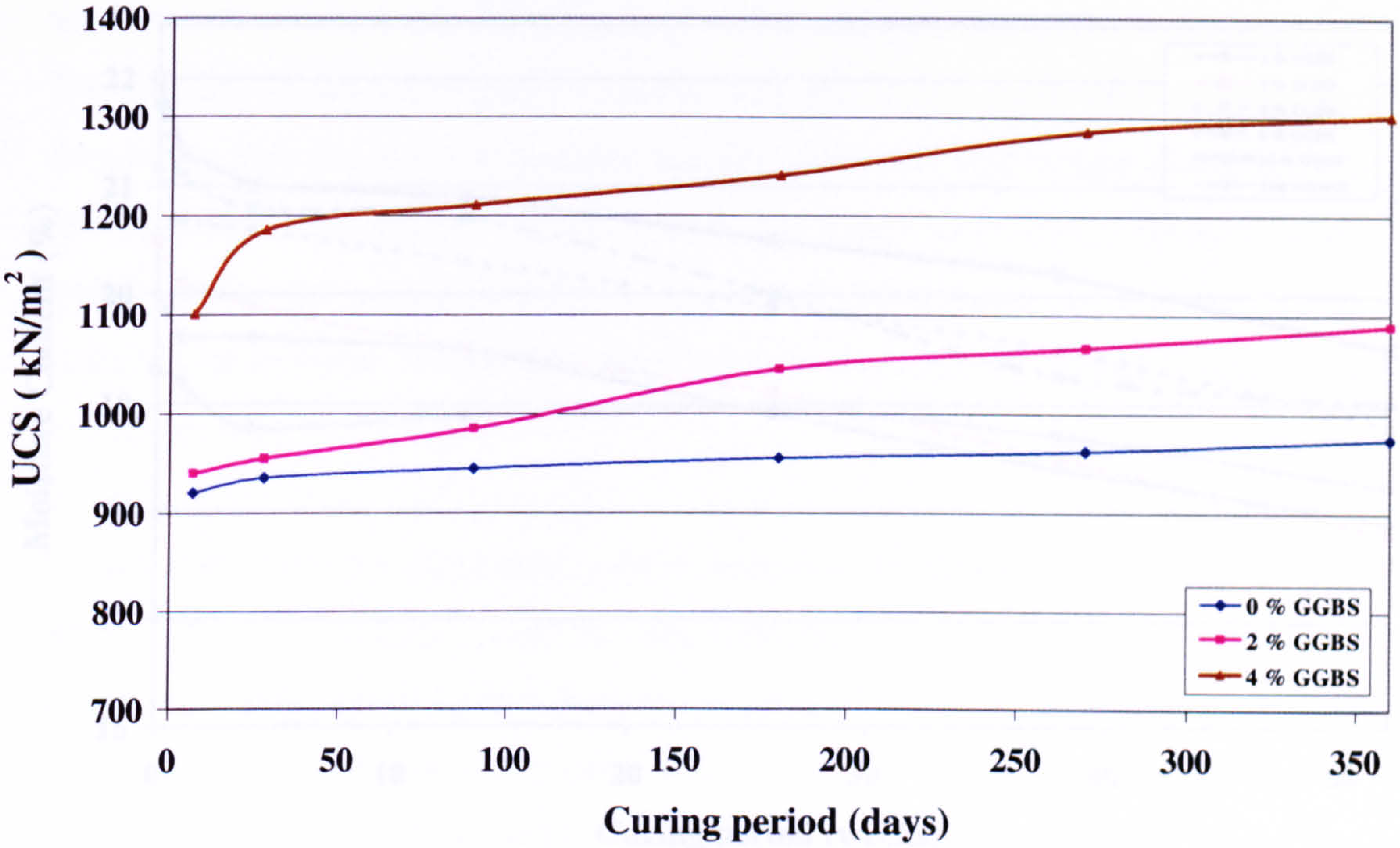

Figure 7.22 Effect of curing period on the UCS of the test soil mixed with 0, 2 and 4 %GGBS, cured under CC2 conditions

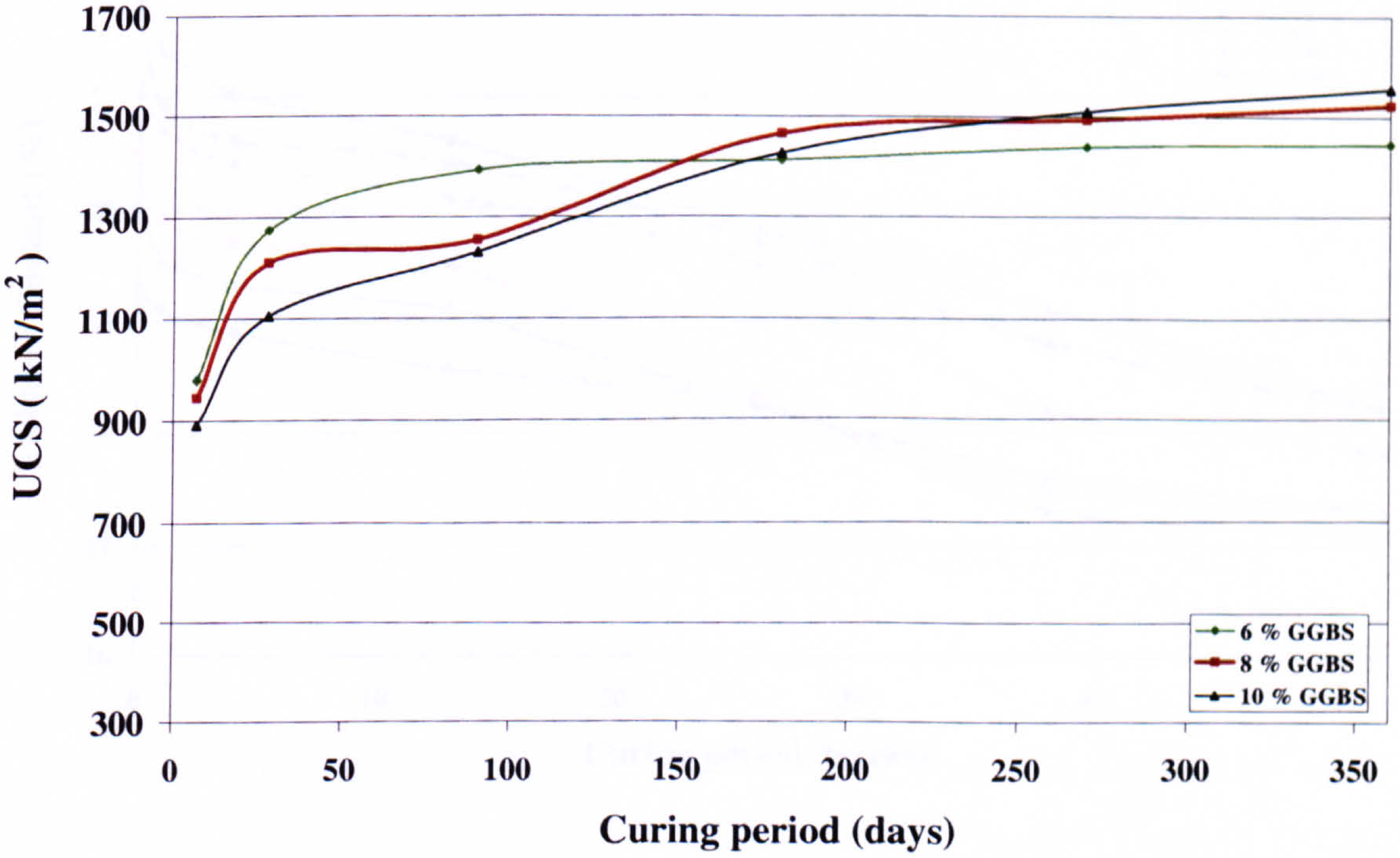

Figure 7.23 Effect of curing period on the UCS of the test soil mixed with 6, 8 and 10 %GGBS, under CC2 conditions

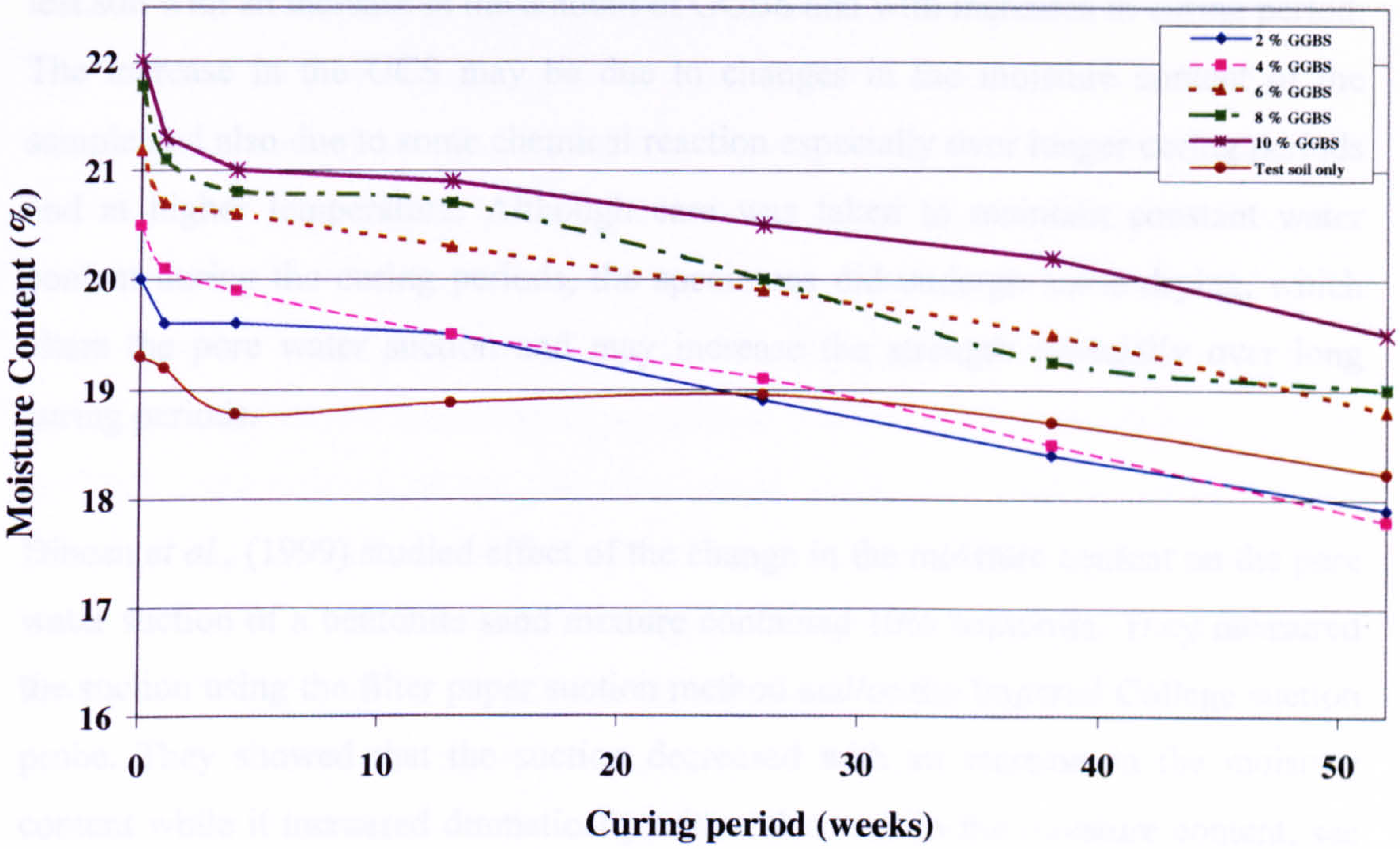

Figure 7.24 Moisture content vs curing period for test soil and GGBS only cured under CC1 conditions

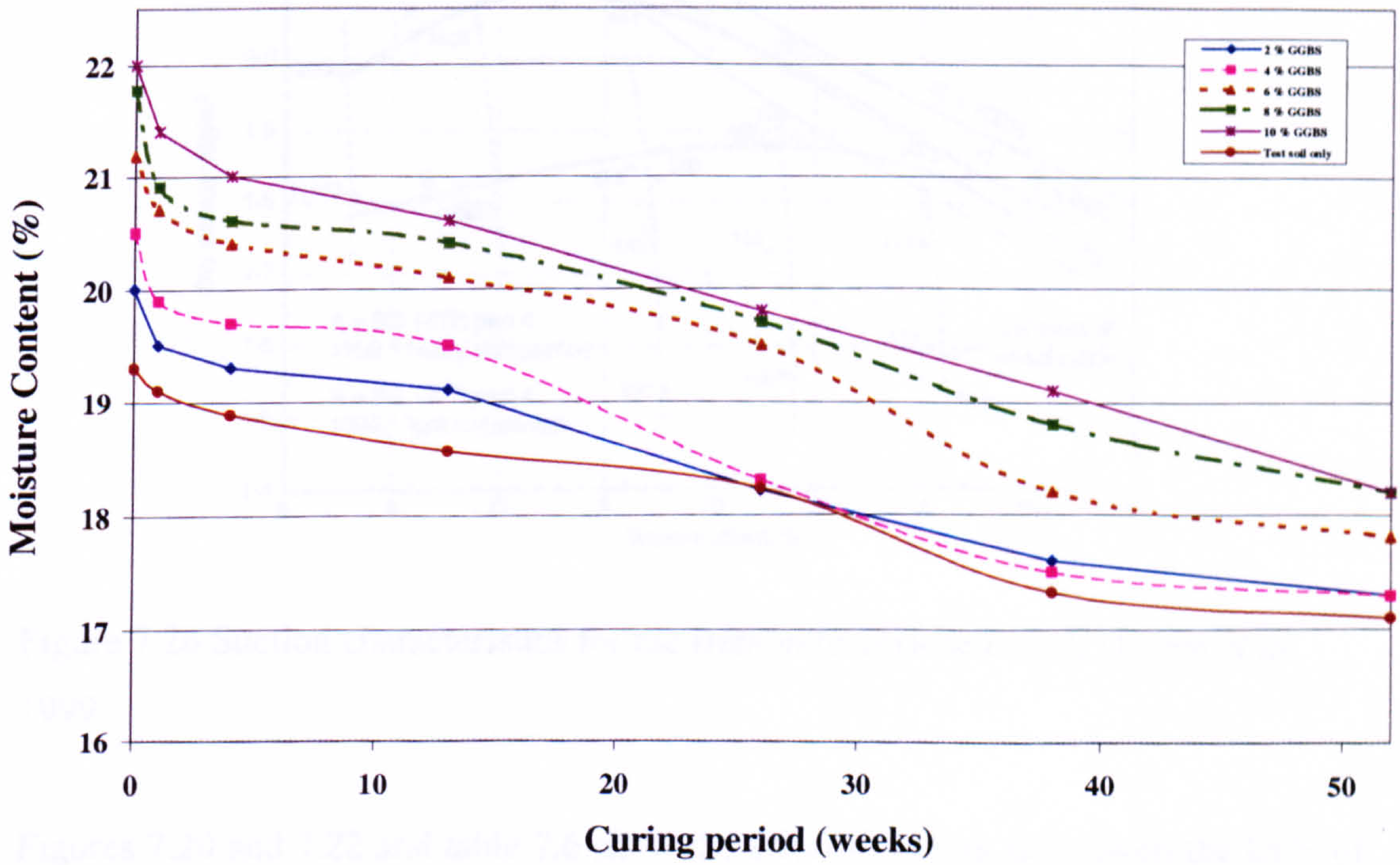

Figure 7.25 Moisture content vs curing period for test soil and GGBS only cured under CC2 conditions

From the previous results, it can be seen that there is an increase in the UCS of the test soil with an increase in the amount of GGBS and with increases in curing period. The increase in the UCS may be due to changes in the moisture content of the sample and also due to some chemical reaction especially over longer curing periods and at higher temperature. Although care was taken to maintain constant water content during the curing periods, the specimens did undergo some drying, which alters the pore water suction and may increase the strength especially over long curing periods.

Dineen *et al.*, (1999) studied effect of the change in the moisture content on the pore water suction of a bentonite sand mixture contained 10% bentonite. They measured the suction using the filter paper suction method and/or the Imperial College suction probe. They showed that the suction decreased with an increase in the moisture content while it increased dramatically with a decrease in the moisture content, see figure 7.26.

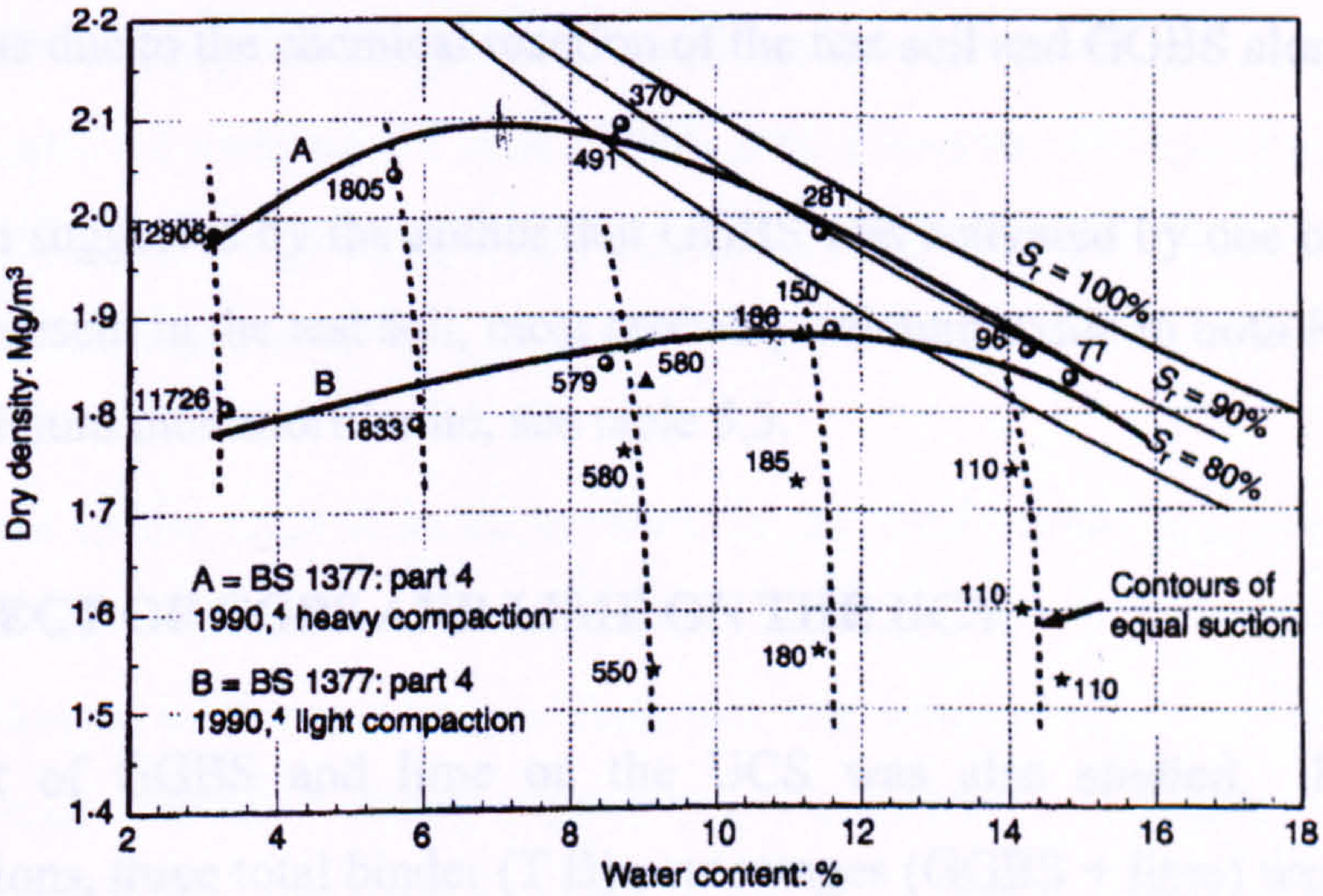

Figure 7.26 Suction characteristics for the Bentonite-enriched sand, Dineen *et al*, 1999

Figures 7.20 and 7.22 and table 7.6 show the effect of curing periods on the UCS of the test soil only and the test soil and GGBS. In the current work the pore water suction was not measured, however, the effect of the change in the moisture content on the UCS of the test soil can be estimated as follows. For any mix of the test soil and GGBS, the UCS of the test soil alone can be deducted from that of this particular mix at the required curing conditions and period. For example, the UCS of the test soil alone cured under CC1 after 7 days (control sample) is 882 kN/m^2 and increasing the curing period to 6 months caused an insignificant increase in the UCS to 912 kN/m^2 (3%), while the addition of 4% GGBS increased the UCS to 990 kN/m^2 (11% from the control sample). From this example it can be observed that the increase in the UCS due to the change in the pore water suction is 20 kN/m^2 while the increase due to the formation of pozzolanic reaction products is 78 kN/m^2. Also, the UCS for a mixture of test soil alone cured under CC1 after 12 months is 920 kN/m^2 (4.5%), the addition of 10% GGBS increased the UCS to 1108 kN/m^2 (25%). It can be observed that the increase in the UCS due to the change in the pore water suction is 38 kN/m^2 while the increase due to the chemical reaction is 188 kN/m^2. It can be also observed that the effect of change in the pore water suction on the UCS increases with an increase in the curing temperature and period. Also, the change in the UCS due to the change in the pore water suction is relatively large compared to the increase due to the chemical reaction of the test soil and GGBS alone.

It has been suggested by the author that GGBS was activated by one or more oxides naturally present in the test soil, most probably calcium oxide in both River Aire soil and the calcium montmorillonite, see table 5.3.

7.5.3 EFFECT OF GGBS AND LIME ON THE UCS

The effect of GGBS and lime on the UCS was also studied. For economic considerations, three total binder (T B) percentages (GGBS + lime) were used in this investigation 2, 4 and 6%. Three different percentages of GGBS replacement by lime (10, 20 and 30%) were used with the three total binders, see section 7.3. The same curing conditions, CC1 and CC2 were used.

The effects of GGBS activated by lime on the UCS of the test soil are presented in figures 7.27 to 7.32 and table 7.7. The general effect of GGBS only on the UCS of the test soil is small compared with the effect of GGBS activated by lime on the UCS. It can be seen that generally the UCS of the test soil increased with an increase in the total binder and with an increase in lime/GGBS ratio. The UCS also increased with an increase in the curing period and an increase in the curing temperature for the same binder and lime/GGBS ratio.

For example, the UCS of the test soil with 2% binder, after 7 days curing, increased slightly from 887 kN/m^2 to 933 kN/m^2 with an increase in the lime/GGBS ratio from 0 to 0.30 under CC1 conditions. The UCS increased to 1121 kN/m^2 with an increase in the curing period to 28 days, keeping all the others conditions constant. However, the UCS increased more than twice with an increase in the binder content to 4%, keeping all the previous conditions constant. Mateos (1964) found that clay specimens cured at 35°C produced twice the strength of those cured at 25°C which implies the great effect of the temperature on the UCS. Also, a further increase was observed in the UCS to 2750 kN/m^2 with an increase in the binder content to 6%, keeping all curing conditions and period constant (CC1, 28 days curing, lime/GGBS ratio = 0.30). It was also observed that the UCS increased to approximately the same level (2753 kN/m^2) with only 4% total binder but with changing the curing conditions to CC2 keeping the lime/GGBS ratio constant. The maximum increase in the UCS was observed at 6% total binder, lime/GGBS of 0.30, under CC2 conditions after 28 days curing, see table 7.7.

The increase in the UCS with an increase in the total binder probably results from the reaction of the GGBS activated by lime with the clay portion of the test soil and the formation of new cementitious materials, see chapter eight. Many investigators have observed that high curing temperatures accelerate the reaction and result in higher early strength gain (Bell, 1988; Wild *et al*, 1987 and others). The rate of formation of cementitious materials, which is the main reason for the strength development increases at the high temperatures. The strength of the test soil also increases with an increase in the curing period due to the development in the crystallinity and percentage of the cementitious materials. The increase in the UCS with increases of the lime/GGBS ratio at the same binder content is due to the GGBS needing a sufficient amount of lime to activate it. The optimum amount of lime depends primarily on the type of soil and curing conditions. Higgins *et al.,* (1998) found that the optimum lime/GGBS ratio to achieve maximum UCS is 1:5 (the total binder was 6% of the dry soil weight) for kaolinite clay and that the optimum is about 2/3 for Kimmeridge clay (the total binder was 5%).

Wild *et al.,* (1998) found no effect with GGBS alone on both the 7 and 28 day strengths of kaolinite while it produced significant strength enhancement in case of Kimmeridge clay. Replacement of lime by GGBS with Kimmeridge clay provided a maximum strength at a different lime/GGBS ratios depending on the total stabiliser percentage. In the case of kaolinite, strength increases with a decrease in lime/GGBS ratio, although the strength falls with no lime content to activate the GGBS

It was also suggested by Wild and Tasong (1999) that a lime/GGBS ratio of 1:5 is enough to activate GGBS. They also suggested that the lime activated GGBS hydration reaction is quicker than the pozzolanic reaction of lime with clay and the main reaction products are C-A-S-H gel and hydrotalcite. Detailed discussion is presented in chapter nine.

Table 7.7 Effect of GGBS and lime on the UCS of the test soil

Lime/GGBS ratio		20°C		35°C	
		7 days	28 days	7 days	28 days
0	2 % GGBS	887	890	940	955
10		895	931	952	1126
20		915	1054	969	1239
30		933	1121	973	1280
0	4 % GGBS	899	948	1100	1185
10		1150	1398	1249	1482
20		1652	2202	1852	2350
30		2041	2561	2152	2753
0	6 % GGBS	883	941	978	1275
10		1150	1429	1201	1549
20		1871	2381	1952	2449
30		2251	2750	2245	2931

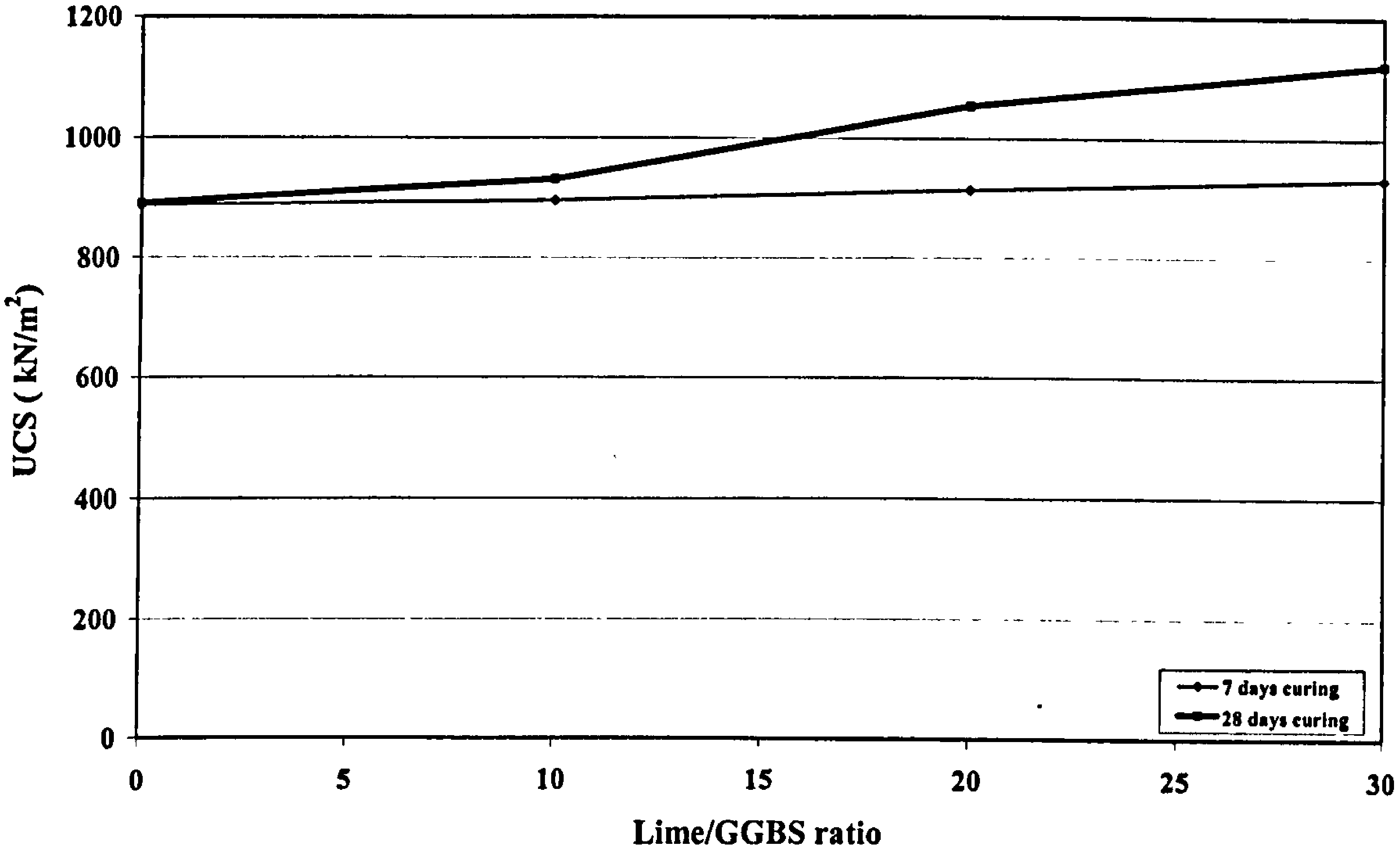

Figure 7.27 Effect of lime/GGBS ratio on the UCS of the test soil at a total binder content of 2 % under CC1 conditions

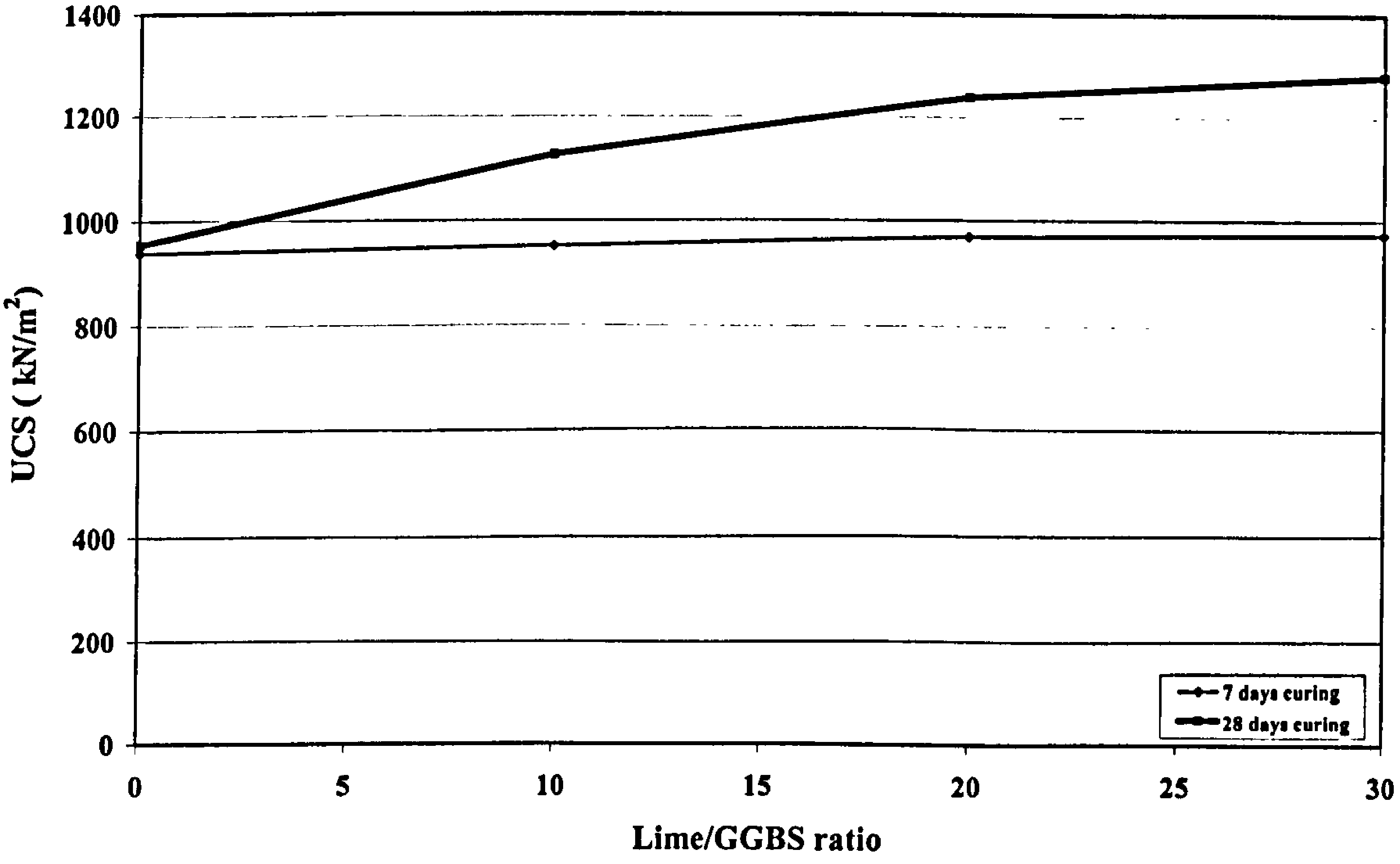

Figure 7.28 Effect of lime/GGBS ratio on the UCS of the test soil at a total binder content of 2 % under CC2 conditions

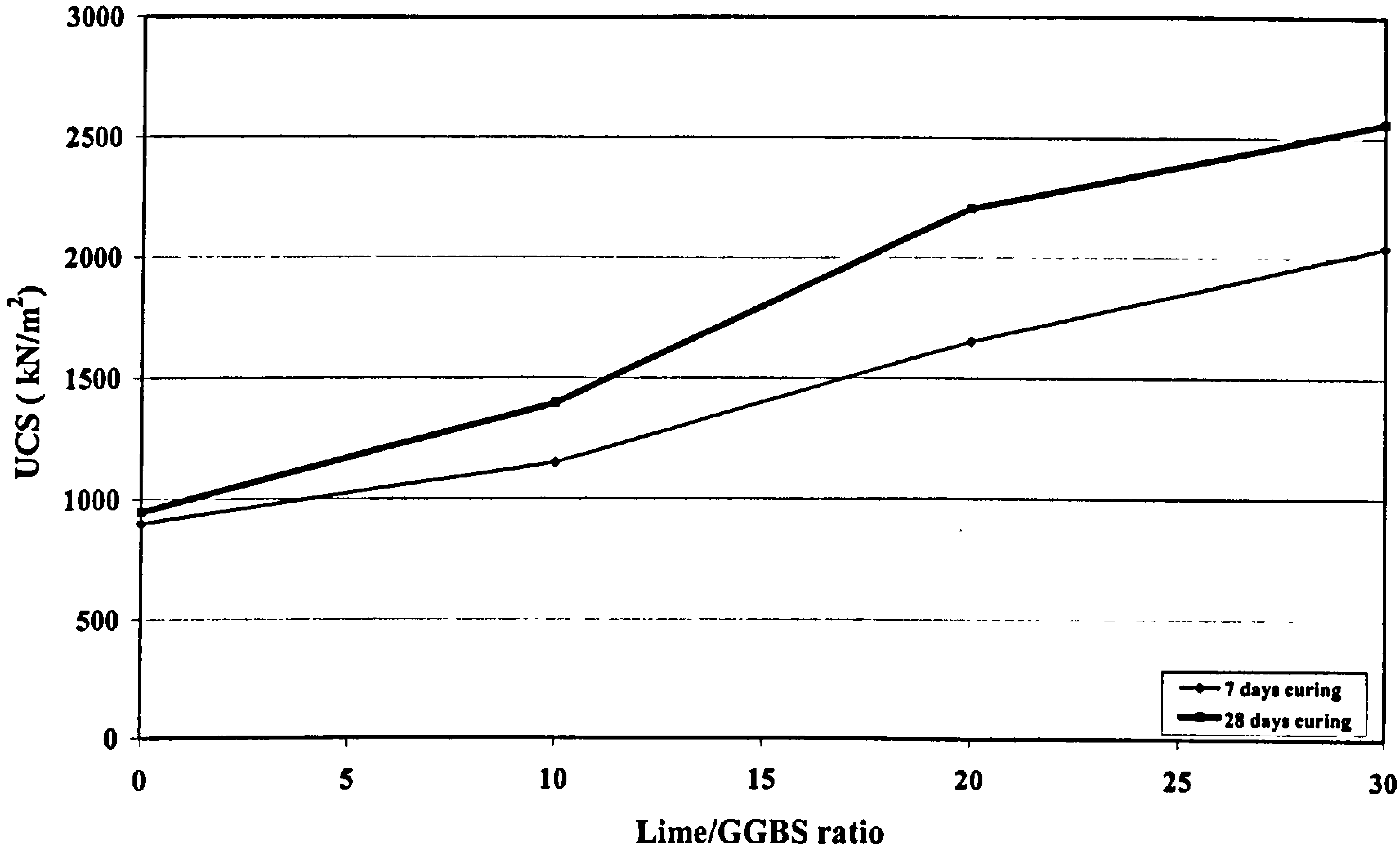

Figure 7.29 Effect of lime/GGBS ratio on the UCS of the test soil at a total binder content of 4 % under CC1 conditions

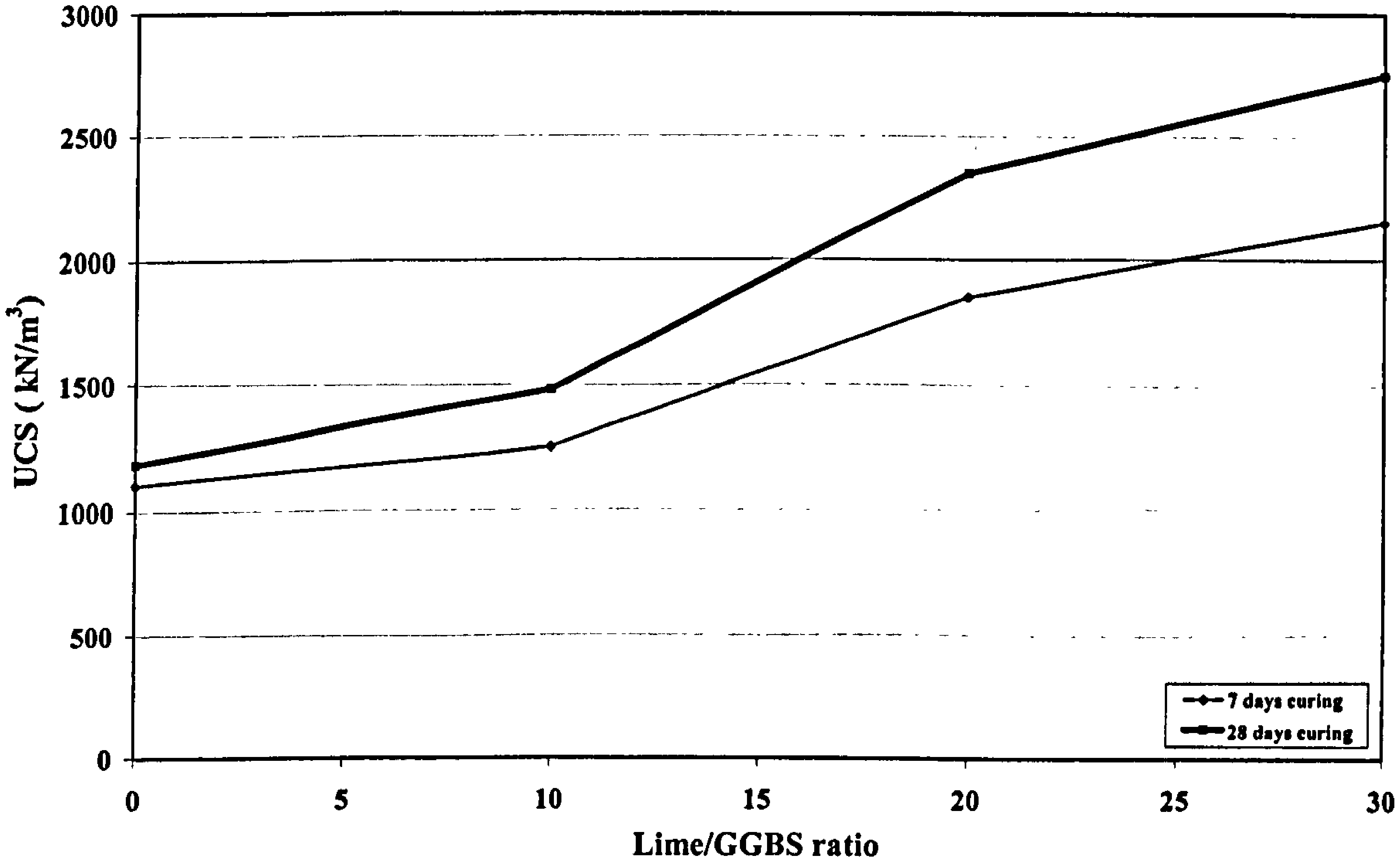

Figure 7.30 Effect of lime/GGBS ratio on the UCS of the test soil at a total binder content of 4 % under CC2 conditions

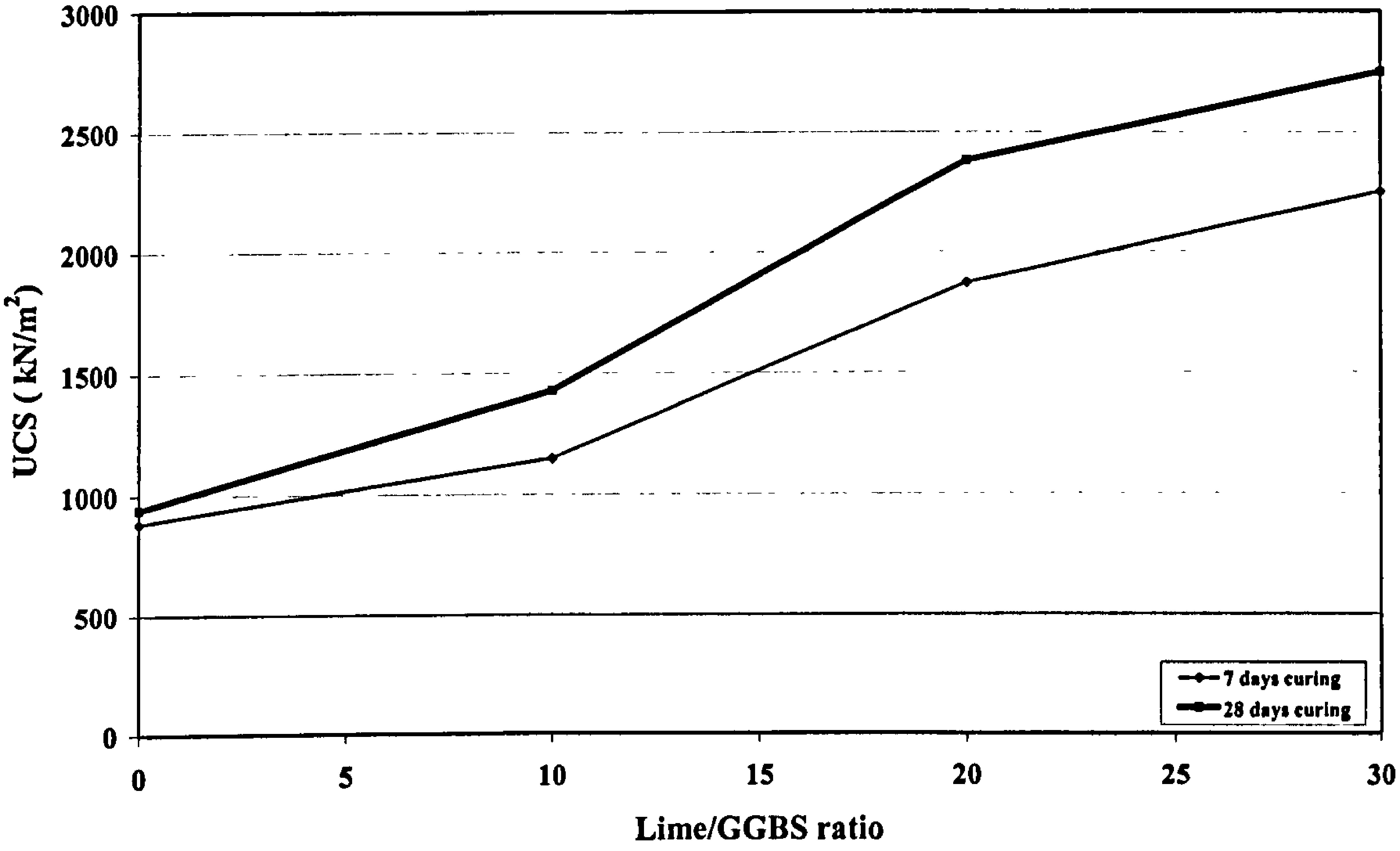

Figure 7.31 Effect of lime/GGBS ratio on the UCS of the test soil at a total binder content of 6 % under CC1 conditions

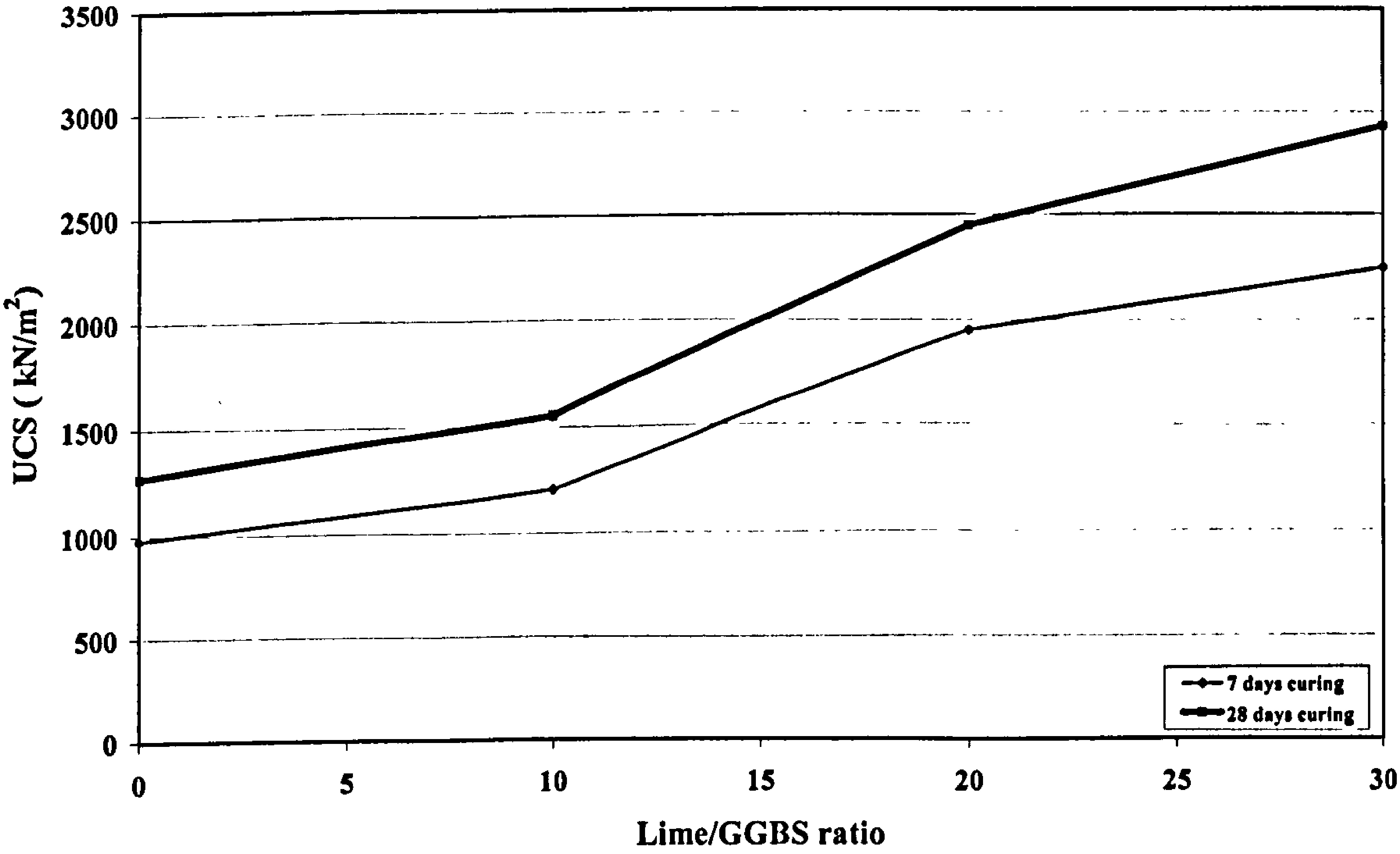

Figure 7.32 Effect of lime/GGBS ratio on the UCS of the test soil at a total binder content of 6 % under CC2 conditions

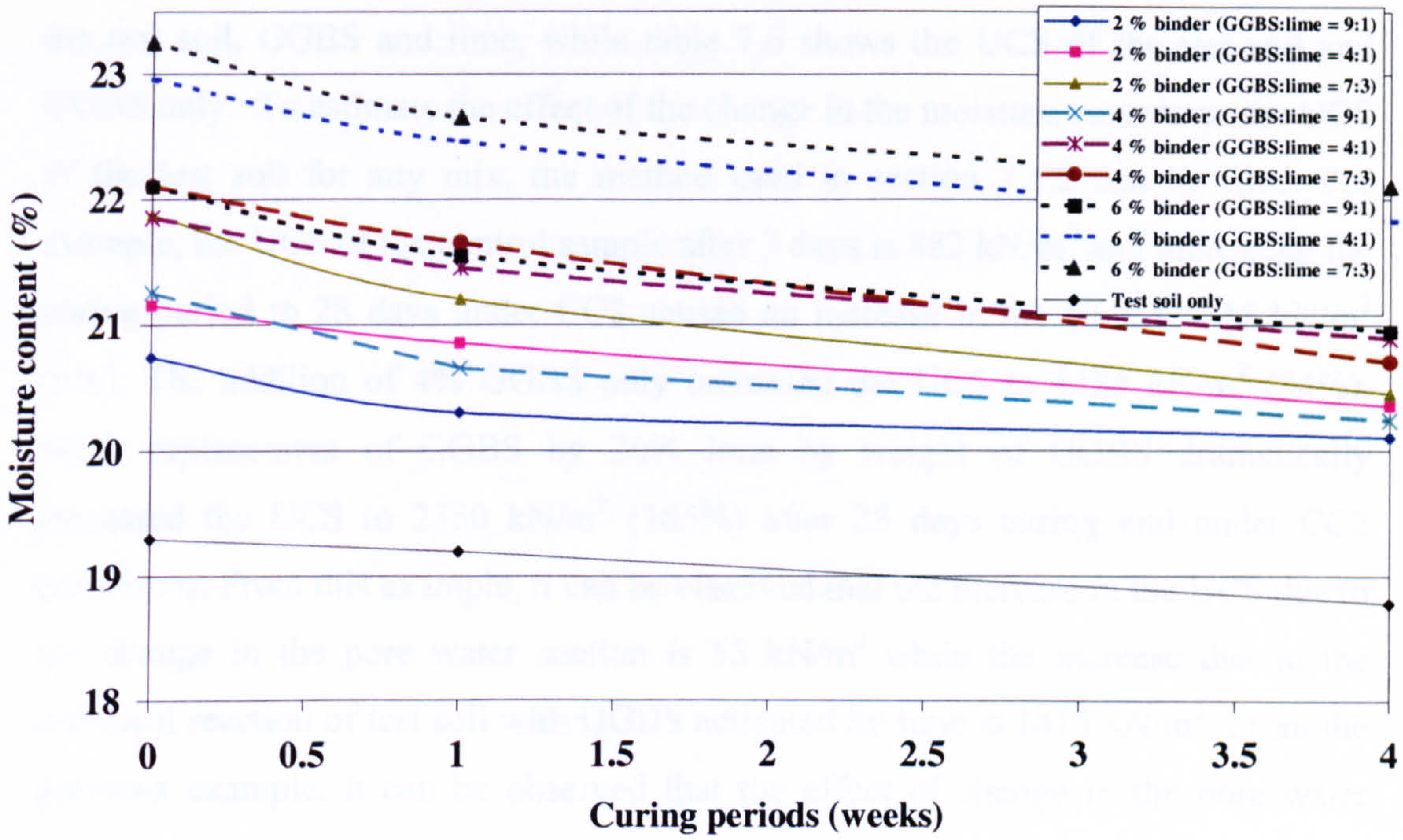

Figure 7.33 Effect of curing period on the initial moisture content for the test soil, GGBS and lime cured under CC1 conditions

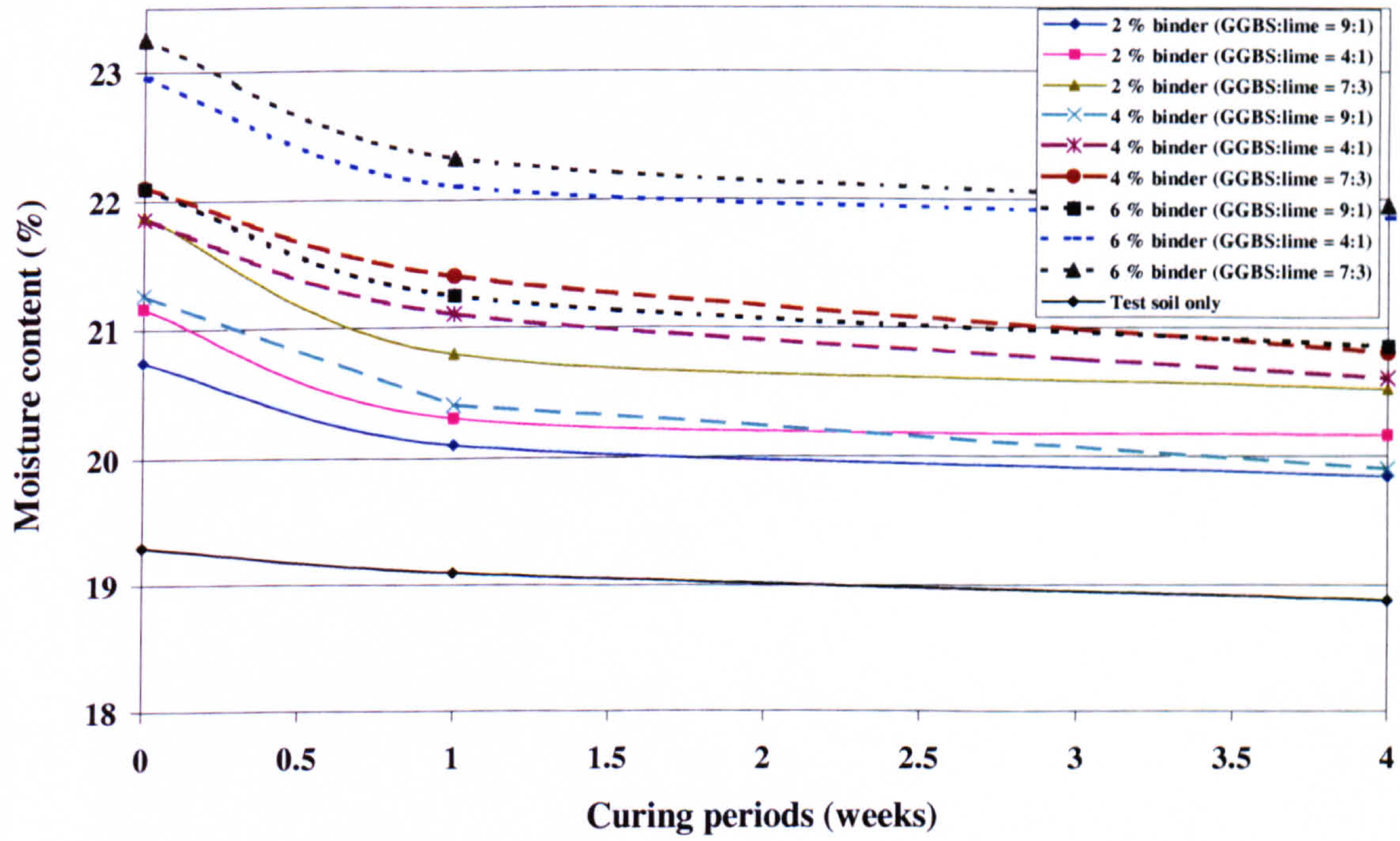

Figure 7.34 Effect of curing period on the initial moisture content for the test soil, GGBS and lime cured under CC2 conditions

Figures 7.27 to 7.32 and table 7.7 show the effect of curing periods on the UCS of the test soil, GGBS and lime, while table 7.6 shows the UCS of the test soil and GGBS only. To estimate the effect of the change in the moisture content on the UCS of the test soil for any mix, the method used in section 7.5.2 can be used. For example, the UCS of the control sample after 7 days is 882 kN/m^2 and increasing the curing period to 28 days under CC2 caused an increase in the UCS to 935 kN/m^2 (6%). The addition of 4% GGBS only increased the UCS to 1185 kN/m^2 (34%), while replacement of GGBS by 20% lime by weight of GGBS dramatically increased the UCS to 2350 kN/m^2 (165%) after 28 days curing and under CC2 conditions. From this example, it can be observed that the increase in the UCS due to the change in the pore water suction is 53 kN/m^2 while the increase due to the chemical reaction of test soil with GGBS activated by lime is 1415 kN/m^2. From the previous example, it can be observed that the effect of change in the pore water suction on the UCS is very small compared to the effect of the chemical reaction on the UCS of the test soil.

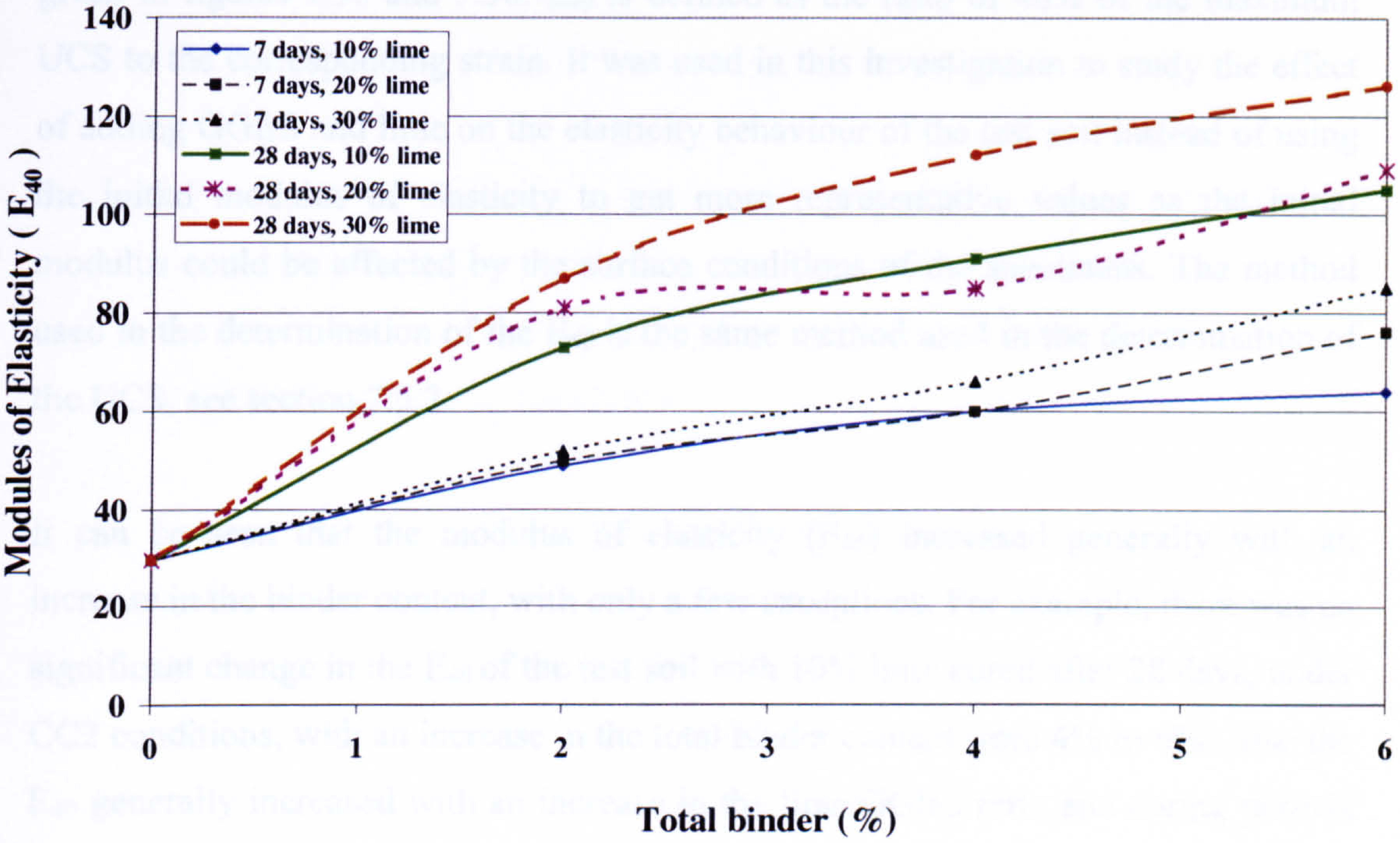

Figure 7.35 Effect of the binder content (%) on modulus of elasticity (E_{40}) under CC1 curing conditions

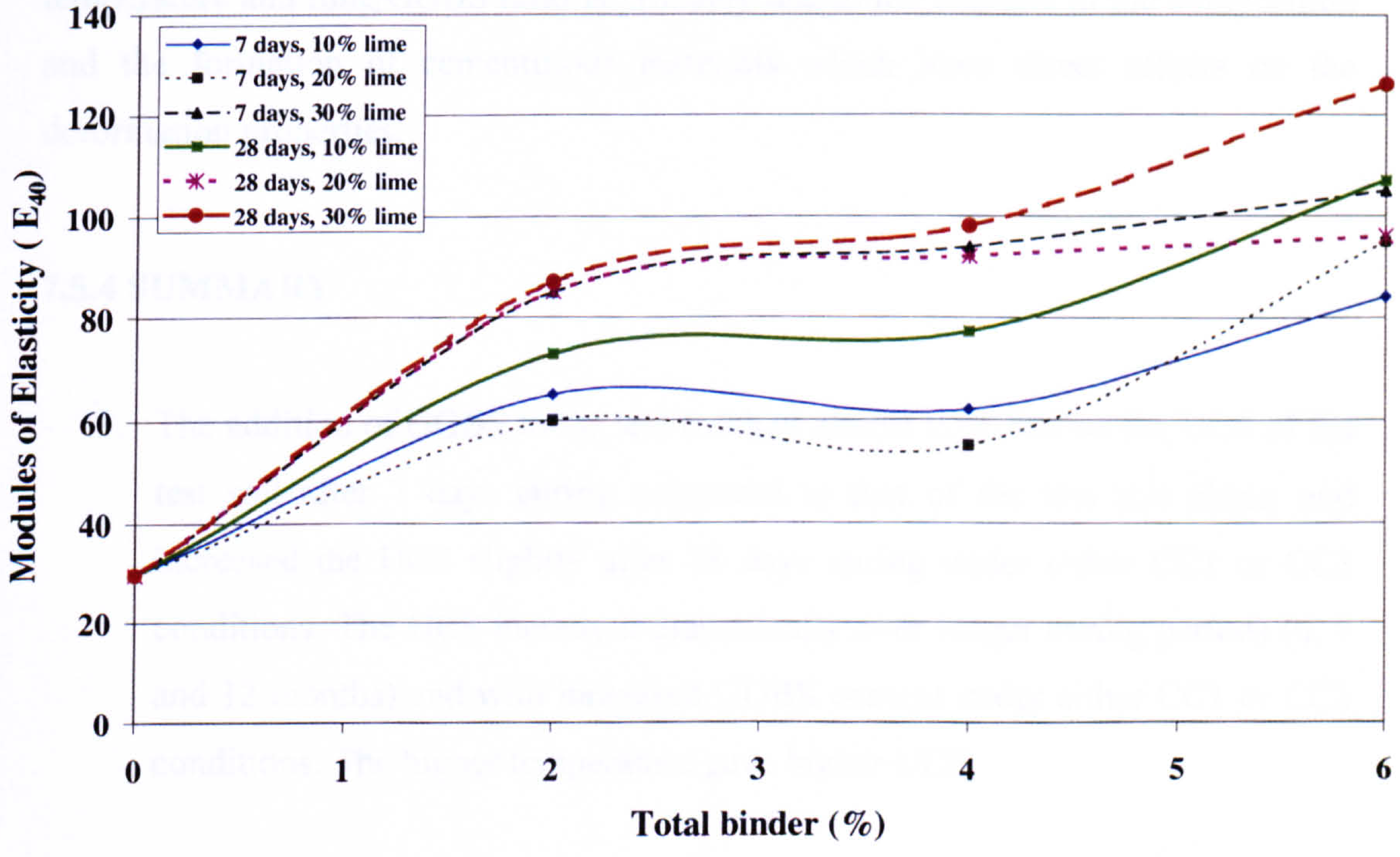

Figure 7.36 Effect of the binder content (%) on modulus of elasticity (E_{40}) under CC2 curing conditions

The effect of GGBS and lime on the σ/ε behaviour as described E_{40} of the test soil is given in figures 7.35 and 7.36. E_{40} is defined as the ratio of 40% of the maximum UCS to the corresponding strain. It was used in this investigation to study the effect of adding GGBS and lime on the elasticity behaviour of the test soil instead of using the initial modulus of elasticity to get more representative values as the initial modulus could be affected by the surface conditions of the specimens. The method used in the determination of the E_{40} is the same method used in the determination of the UCS, see section 7.5.2.

It can be seen that the modulus of elasticity (E_{40}) increased generally with an increase in the binder content, with only a few exceptions. For example, there was no significant change in the E_{40} of the test soil with 10% lime cured after 28 days, under CC2 conditions, with an increase in the total binder content from 4% to 6%. Also the E_{40} generally increased with an increase in the lime/GGBS ratio and curing periods and it increased slightly with an increase in the curing temperature.

The increase in E_{40} with an increase in the binder content, curing period and temperature and lime/GGBS ratio is probably due to the changes in the composition and the formation of cementitious materials which have direct effects on the deformation properties.

7.5.4 SUMMARY

1. The addition of GGBS to the test soil had almost no effect on the UCS of the test soil after 7 days curing compared to that of the test soil alone, and increased the UCS slightly after 28 days curing under either CC1 or CC2 conditions. The UCS increased dramatically over longer curing periods (6, 9 and 12 months) and with increased GGBS content under either CC1 or CC2 conditions. The higher temperature gave higher UCS.

2. The addition of GGBS and lime to the test soil increased the UCS dramatically with an increase in the total binder, and with an increase in lime/GGBS ratio, curing periods and curing temperature.

3. A desired value of the UCS can be obtained either by increasing the total binder content, and/or increasing the lime/GGBS ratio, or changing the curing conditions or periods depending on the availability of the materials, and site environmental conditions.

4. The addition of the GGBS and lime to the test soil generally increased the modulus of elasticity (E_{40}). The E_{40} generally increased with an increase in the binder content (with only a few exceptions), lime/GGBS ratio and curing period and also increased slightly with an increase in curing temperature.

7.6 SWELLING CHARACTERISTICS OF GGBS–SOIL AND GGBS–LIME–SOIL MIXTURES

7.6.1 INTRODUCTION

The objective of this section is to present data and associated discussion concerning the effects of GGBS with and without lime on the swelling characteristics of the test soil. The swelling characteristics of soils can be expressed in many ways e.g. swelling pressure, linear expansion or percentage of free swelling. The free swelling test as described in the British standard, (BS 1377, 1990), was employed in this investigation. Free swelling percent can be used as an index for the classification of expansive soils for engineering purposes and to measure the effect of GGBS with and without lime on the swelling. Certain behavioural characteristics have been associated with soils having free swells that fall within certain ranges. However, only general indications can be drawn from these limits, since the swelling characteristics of expansive clays are greatly affected by their mineralogical composition, weather conditions and chemical environment.

The effects of GGBS on the swelling characteristics of the test soil were studied in two stages. The first was the effect of GGBS alone on the swelling characteristics of the test soil, cured for different curing periods and different curing conditions. The second was to study the effect of GGBS activated by lime on the swelling of the test soil. Two different curing conditions were employed (CC1) and (CC2), for both two phases. Six curing periods were employed for the first phase, 7 and 28 days and 3, 6, 9 and 12 months while two curing periods were employed for the second phase of tests, 7 and 28 days.

The method used in the determination of the free swelling results is described below. In this procedure two tests were carried out for each combination of slag-soil and slag-lime-soil and the average of the two values obtained for each specific test condition. The average was taken to be a representative unless the values deviate by more than 5% from the mean. If so, a third test was carried out and the mean was recalculated for the triplicate results unless one of the values deviates by more than 5% from the mean. When this takes place, the deviant value is disregarded and the mean of the remaining two results taken as a representative value. If so, a third test was carried out and the closest values selected and checked as the value for the initial values.

7.6.2 EFFECT OF GGBS ONLY ON THE FREE SWELLING CHARACTERISTICS OF THE TEST SOIL

The effects of GGBS only on the swelling characteristics of the test soil, cured under CC1 and CC2 conditions, are illustrated in figures 7.37 and 7.38. It can be seen from these figures that the percentage of free swelling decreased with increasing GGBS content. For example, addition of 4% GGBS decreased the percentage of free swelling by 5% swelling, while the addition of 10% GGBS decreased it by 15% swelling under both of the two curing conditions, after 7 days. Increasing the curing periods produced further decreases in free swelling.

Under the same curing conditions and period, the percentage of free swelling decreases with an increase in the GGBS content. For example, the percentage of free swelling decreased from 30% to 15% with an increase in the GGBS to 6%, after 9 months while it decreased to 10% with an increase in GGBS content to 8% for the same curing conditions. For the same curing period and GGBS content, the percentage of free swelling remained the same or sometimes decreased with increasing curing temperature, especially for high GGBS content. For example, the percentage of free swelling decreased from 30% for the control sample to 10% with 8% GGBS after 9 months under CC1 conditions, while it decreased to 5% under CC2 conditions, at the same curing period and the same GGBS content, see table 7.8.

7.6.3 EFFECT OF CURING PERIODS ON THE SWELLING CHARACTERISTICS OF THE TEST SOIL TREATED WITH GGBS

The effects of curing periods on the free swelling characteristics, under CC1 and CC2 conditions, are illustrated in figures 7.37 to 7.38. It can be observed from these figures that the percentage of free swelling decreased with an increase in the curing period, keeping the GGBS content constant, under the same curing conditions. For example, the percentage of free swelling decreased from 30% for the control sample to 15% with an increase in GGBS to 10% after 28 days, under CC1 conditions, while the percentage of free swelling also decreased the same percentage with an.increase in the GGBS to 6% after 6 months. Also, the percentage of free swelling decreased to 15% with an increase in GGBS to 8% after 28 days, under CC1 conditions, while it decreased to 10% with an increase in the curing period to 9 months under the same curing conditions and GGBS content.

Table 7.8 Effect of GGBS alone on the free swelling of the test soil

Curing period GGBS%		Free swelling percentage					
		7 days	28 days	90 days	180 days	270 days	360 days
0	20°C	30	30	30	30	30	30
2		30	30	30	30	30	30
4		25	20	20	20	20	17
6		20	18	18	15	15	15
8		20	15	12	12	10	5
10		15	15	12	10	10	5
0	35°C	30	30	30	30	30	30
2		30	30	30	30	25	25
4		25	20	20	20	15	15
6		20	15	15	15	10	10
8		15	10	10	10	5	0
10		15	10	10	5	5	0

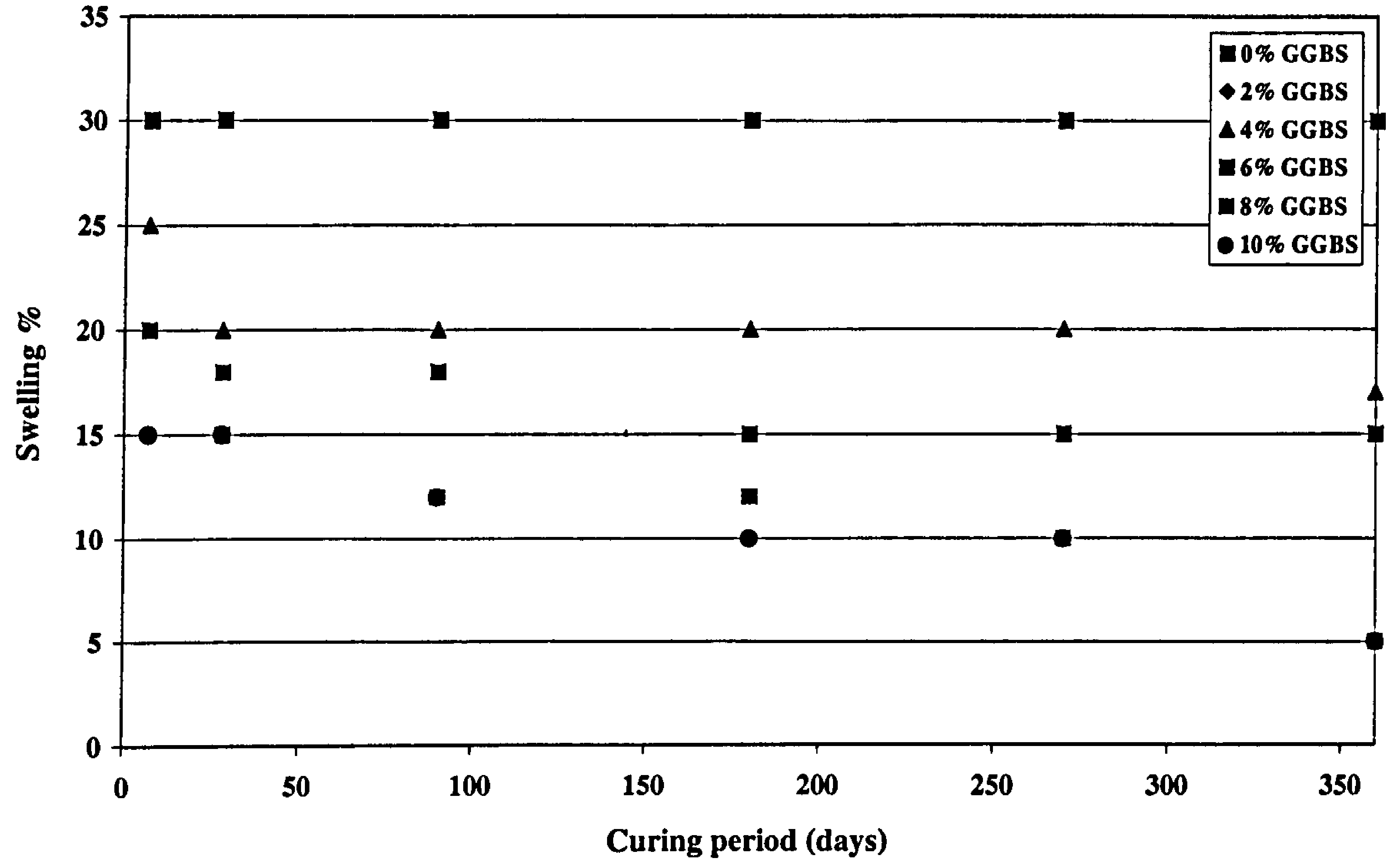

Figure 7.37 Effect of GGBS (%) on the free swelling of the test soil, cured under CC1 conditions, at different curing periods

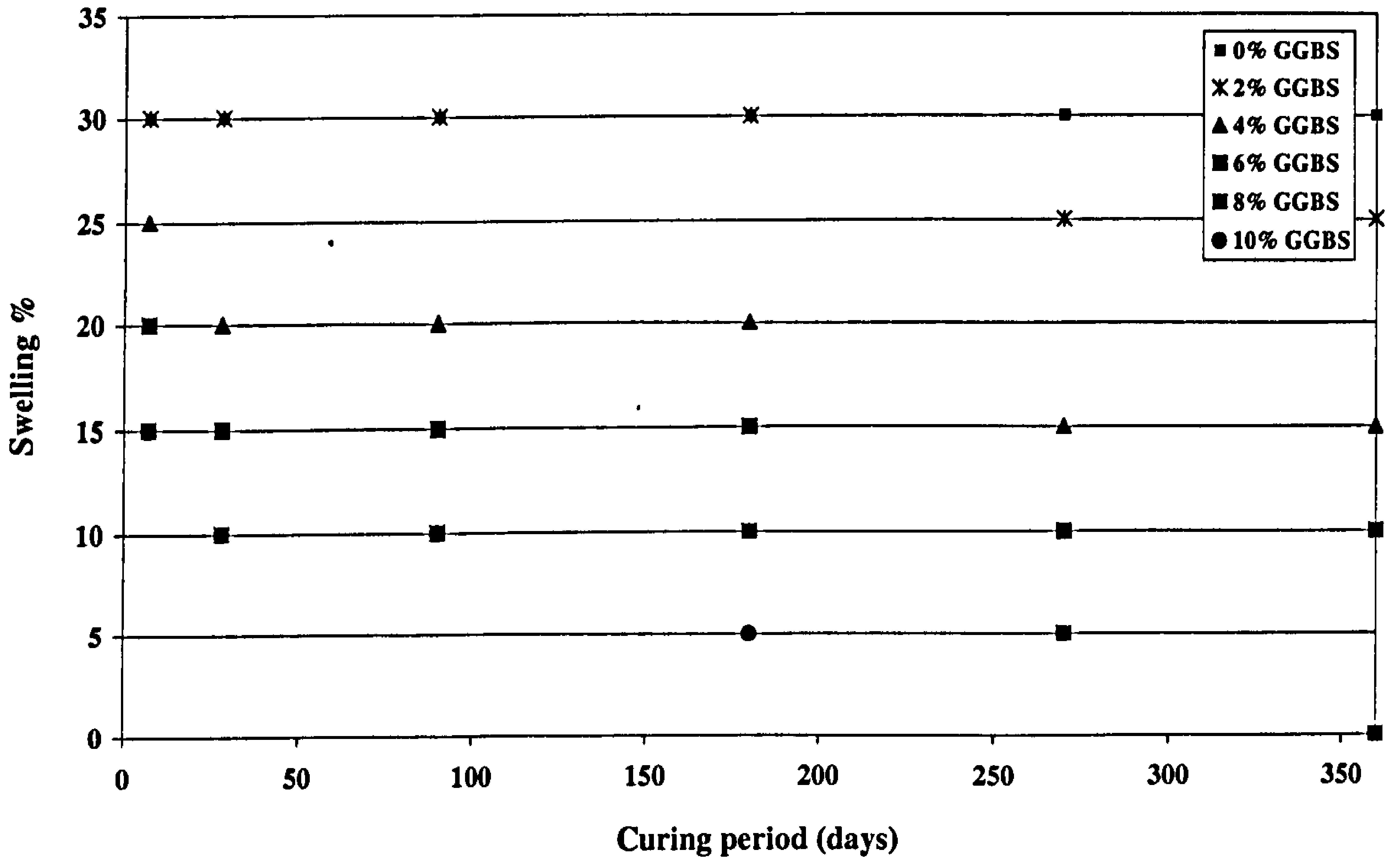

Figure 7.38 Effect of GGBS (%) on the free swelling of the test soil, cured under CC2 conditions, at different curing periods

7.6.4 EFFECT OF GGBS ACTIVATED BY LIME ON THE SWELLING CHARACTERISTICS OF THE TEST SOIL

Two curing conditions and two curing periods, 7 and 28 days, were employed in the study of the effect of GGBS activated by lime on the free swelling behaviour of the test soil. Figures 7.39 to 7.42 illustrate the effect of total binder, with different lime/GGBS ratios, on the percentage of free swelling, for samples cured under CC1 and CC2 conditions, after 7 and 28 days. It can be observed that percentage of free swelling decreases with increasing the total binder content for constant lime/GGBS ratio, with an increase in lime/GGBS ratio at constant binder content, with increasing the curing temperature keeping the binder content and lime/GGBS ratio constant, and with increasing the curing period keeping all other parameters constant. For example, increasing the binder content to 4% decreased the free swelling percent to 15% after 7 days under both curing conditions at a lime/GBS ratio of 0.25. Increasing curing period to 28 days decreased the free swelling percent to 10%, at both curing conditions, while it decreased to 5% only with an increase in lime/GGBS ratio to 0.43, under CC2 conditions, see figures 7.41 to 7.44 and table 7.9.

Increasing the total binder to 6% decreased the free swelling percent to 20% at a lime/GGBS ratio of 0.11, under both curing conditions after 7 days. Increasing the lime/GGBS ratio to 0.43 decreased the free swelling percent to 10% at the same binder content, under both curing conditions, after 7 days. Increasing curing periods to 28 days decreased the free swelling percent to 5% under both curing conditions at Keeping the binder content and the lime/GGBS ratio constant.

Table 7.9 Effect of GGBS and lime on the swelling characteristics of the test soil

Total binder (%)		Free swelling %			
		20°C		35°C	
		7 days	28 days	7 days	28 days
0	Lime/GGBS = 0	30	30	30	30
2		30	30	30	30
4		25	20	25	20
6		20	18	20	15
0	Lime/GGBS = 0.11	30	30	30	30
2		30	25	30	25
4		15	10	15	10
6		15	10	10	5
0	Lime/GGBS = 0.25	30	30	30	30
2		25	20	25	20
4		15	10	15	10
6		12	10	10	5
0	Lime/GGBS = 0.43	30	30	30	30
2		25	20	20	20
4		15	10	15	5
6		10	5	10	5

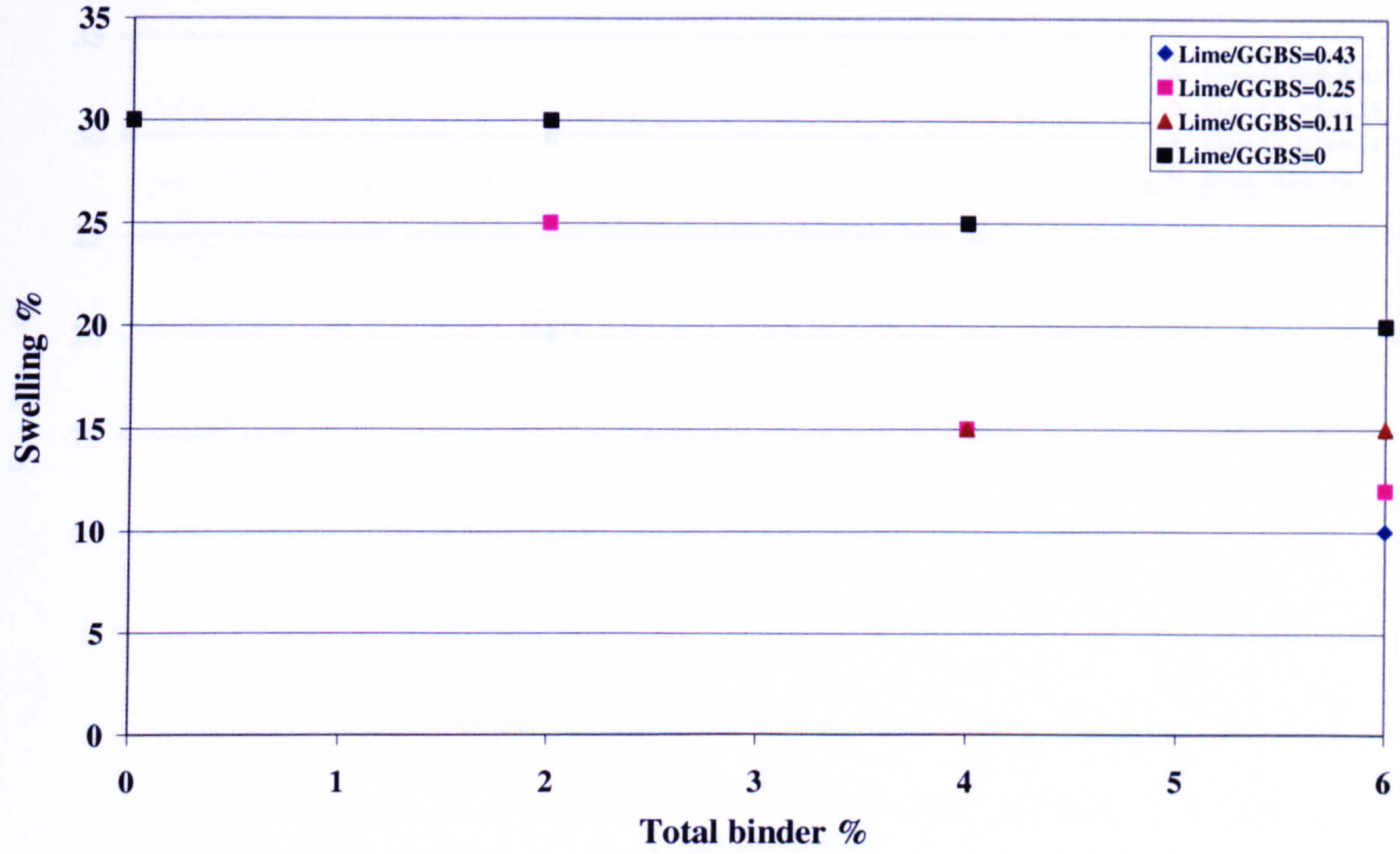

Figure 7.39 Effect of GGBS and lime on the free swelling percent of the test soil, cured under CC1 conditions, after 7 days

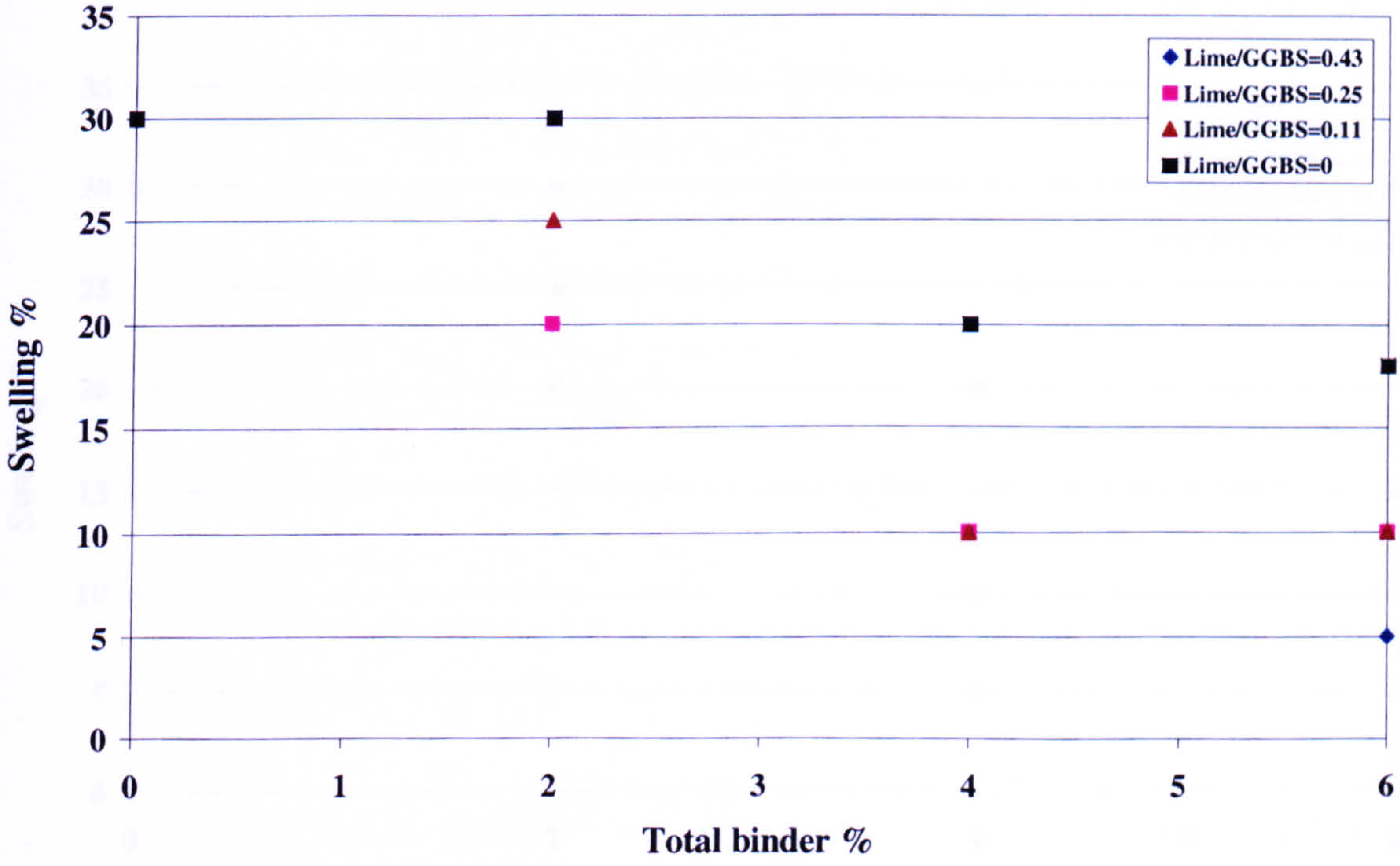

Figure 7.40 Effect of GGBS and lime on the free swelling of the test soil cured under CC1 conditions, after 28 days

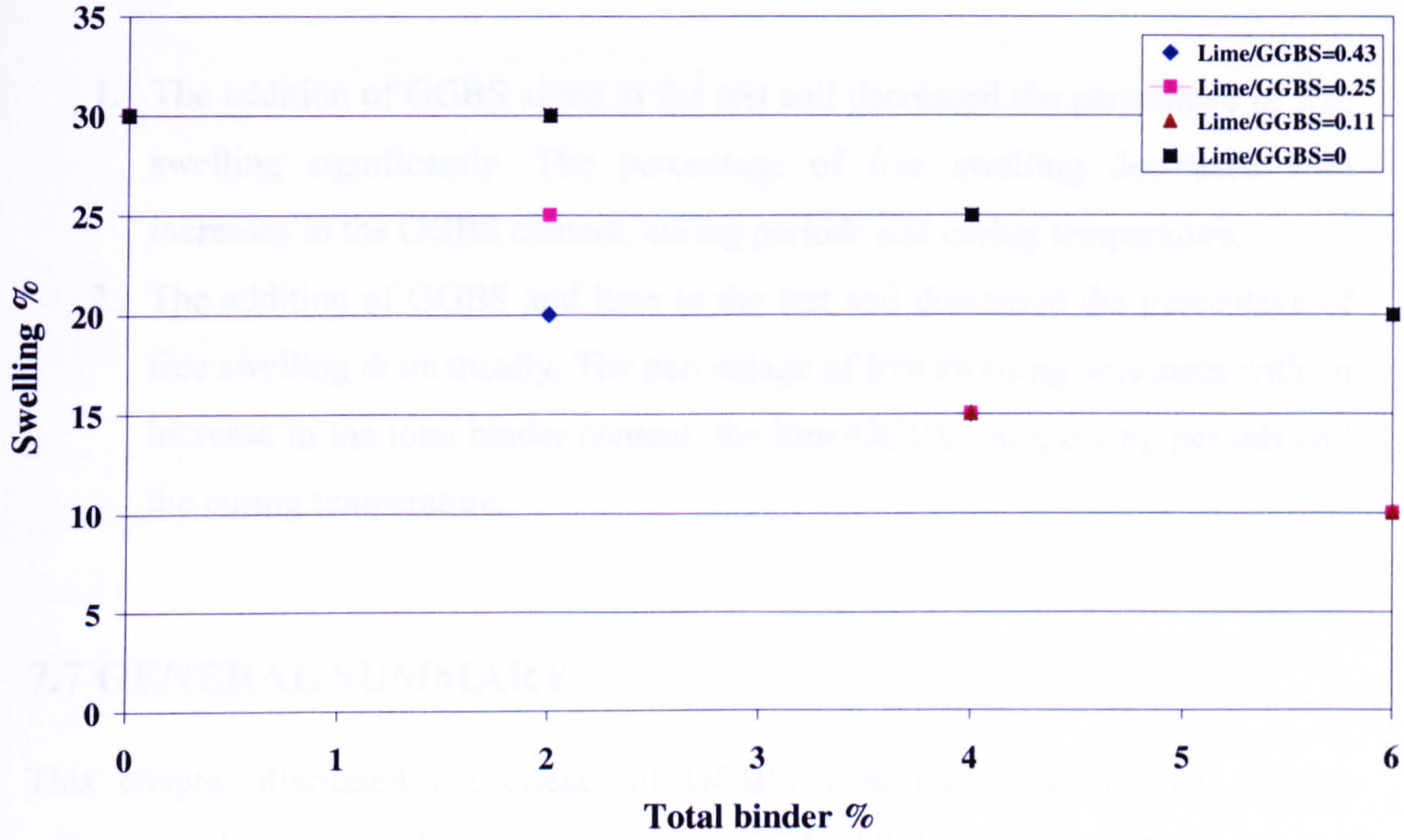

Figure 7.41 Effect of GGBS and lime on the free swelling of the test soil cured under CC2 conditions, after 7 days

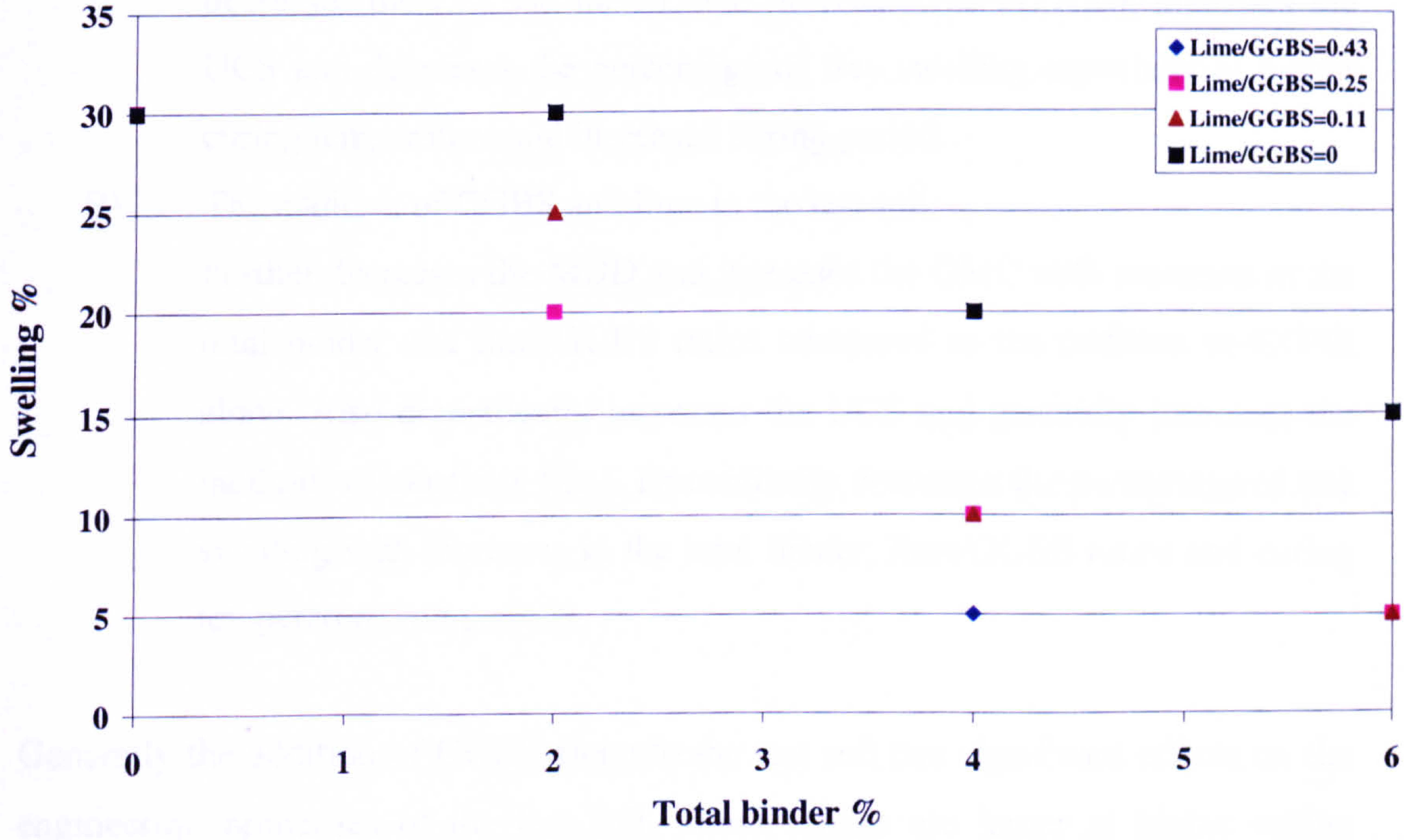

Figure 7.42 Effect of GGBS and lime on the free swelling of the test soil cured under CC2 conditions, after 28 days

7.6.5 SUMMARY

1. The addition of GGBS alone to the test soil decreased the percentage of free swelling significantly. The percentage of free swelling decreased with increases in the GGBS content, curing periods and curing temperature.
2. The addition of GGBS and lime to the test soil decreased the percentage of free swelling dramatically. The percentage of free swelling decreases with an increase in the total binder content, the lime/GGBS ratio, curing periods and the curing temperature.

7.7 GENERAL SUMMARY

This chapter discussed the effects of GGBS, with and without lime, on the compaction behaviour, plasticity characteristics, the UCS and the swelling potential of the test soil. The main conclusions can be summarised as:

A) The addition of GGBS alone to the test soil;
Slightly decreases the MDD and slightly increases the OMC. Slightly decreases the LL, and the PL and decreases the PI. Also, increases the UCS and decreases the percentage of free swelling especially at higher curing temperature and increased curing period.

B) The addition of GGBS and lime to the test soil:
Further decreases the MDD and increases the OMC with increases in the total binder and lime/GGBS ratios compared to the addition of GGBS alone. Also dramatically increases the UCS and generally increases the modulus of elasticity (E_{40}). Dramatically decreases the percentage of free swelling with increases in the total binder, lime/GGBS ratios and curing temperature and periods.

Generally the addition of GGBS alone to the test soil has significant effects on the engineering properties of the test soil. These effects are larger at higher curing temperature and with longer curing periods, implying physical and some chemical effects only at higher temperature and after a long curing period. The addition of GGBS and lime to the test soil have relatively much greater effects on the

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CHAPTER EIGHT

ANALYTICAL TEST RESULTS

8.1 INTRODUCTION

The previous chapter presented data which illustrate the effects of GGBS only and GGBS activated by lime on various physical characteristics and engineering properties of both test and pure clay test soils. Significant changes in compaction behaviour, unconfined compressive strength (UCS) and swelling behaviour were found in the test soil and significant changes were observed in the plasticity characteristics of pure clay test soils, with the addition of GGBS only. The changes in both test and pure clay soils with the addition of GGBS and lime were found to be higher than that caused by the addition of GGBS only.

Previous work on lime-clay systems indicated that it would be very difficult to study the reactions of small amounts of binder added to soils, due to the complex composition of soil minerals and the degree of varying crystallinity of the clay fraction. In many cases, it has been difficult to identify all of the mineral components of the natural soils by any method of analysis and this has led to difficulties in identifying the main reaction products (see section 5.6).

To understand the nature and the causes of the changes in behaviour and the reaction products associated with the stabilisation of clay soils by the addition of GGBS and lime, four analytical techniques were used on samples of pure clay test soil which represents the clay portion of the test soil. See appendix 5.1 for details of this soil.

The reaction products of clay-lime mixes are well documented in previous studies. The formation of calcium aluminate hydrate phases, C-A-H, calcium alumino silicate hydrate, C-A-S-H, and calcium carbonate, $CaCO_3$, have been reported by many authors as the common reaction products, (see section 2.8.5).

However, only limited work has been published on clay-lime-GGBS reaction products. Clay–lime–GGBS reactions produce C-A-S-H gel and hydrotalcite type phases containing magnesium (Meng *et al.*, 1998; Wild and Tasong, 1999). These works are discussed in some detail in chapter 3, "Review of slag stabilisation".

The objectives of this chapter are to present and analyse the test results of the analytical techniques used in this research programme. It is hoped that they may provide an explanation of the effect of GGBS and lime on the engineering properties of the test soil. As previously mentioned (see chapters 5 and 6) representative samples were prepared for the analysis, and the analytical techniques used in this investigation were X-ray diffraction (XRD), scanning electron microscopy (SEM), differential thermal analysis (DTA) and nuclear magnetic resonance (NMR) spectroscopy.

Hilt and Davidson (1961) studied clay–lime reactions, using X-ray diffraction techniques, they recommended mixing the samples at high moisture contents to get clear signals. They observed an increase in the height of the new peaks with an increase in lime content. However, others recommend using optimum moisture content when mixing the samples, (Wild *et al.*, 1998). Optimum moisture content was used in mixing the pure clay test soil in this investigation as the author believes that the change in moisture content may lead to a change in the reaction conditions and consequently the reaction products and may affect the properties of materials.

DTA and NMR tests were carried out on a large number of specimens, while XRD and SEM studies, due to their cost and limited availability in the School of Civil Engineering, were limited to a few carefully selected specimens. These specimens were the control specimen and one of the mixes that had a large amount of binder to detect most of the reaction products, to give indications of the nature (and possibly the sites) of the reactions and reaction products.

8.2 X-RAY DIFFRACTION (XRD)

8.2.1 INTRODUCTION

Two specimens were analysed using XRD; the control specimen of pure clay soil (55% of montmorillonite and 45% of kaolinite) and one of the specimens which produced larger changes in strength [(control specimen + 27 % binder (20%lime)]. Both sets of specimens were cured for 6 months under CC2 conditions. The preparation of the specimens and the XRD equipment and operation are described in section 6.4.2

8.2.2 X-RAY DIFFRACTION RESULTS

The X- ray diffraction diagrams for the raw materials (Montmorillonite, lime and GGBS) and control and hydrated specimens are shown in figure 8.1. The diffraction patterns were obtained between Bragg angles of 5° and 65°. XRD traces of montmorillonite showed four main broad peaks at 12.86 Å[1] (6.875°), 4.5 Å (19.725°), 2.55 Å (35.175°) and 1.48 Å (62.775°), while hydrated lime traces showed peaks at 4.89 Å (18.125°), 3.11 Å (28.775°), 2.62 Å (34.225°), 1.924 Å (47.225°), 1.79 Å (50.875°) and 1.68 Å (54.475°). GGBS traces showed a broad band at 3.34 Å (26.675°). The test soil traces showed strong kaolinite peaks at 7.152 Å (12.375°) and 3.58 Å (24.825°) and medium kaolinite peaks at 2.33 Å (38.475°) and 2.29 Å (39.275°). The XRD traces of the hydrated specimen showed new peaks at 7.69 Å (11.50°), 7.85 Å (11.275°), 3.07 Å (29.075°), 2.45 Å (36.68°), 2.86 Å (31.27°), 2.88 Å (31.05°) and 3.035 Å (29.425°), while medium peaks were observed at 9.80 Å (9.025°) and 6.52 Å (13.575°). Also, weak peaks were also observed at 5.48 Å (16.175°) and 2.824 Å (31.675°).

After 6 months curing, the montmorillonite peak at 1.48 Å (62.775°) completely disappeared while the peaks intensities at 12.86 Å (6.875°), 4.5 Å (19.725°) were considerably reduced. The intensities of the kaolinite peaks at 7.152 Å (12.375°), 3.58 Å (24.825°), 2.33 Å (38.475°) and 2.29 Å (39.275°) were considerably reduced

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[1] The figures between brackets following the d spacing values are the 2θ values

in the hydrated specimen. The calcium hydroxide peak at 3.11 Å (28.775°) was reduced while the other peaks at 4.89 Å (18.125°), 2.62 Å (34.225°), 1.924 Å (47.225°), 1.79 Å (50.875°) and 1.68 Å (54.475°) completely disappeared indicating that nearly all the lime content was consumed at this stage of reaction. Also, the GGBS peak at 3.34 Å (26.675°) completely disappeared which indicates that most of GGBS was consumed.

The reaction products can be classified into eight groups. Group a shows the formation of the hydrotalcite type phase, which contains the strong peak at 7.69 Å (11.50°) (Wilding and McHugh, 1986). Group b shows the formation of $CaCO_3$, which contains the strong peak at 3.035 Å (29.425°). Group c shows the formation of semi-crystallized C-S-H gel, which contains the peak at 3.07 Å (29.075°), while group d shows the formation of calcium aluminate hydrate (C_4AH_{13}), which contains the peak at 7.85 Å (11.275°) (Taylor, 1964; Turriziani, 1964; Mostafa *et al.*, 2001 a and b). Group e shows the formation of α_1 C_4AH_{19} which contains the peak at 2.45 Å (36.68°) and 2.88 Å (31.05°) (Taylor, 1964). Group f could be due to the formation of C-S-H II, which contains the peak at 9.80 Å (9.025°) (Taylor, 1964). Group g which contains the peaks at 6.52 Å (13.575°), 5.48 Å (16.175°) and 2.824 Å (31.675°) could be due to the formation of a member of the hydrogarnet solid solution series (C_3AH_6-C_3AS_3). Also, group h which contains the peaks at 7.6 Å (5.82°), 3.8 Å (11.70°), 2.86 Å (15.637°) and 2.49 Å (18.03°) could be due to the formation of calcium aluminate crbonate hydrate $C_3AC\overline{C}H_{12}$.

8.2.3 SUMMARY

XRD results showed the presence of the crystalline calcium aluminate hydrates (C_4AH_{13} and α_1 C_4AH_{19}) together with semi-crystalline C-S-H gel. Hydrotalcite type phase and carbonation are also present in the hydrated specimen. The results also showed that large amounts of lime and GGBS were consumed while large amounts of kaolinite and montmorillonite clays were transformed to other phases during the reaction.

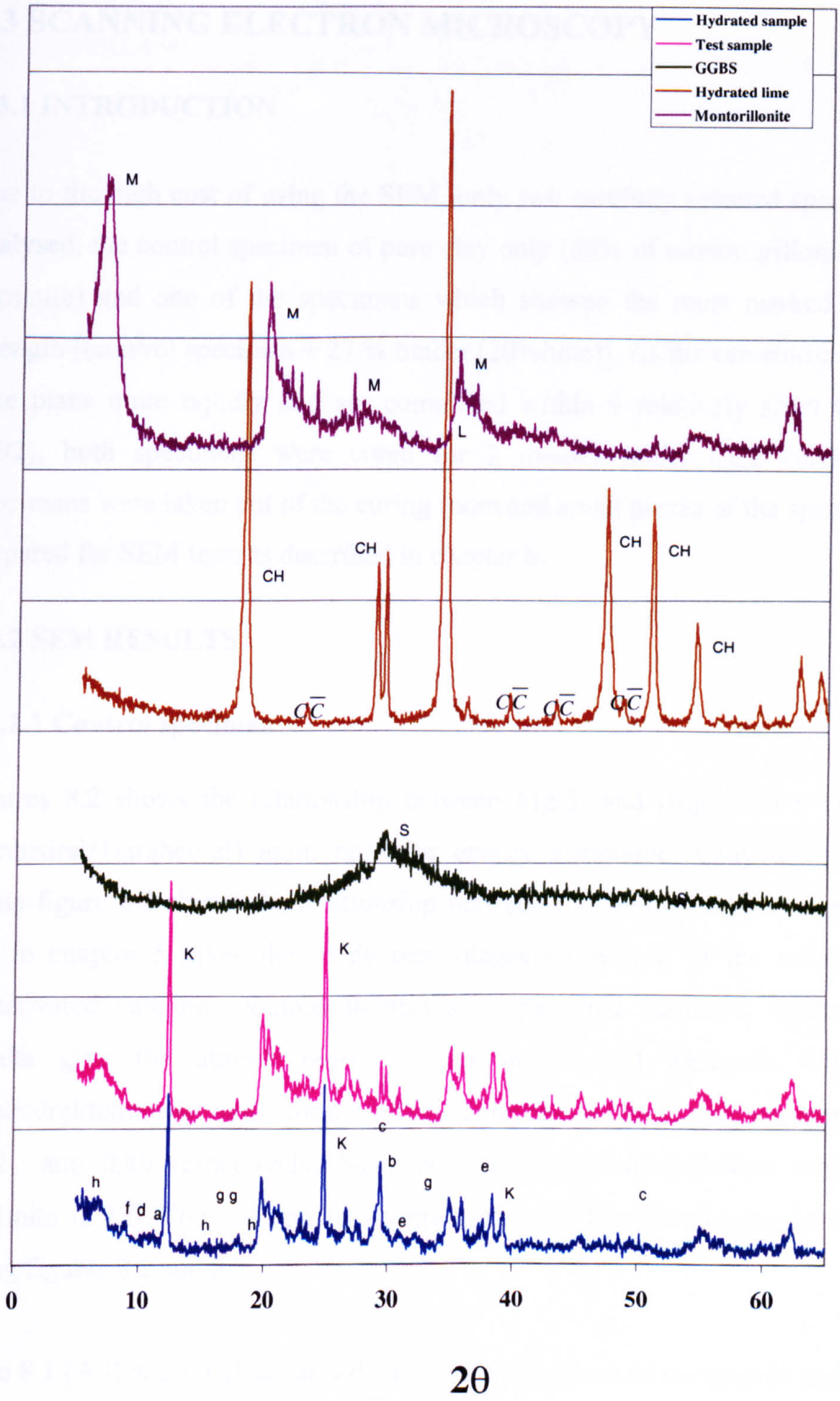


Figure 8.1 X- ray diffraction trace of the BB1, lime, GGBS, pure clay test soil and the hydrated sample

K: Kaolinite
M: Montmorillonite
S: GGBS
CH: Hydrated lime
$C\overline{C}$: Calcium carbonate

8.3 SCANNING ELECTRON MICROSCOPY

8.3.1 INTRODUCTION

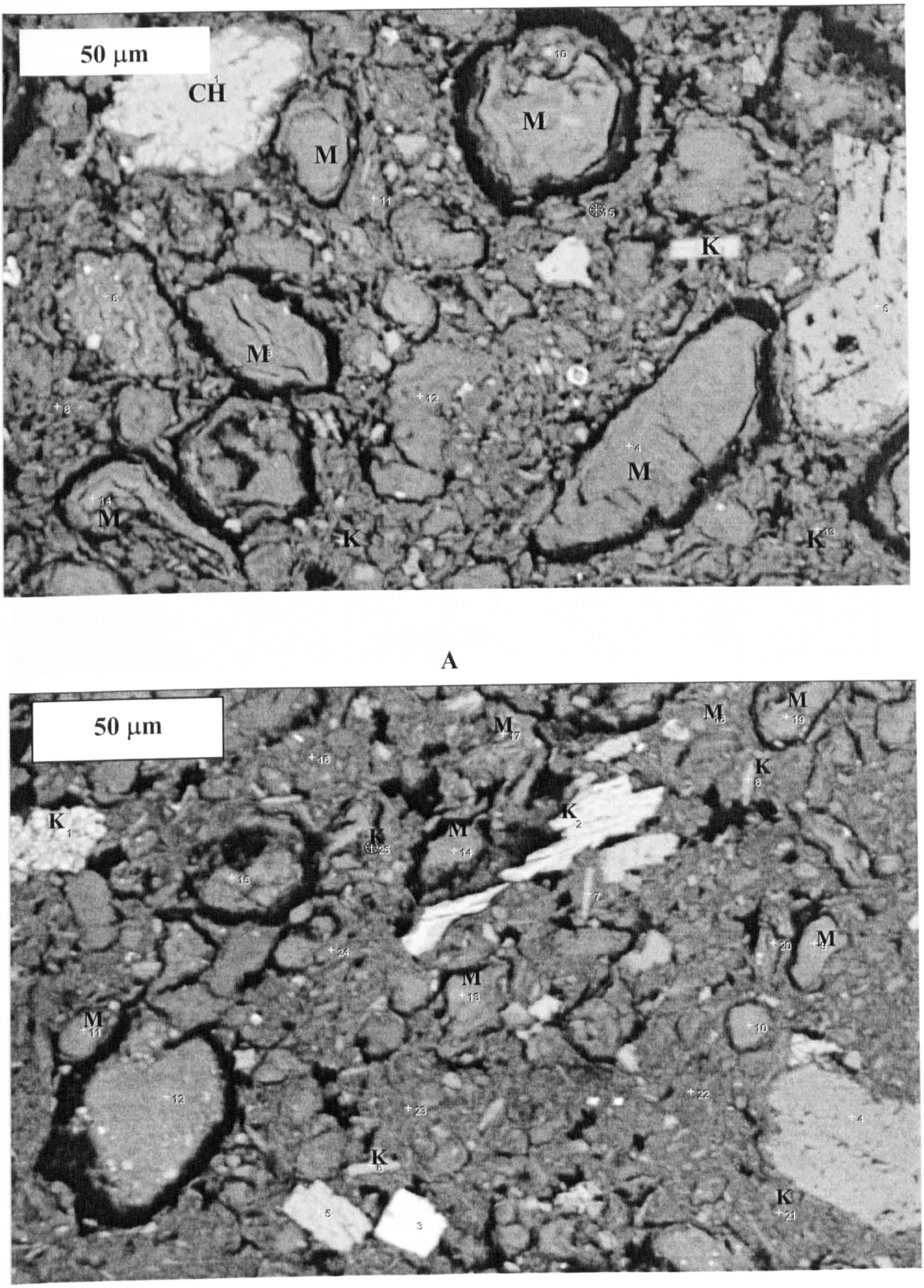
Due to the high cost of using the SEM, only two carefully selected specimens were analysed, the control specimen of pure clay only (55% of montmorillonite + 45% of kaolinite) and one of the specimens which showed the most marked changes in strength [(control specimen + 27 % binder (20%lime)]. As the cementitious reactions take place quite rapidly and are completed within a relatively short time (Abdi, 1992), both specimens were cured for 2 months under CC2 conditions. The specimens were taken out of the curing room and small pieces of the specimens were prepared for SEM tests as described in chapter 6.

8.3.2 SEM RESULTS

8.3.2.1 Control specimen

Figures 8.2 shows the relationship between Mg/Si and (Mg+Al+Fe+Ti+Mn) / Si (Octhedral/Tetrahedral) atom ratio for energy dispersive X-ray analysis (EDX), while figure 8.3 shows the relationship between Si/Ca and Si/Al atom ratio. Table 5.3 in chapter 5 gives the oxide percentages by weight of the montmorillonite (inactivated calcium montmorillonite) and speswhite kaolinite, while the SEM results give the atomic ratios of the investigated elements. The average octahedral/tetrahedrl ratios for montmorillonite and kaolinite used are approximately 0.50 and 0.80 respectively, however, the octahedral/tetrahedral ratio for pure kaolinite is 1.0. The location of kaolinite and montmorillonite can be determined using figures 8.2 and 8.3.

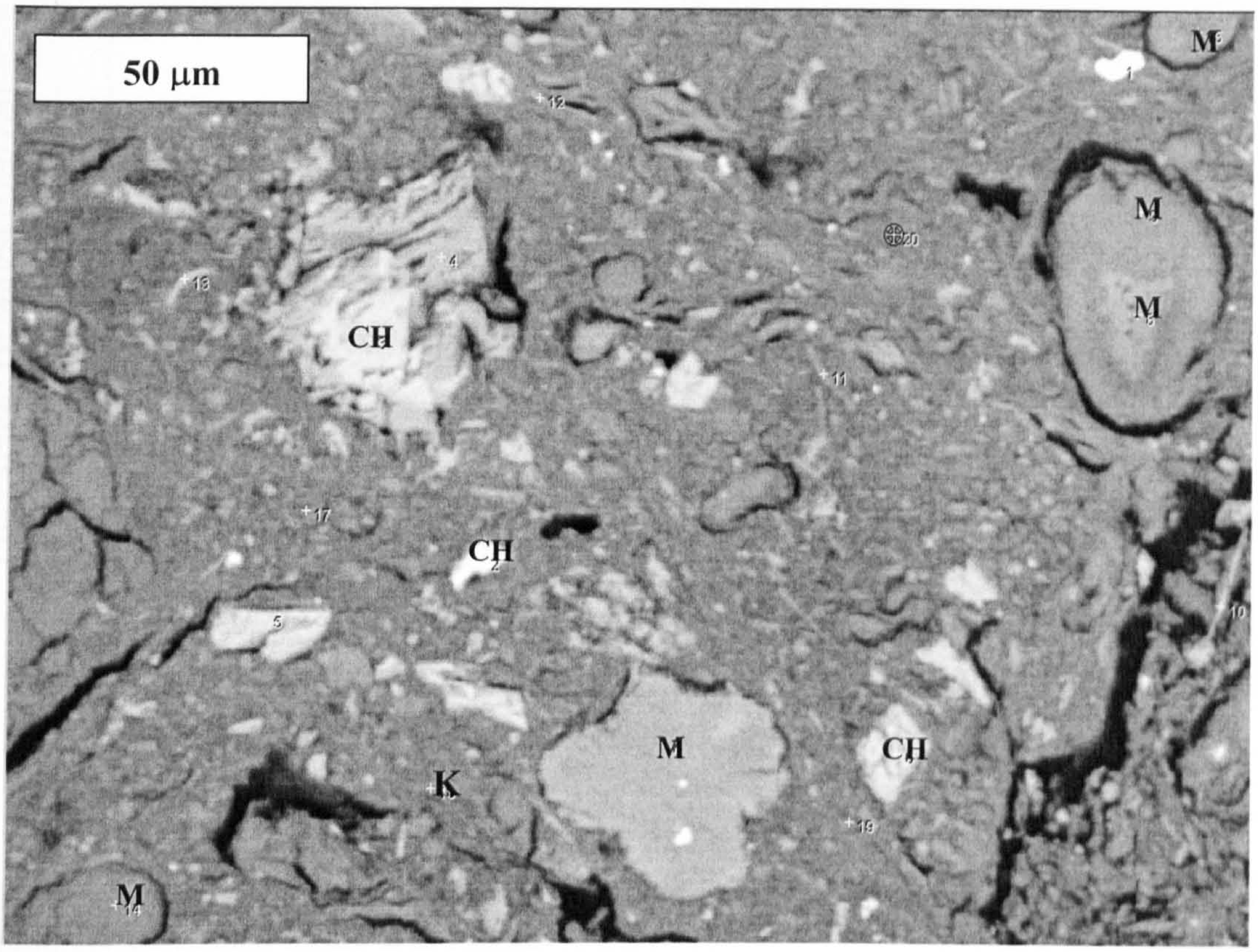
Plate 8.1 (A,B and C), illustrates the general appearance of the control sample of the pure clay test soil. As the pure clay test soil is mainly composed of montmorillonite and kaolinite with very small amounts of illite, mica and chlorite, and using the data given in figures 8.2 and 8.3, the montmorillonite flakes are visible in large areas, see plate 8.1 (A, B and C).



A

B

Plate 8.1 (A,B and C) SEM micrograph showing the pure clay test soil



C

Plate 8.1continue.

Kaolinite stacked plates which make up the granular aggregates are also visible at this magnification in large areas in plate 8.1 (A, B and C). Calcium hydroxide is also present in minor amounts, see plate 8.1 (a, b and c). The location of calcium hydroxide can be seen in figure 8.3. The main oxides found in the control specimen given by the SEM are silicon, followed by aluminium, calcium, titanum, iron and magnesium which in agreement with the X ray flouresence results.

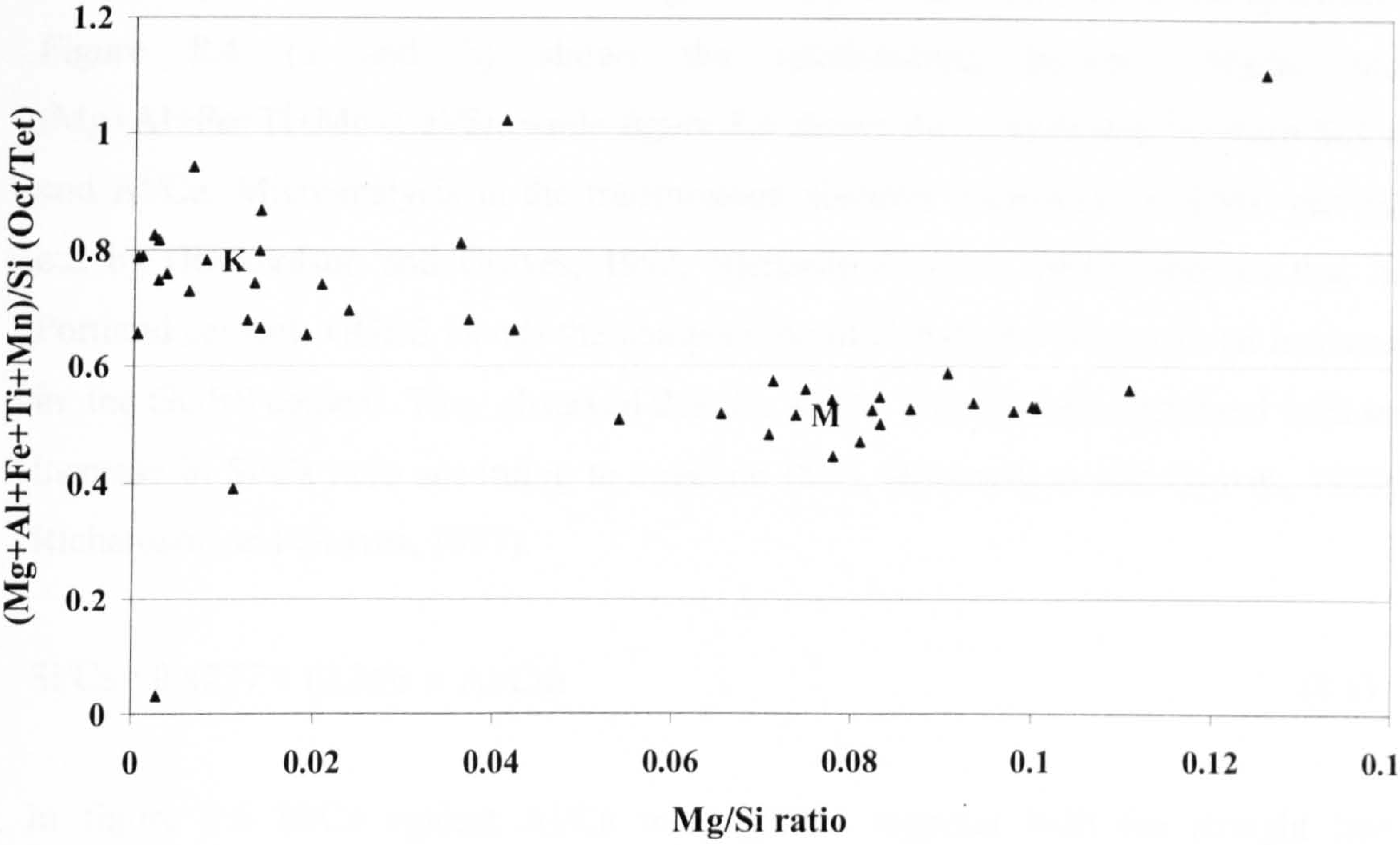


Figure 8.2 Mg/Si against (Mg+Al+Fe+Ti+Mn) / Si (Octhedral/Tetrahedral) atom ratio for EDX data of the area in plate 8.1 (a,b and c)

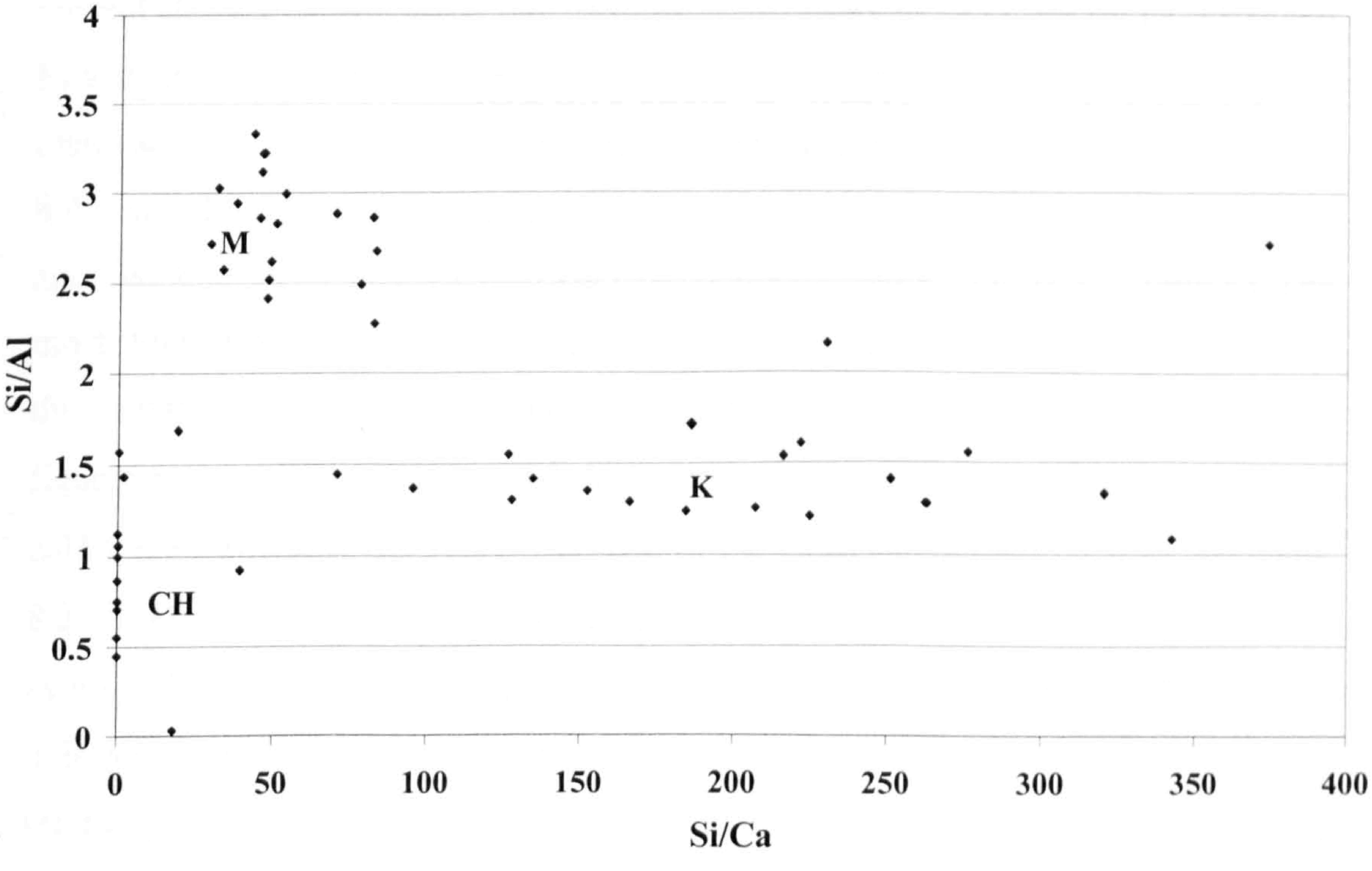


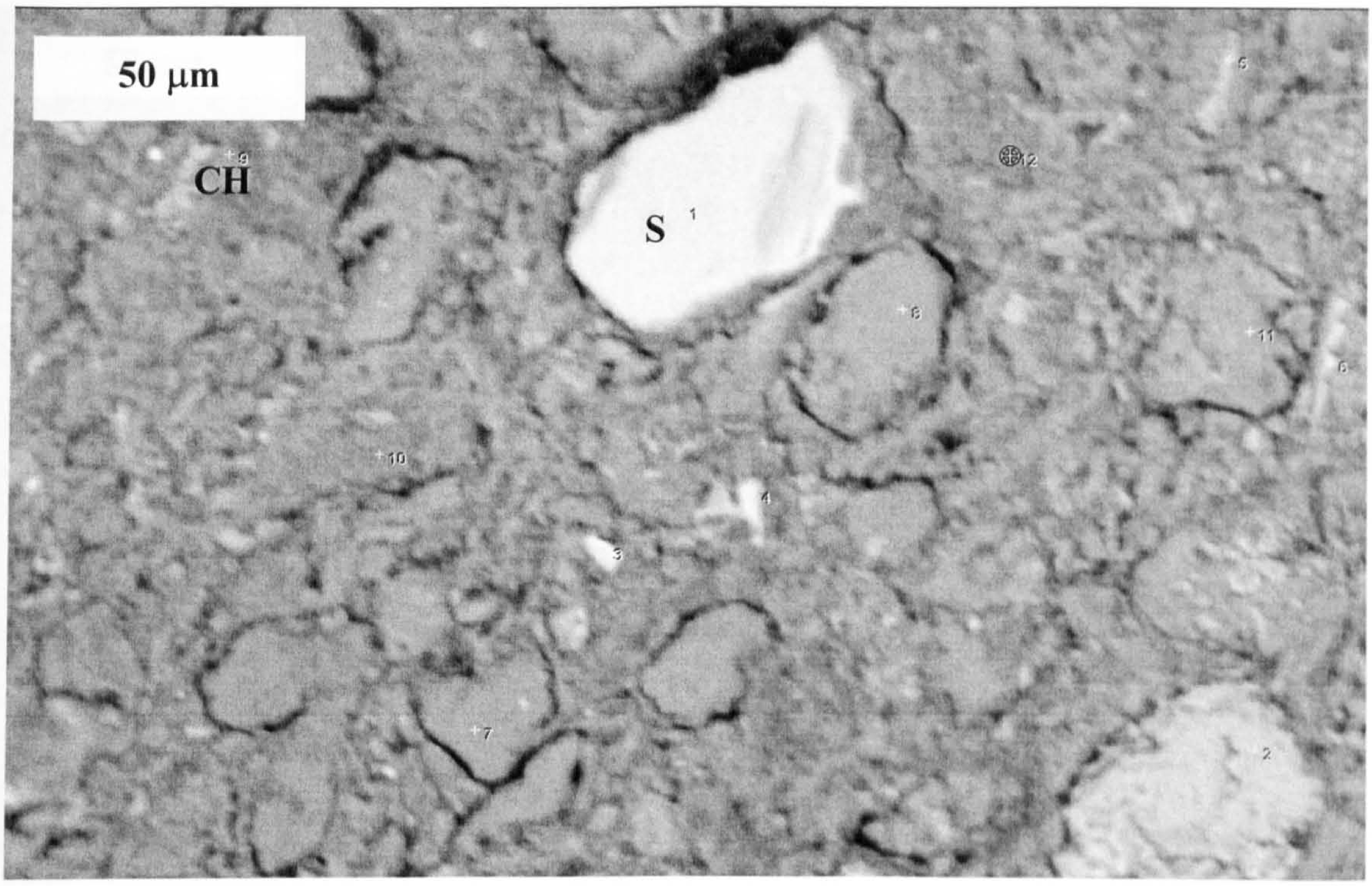
Figure 8.3 Si/Ca against Si/Al atom ratio for EDX data of the area in plate 8.1 (a, b and c)

Plate 8.2 (a, b, c, d, e and f) shows the general appearance of the stabilised specimen. Figure 8.4 (a and b) shows the relationships between Mg/Si and (Mg+Al+Fe+Ti+Mn+Ca)/Si, while figure 8.6 shows the relationship between Si/Ca and Al/Ca. Microanalysis in the transmission electron microscope (TEM) carried out by (Richardson and Groves, 1992; Richardson *et al.,* 1990), showed that in Portland cement- GGBS blends the composition of C-A-S-H varies with an increase in the GGBS content. They observed that the Al/Ca ratio linearly increased with an increase in Si/Ca ratio according to equation (8.1), (Richardson and Groves, 1993; Richardson and Groves, 1997).

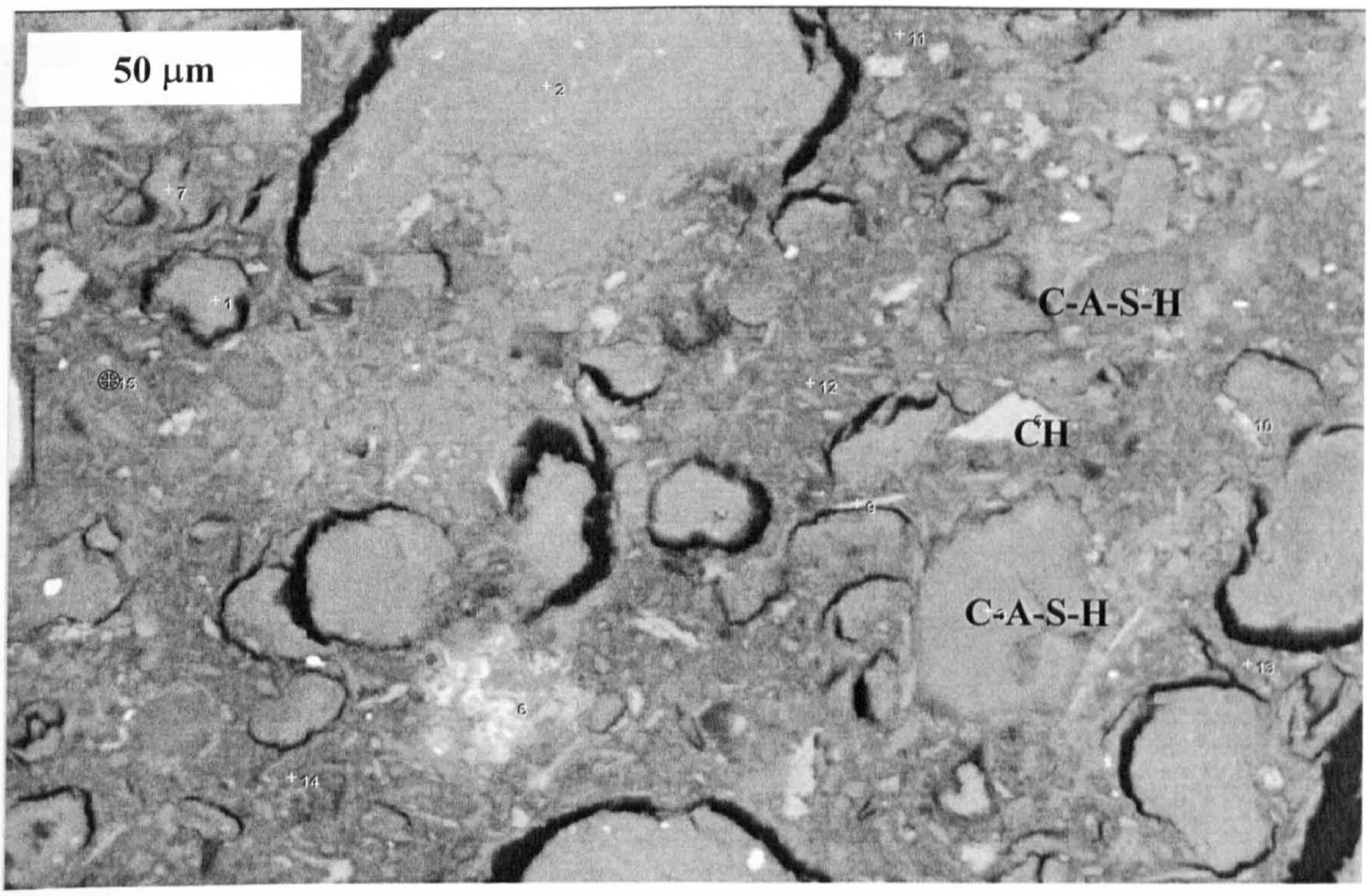
$$\text{Si/Ca} = 0.4277 + (2.366 \times \text{Al/Ca}) \qquad (8.1)$$

In figure 8.6 Si/Ca against Al/Ca were plotted together with the straight line representing the above equation. Although the above equation was based on different system, the equation could be used due to the expected similarity between the C-A-S-H in (Richardson *et al.*, 1990) system and the C-A-S-H formed in the current study. Few unreacted slag particles were observed, see plate 8.2 (A, C, D and F) and figures 8.4 a and 8.6. Hydrotalcite type phase containing magnesium (H) was identified as a main reaction product in GGBS hydration, see plate 8.2 (f) and figures 8.4 b and 8.6. Figure 8.5 gives the locations of calcium hydroxide (CH) in both test and hydrated specimens, see plates 8.1 (A, B and C), and 8.2 (A, B, C, D and F). The most interesting finding is that the C-A-S-H covered most of the image especially the montmorillonite particles. The C-A-S-H was visually identifed clearly using figures 8.4 a and 8.6, and plate 8.2. In fact two phases of aluminum substituted C-A-S-H were formed, firstly (C-A-S-H) due to the GGBS hydration by lime, see plate 8.2 (B, D, E and F). C-A-S-H in these areas has an average Ca/Si ratio ≈ 1.43, an average Al/Ca ratio ≈ 0.16, Ca/(Si +Al) ≈ 1.167 and an average Al/Si ≈ 0.20 which is in good agreement with the same ratio derived from the NMR results for similar mixture, see section 8.5. The second phase of C-A-S-H marked by a letter O, is due to clay-lime reaction, this phase is mixed with clay particles, it has a low calcium content, and a lower average Ca/Si ratio ≈ 0.41, an average Al/Si ratio ≈ 0.33 and Ca/(Si +Al) ≈ 0.29, see plate 8.2 (f).

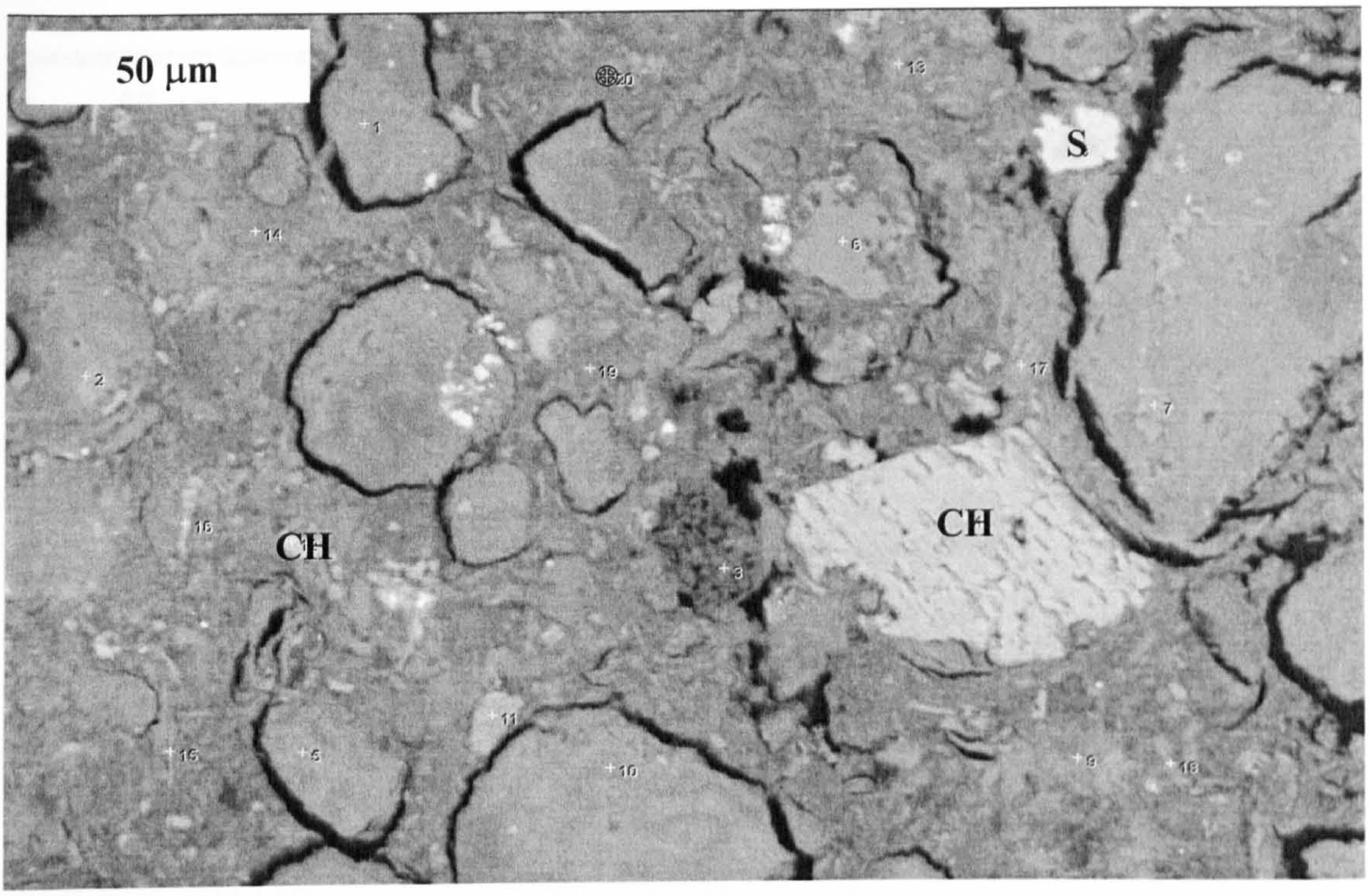
Plate 8.2 shows that generally the formed aluminum substituted C-A-S-H (C-A-S-H) spread and covered the clay particles in large areas of the specimen. Also, the areas of resin were reduced which indicates the porosity of the hydrated specimen was enhanced due to the formation of reaction products (i.e. C-A-S-H) which are considered to be stable and they were not largely affected by freeze drying.


50 µm
5
9
CH
12
S
1
8
11
6
10
4
3
7
2

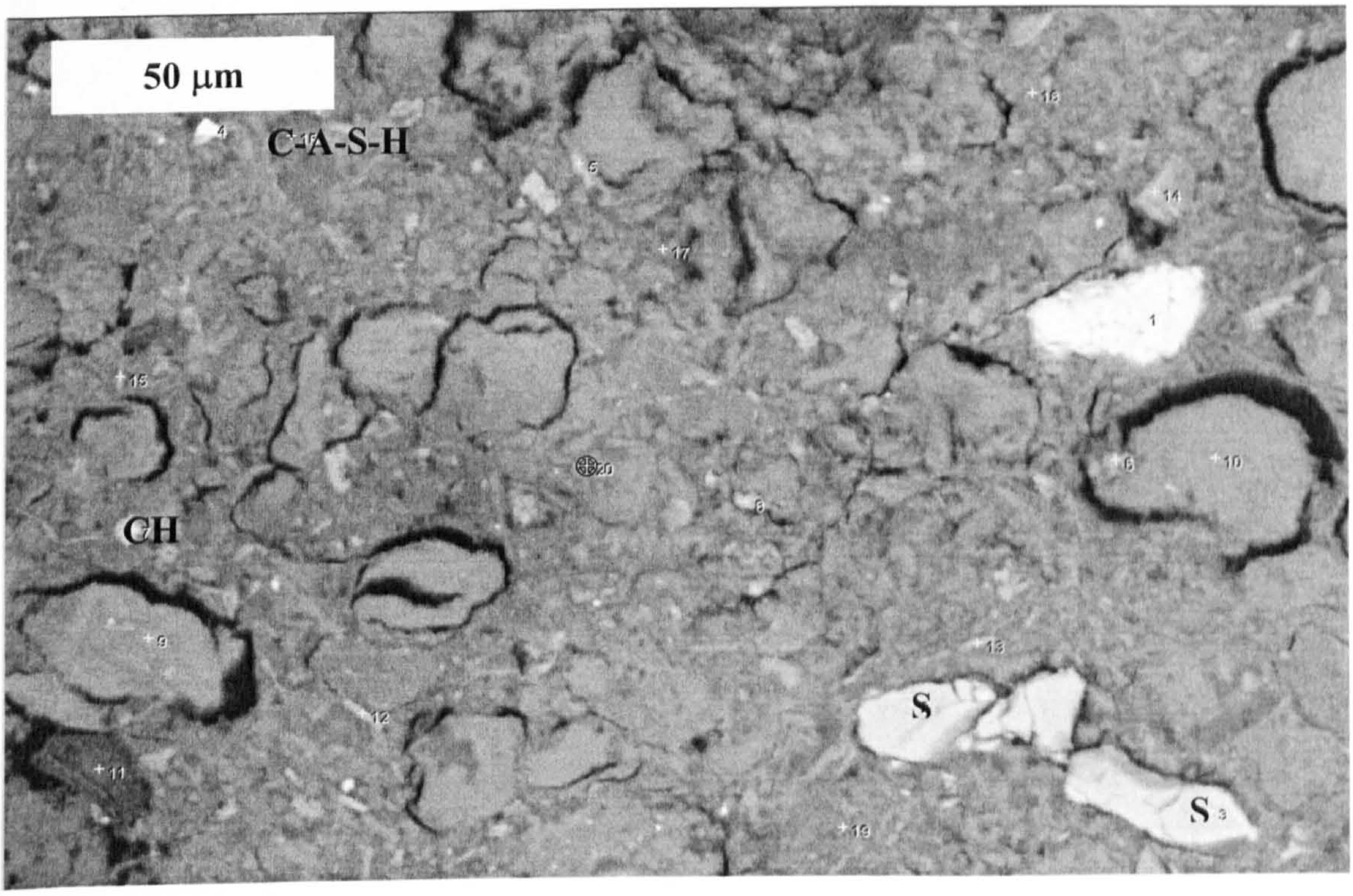
A


50 µm
11
2
7
1
C-A-S-H
15
12
CH
10
9
C-A-S-H
13
6
14

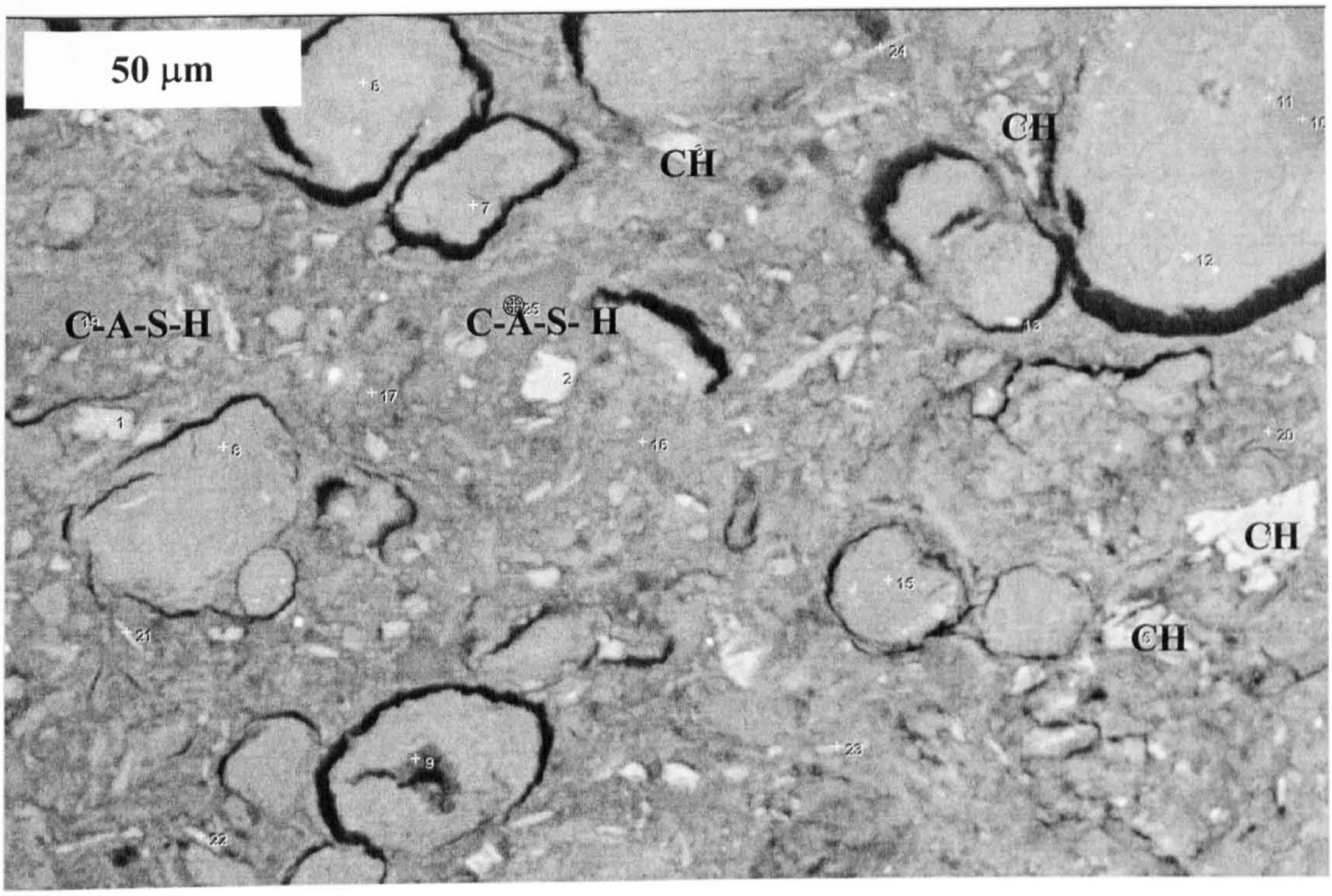
B



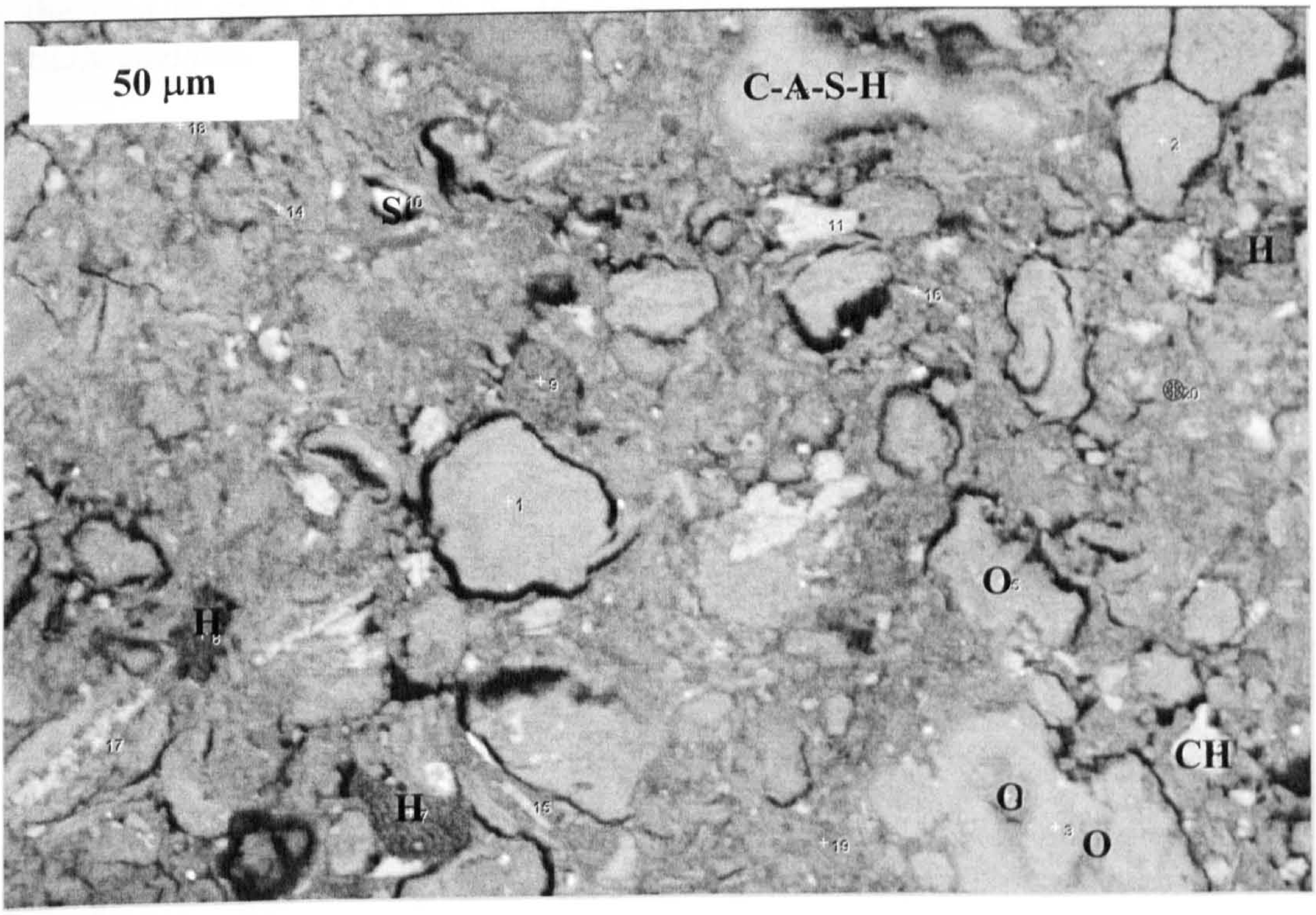
C



D



E



F

Plate 8.2 (A, B, C, D, E and F) SEM micrograph showing the hydrated pure clay test specimen

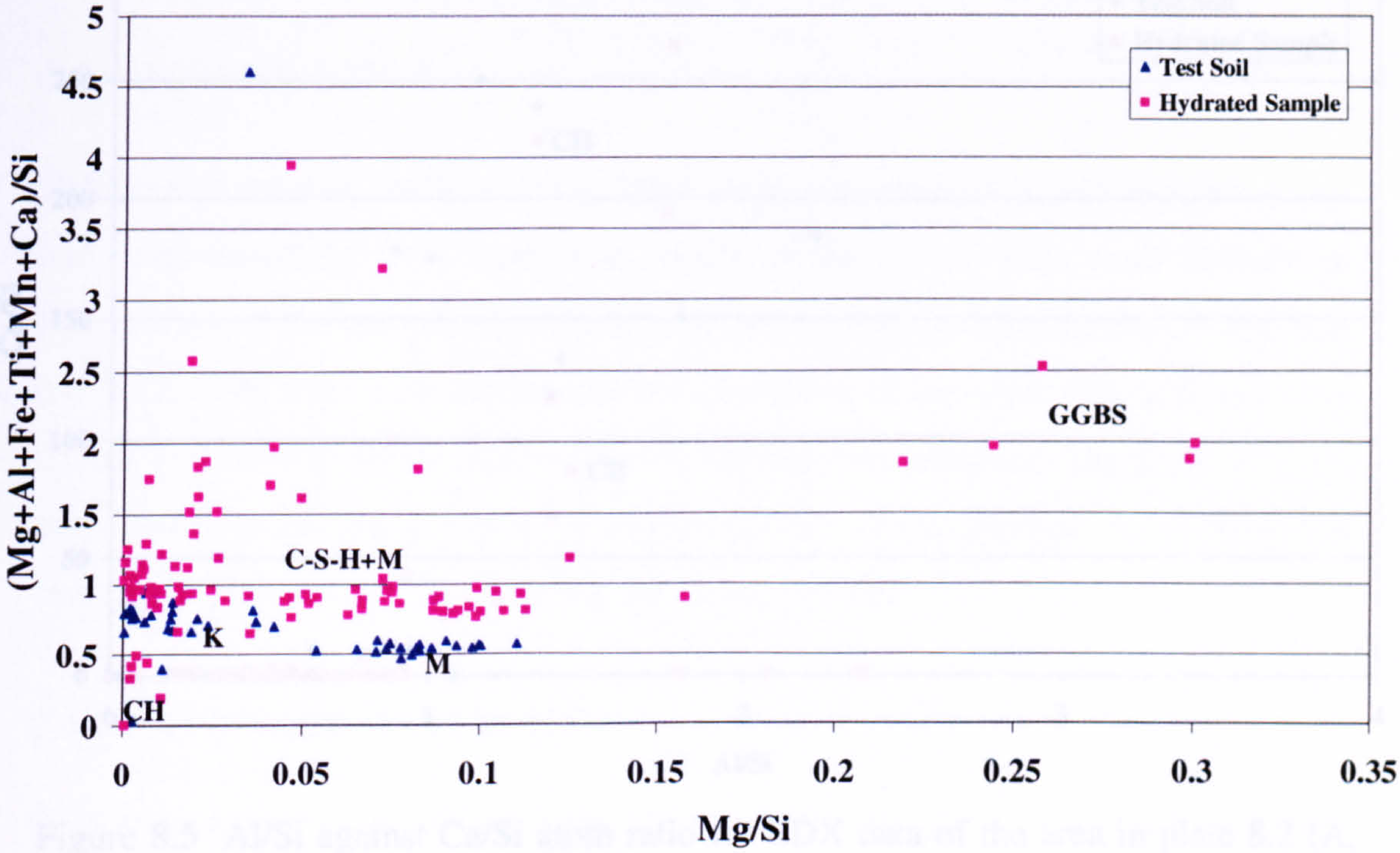


Figure 8.4 a Mg/Si against (Mg+Al+Fe+Ti+Mn+Ca)/Si atom ratio for EDX data of the area in plate 8.2 (A, B, C, D, E and F), hydrated specimen, low Mg/Si, from EDX analysis.

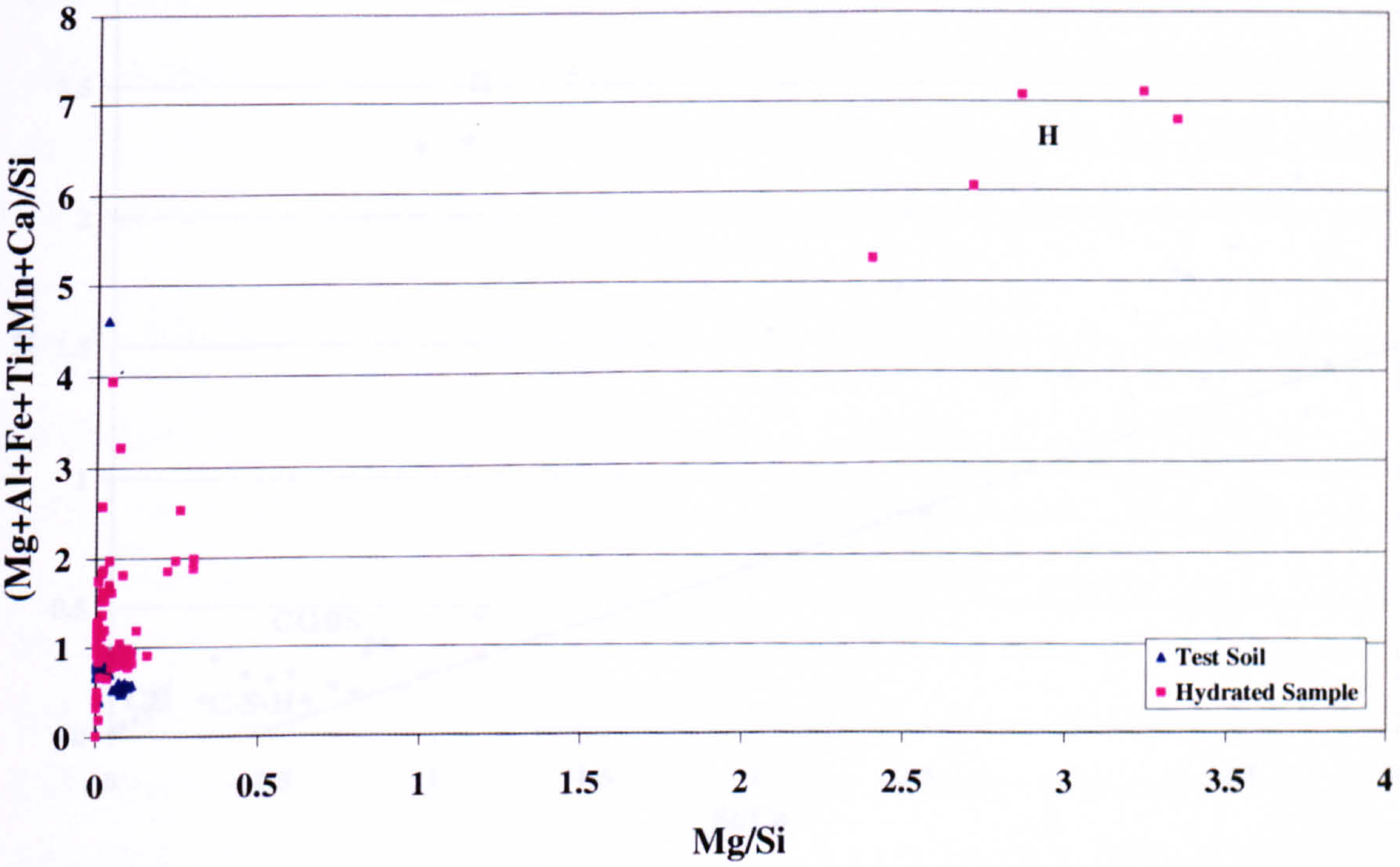


Figure 8.4 b Mg/Si against (Mg+Al+Fe+Ti+Mn+Ca) /Si atom ratio for EDX data of the area in plate 8.2 (A, B, C, D, E and F), hydrated specimen, intermediate Mg/Si ratio, from EDX analyis.

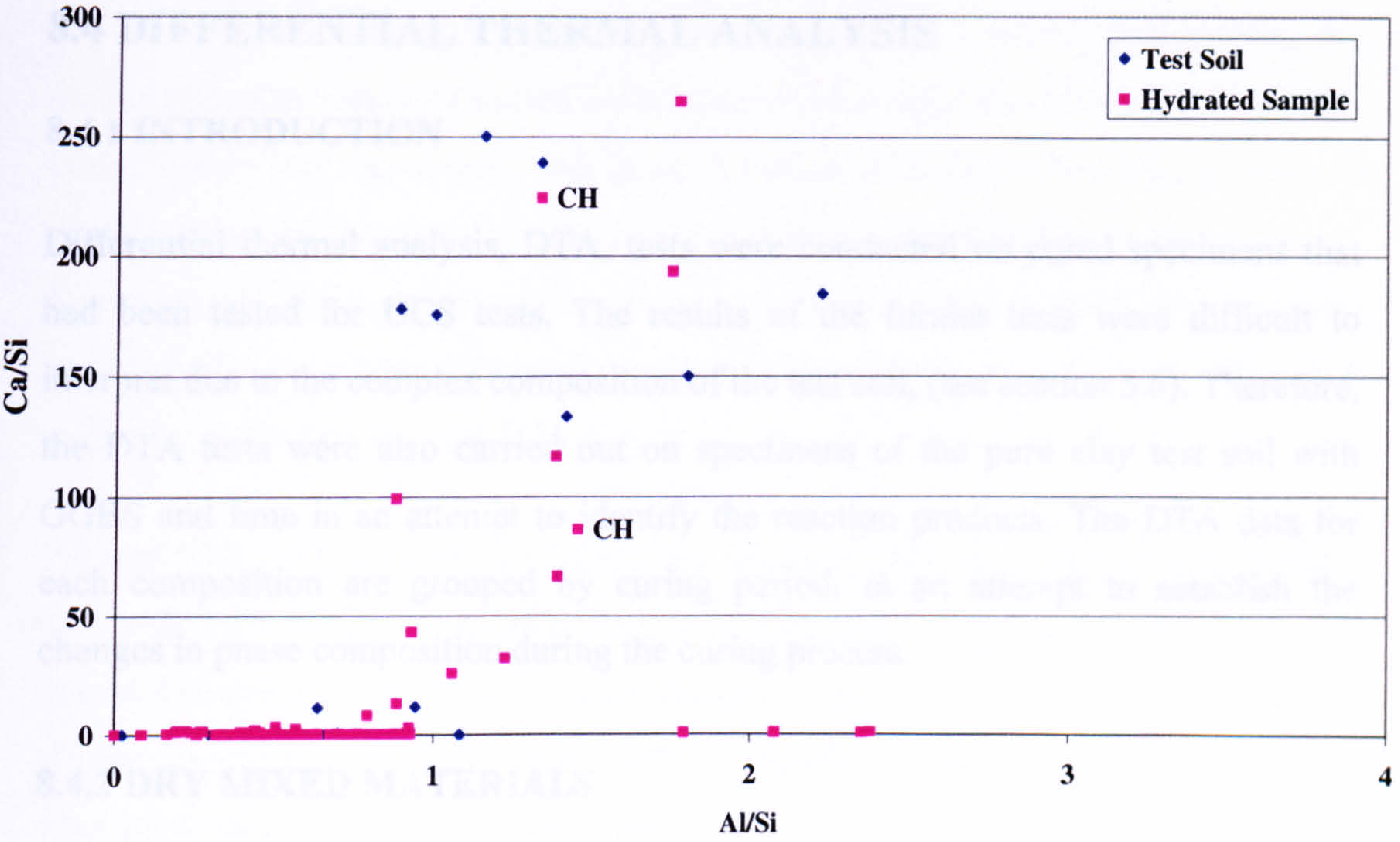


Figure 8.5 Al/Si against Ca/Si atom ratio for EDX data of the area in plate 8.2 (A, B, C, D, E and F), hydrated specimen

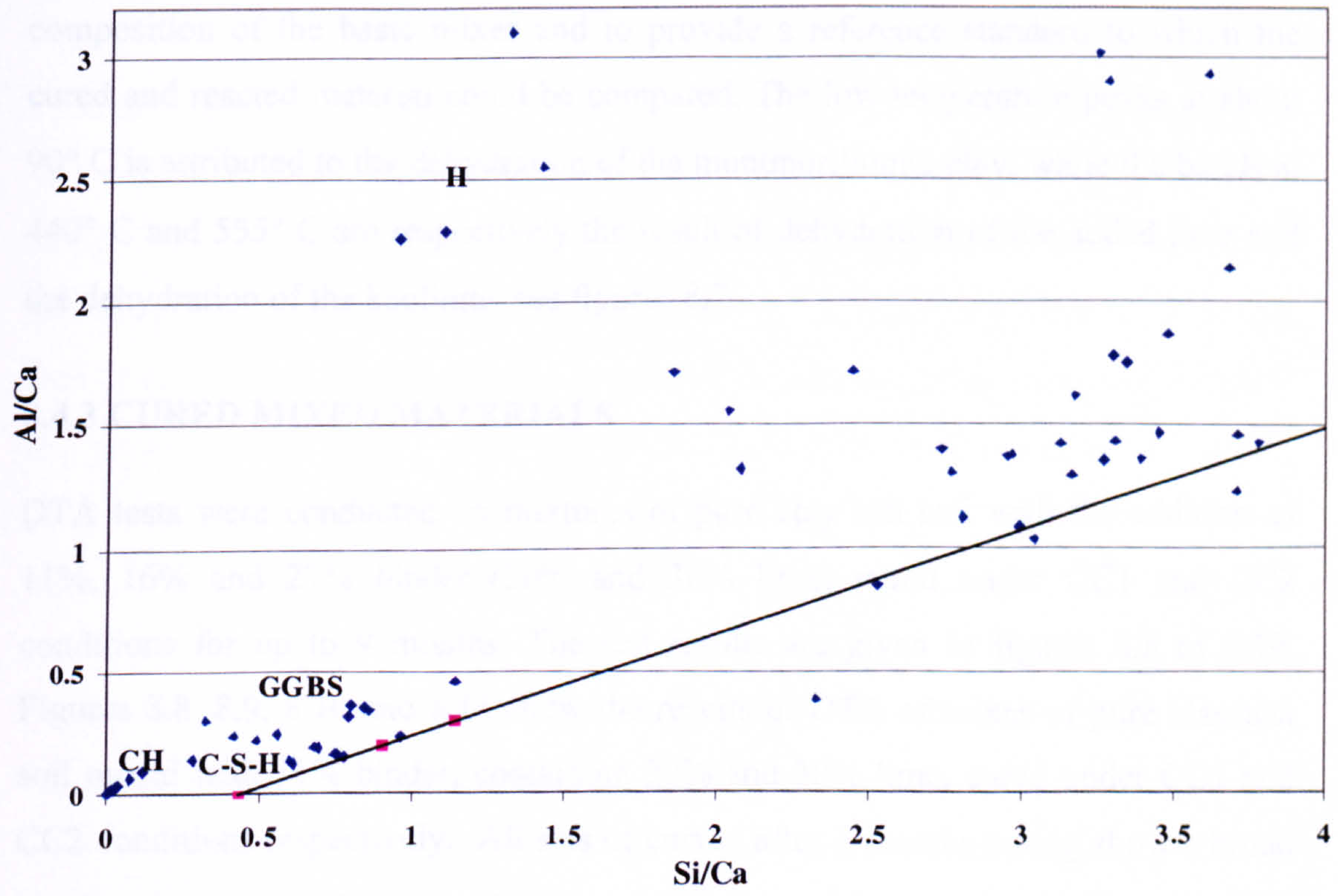


Figure 8.6 Si/Ca against Al/Ca atom ratio for EDX data of the area in plate 8.2 (A, B, C, D, E and F)

8.4 DIFFERENTIAL THERMAL ANALYSIS

8.4.1 INTRODUCTION

Differential thermal analysis, DTA, tests were conducted on cured specimens that had been tested for UCS tests. The results of the former tests were difficult to interpret due to the complex composition of the test soil, (see section 5.6). Therefore, the DTA tests were also carried out on specimens of the pure clay test soil with GGBS and lime in an attempt to identify the reaction products. The DTA data for each composition are grouped by curing period, in an attempt to establish the changes in phase composition during the curing process.

8.4.2 DRY MIXED MATERIALS

DTA tests were carried out on dry mixtures of the pure clay test soil only and pure clay test soil with the addition of 27% binder (GGBS + lime), two lime percentages, 20% and 30%, of binder content were used. These mixtures were tested to obtain the composition of the basic mixes and to provide a reference standard to which the cured and reacted material could be compared. The low temperature peaks at about 90° C is attributed to the dehydration of the montmorillonite clay, while the bands at 440° C and 555° C are respectively the result of dehydration of the added lime and the dehydration of the kaolinite, see figures 8.7.

8.4.3 CURED MIXED MATERIALS

DTA tests were conducted on mixtures of pure clay test soil with the addition of 11%, 16% and 27% binder (20% and 30% lime) cured under CC1 and CC2 conditions for up to 9 months. The test results are given in figures 8.8 to 8.19. Figures 8.8. 8.9, 8.10 and 8.11 show the results of DTA of mixes of pure clay test soil mixed with 11% binder, containing 20% and 30% lime, cured under CC1 and CC2 conditions respectively. All sets of curves after 2 months curing show a broad low temperature weight loss band at 160° C, which may be attributed to the dehydration of C-A-S-H and a weight loss band of calcium hydroxide at 440° C was observed. The carbonation peak at 725° C, the weight loss band due to kaolinite at

555° C and also the montmorillonite peak at 90° C were observed. Increasing the curing period to 3 months caused a decrease in the weight loss band of lime at 440° C and a large increase in the C-S-H peak. A further increase in the curing period to 9 months cause the lime peak to completely disappear while it further increased the C-S-H peak. No significant change was observed in the kaolinite peak after 9 months curing, while the weight loss band of montmorillonite was not observed, although it might be masked by the C-S-H band.

Figures 8.12, 8.13, 8.14 and 8.15 show the results of DTA on mixes of pure clay test soil with 16% binder containing 20% and 30% lime, cured under CC1 and CC2 conditions respectively. Similar trends to those of the previous group were observed except the kaolinite peak decreased with an increase in the curing period. Also, the lime peak in figure 8.14 is larger than that in figure 8.15 because the initial lime content used in the former were 30% of the total binder, while they were 20% of the total binder in the latter. A very small lime peak was observed after 9 months in 8.15 only.

Figures 8.16, 8.17, 8.18 and 8.19 show the results of DTA of mixes of pure clay test soil with 27% binder, containing 20% and 30% lime, cured under CC1 and CC2 respectively. C-S-H gel was dehydrated at a relatively high peak temperature, 165°. The weight loss band of calcium hydroxide at 440° C is still present and the weight loss of kaolinite at 555°C still also present. The most important difference between the mixture with 30% lime and that with 20% lime of the total binder is the kaolinite peak was reduced quickly compared to the lime peak in the former while no significant change was observed in the kaolinite peak in the latter.

Increasing the curing period to 9 months generally increased the C-S-H peaks at 165°C and decreased the calcium hydroxide peak, indicating the consumption of lime with increasing curing period. The C-S-H peak became broader and clearer while kaolinite peak became smaller, indicating the transformation of part of the clay into C-S-H with increasing curing periods. A carbonation peak at 725° C is still present even after 9 months curing period.

XRD results showed the presence of calcium aluminate hydrate phases α C_4AH_{19} and C_4AH_{13}. Although there is no evidence from thermal analysis of the formation of any calcium aluminate hydrate phases due to the formation of the broad peak of C-S-H, three peaks of C_2AH_8 and C_4AH_{13} and aluminum substituted C-S-H at 194°C, 230°C and 210°C respectively might be masked by broad band of the C-S-H band.

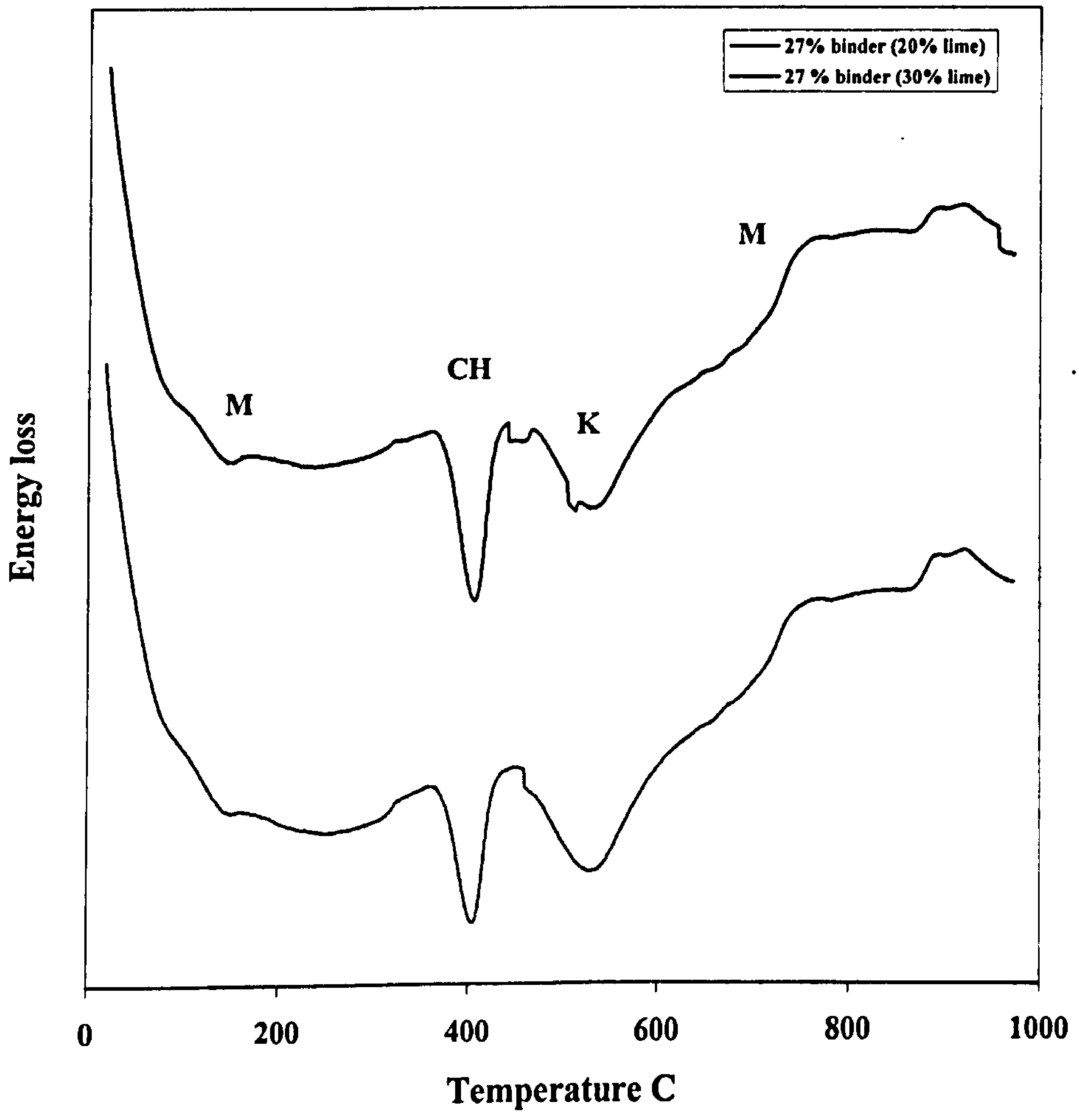


Figure 8.7 DTA for dry pure clay test soil and 27 % binder (20% and 30% lime)

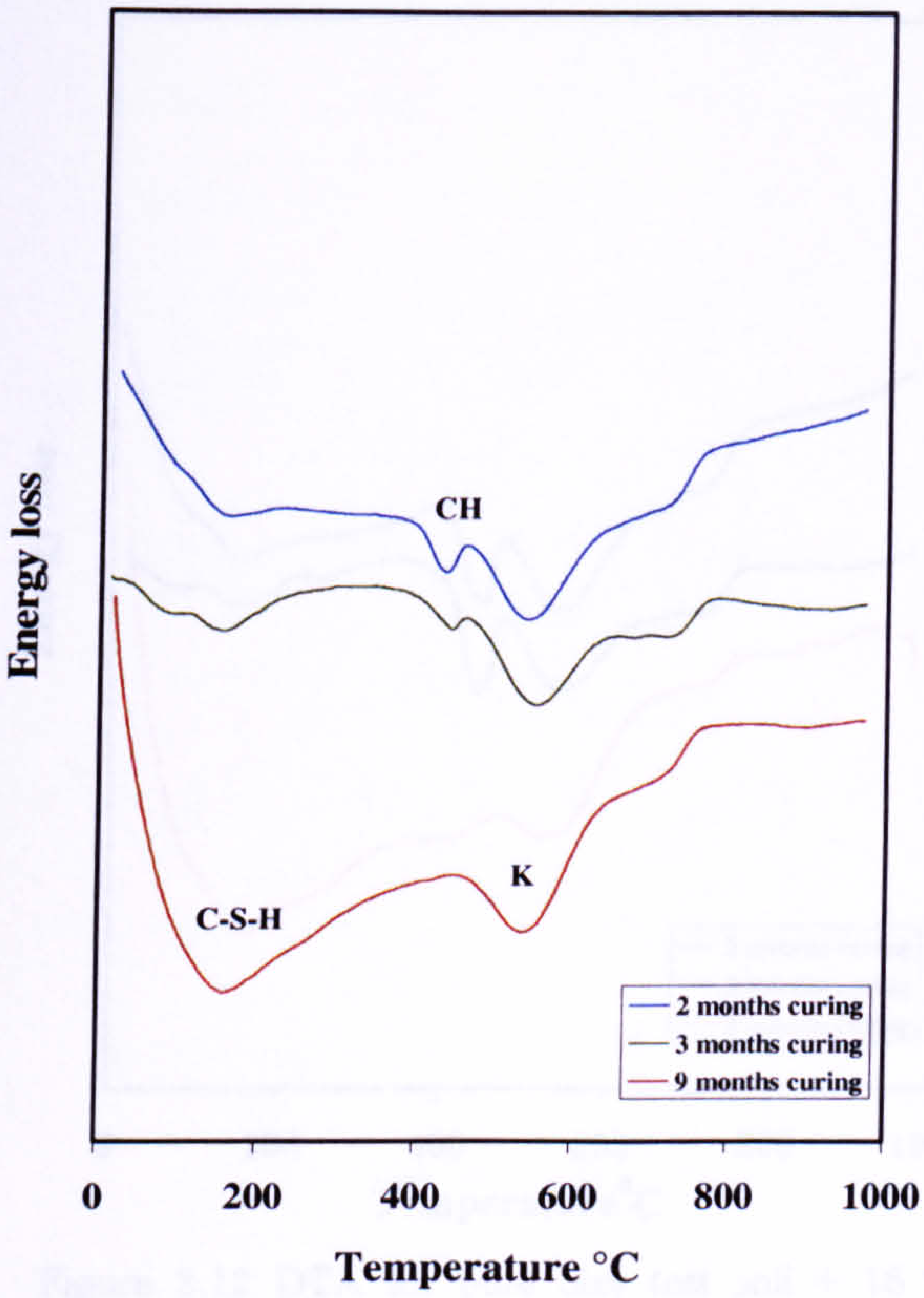


Figure 8.8 DTA for pure clay test soil + 11% binder (30%Lime), cured under CC1 conditions

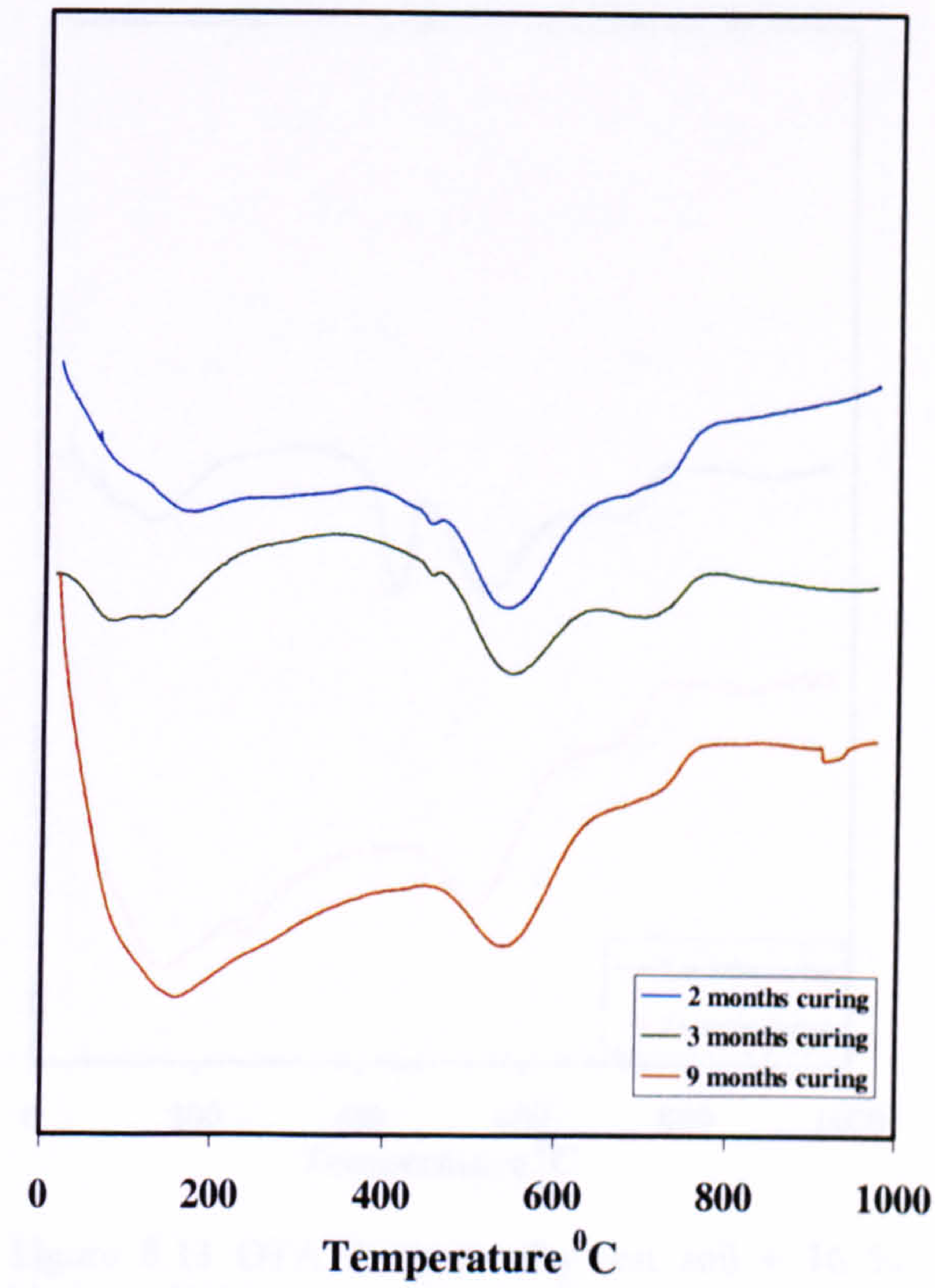


Figure 8.9 DTA for pure clay test soil + 11% binder (20%Lime), cured under CC1 conditions

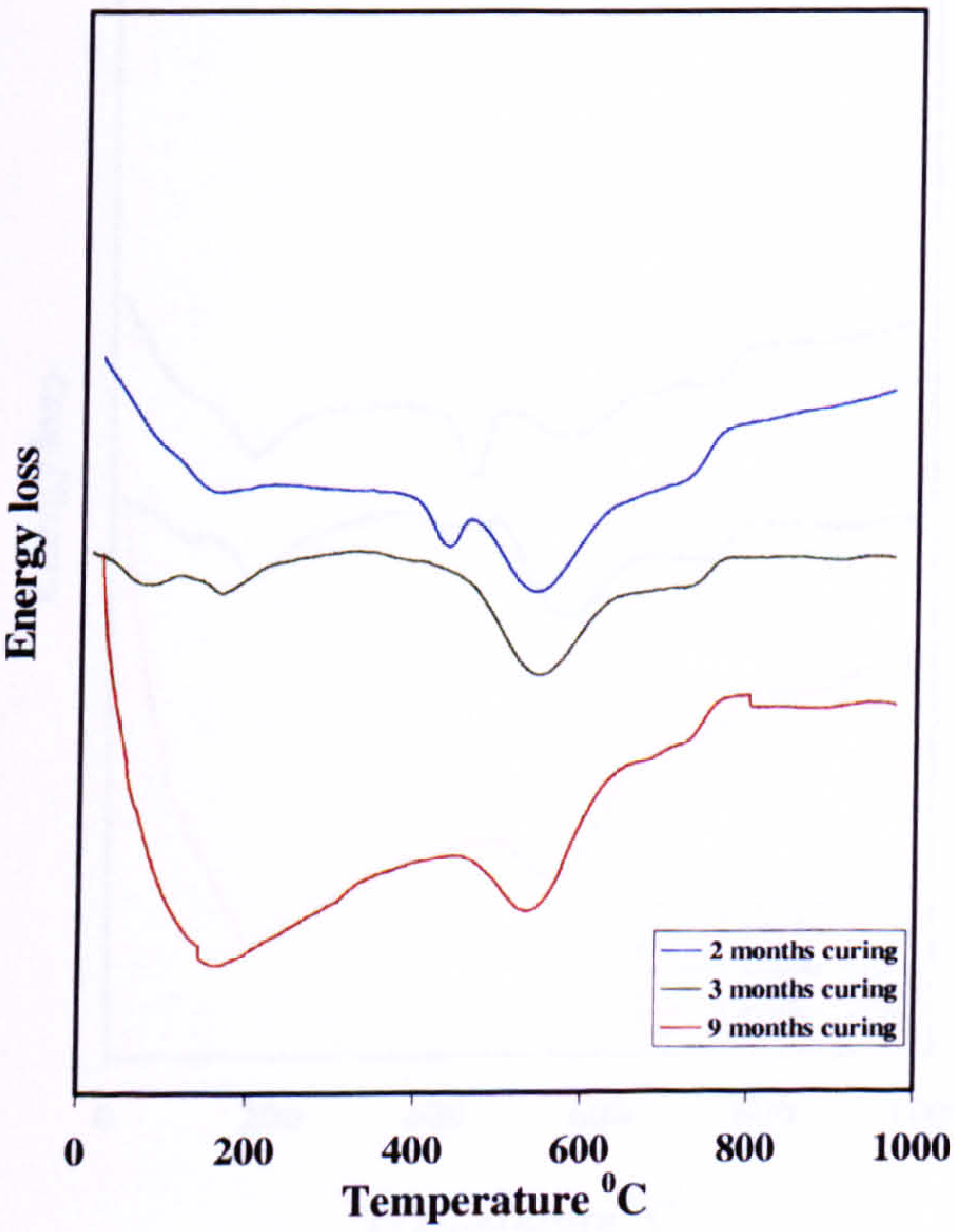


Figure 8.10 DTA for pure clay test soil + 11 % binder (30%Lime), cured under CC2 conditions

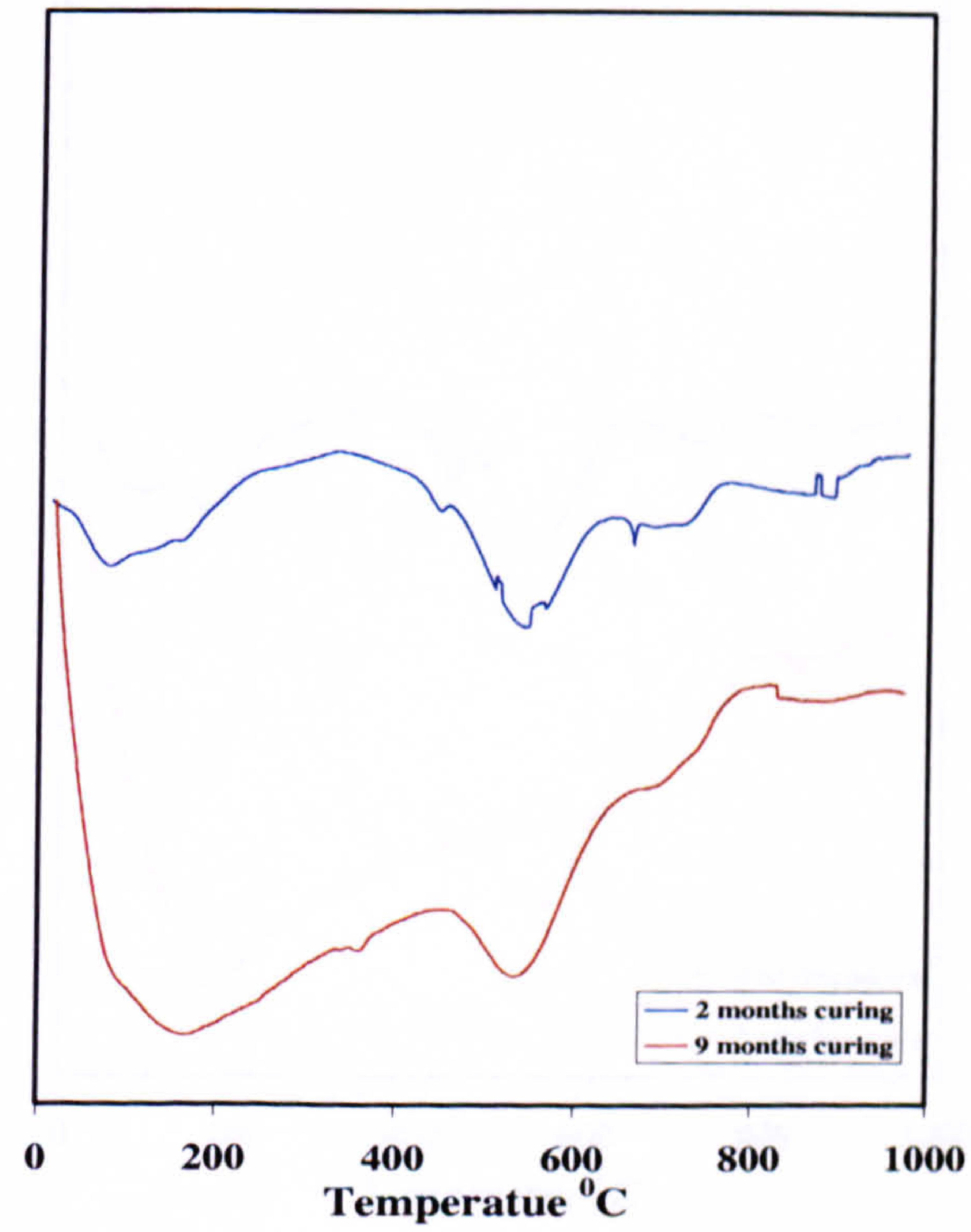


Figure 8.11 DTA for pure clay test soil + 11 % binder (20%Lime), cured under CC2 conditions

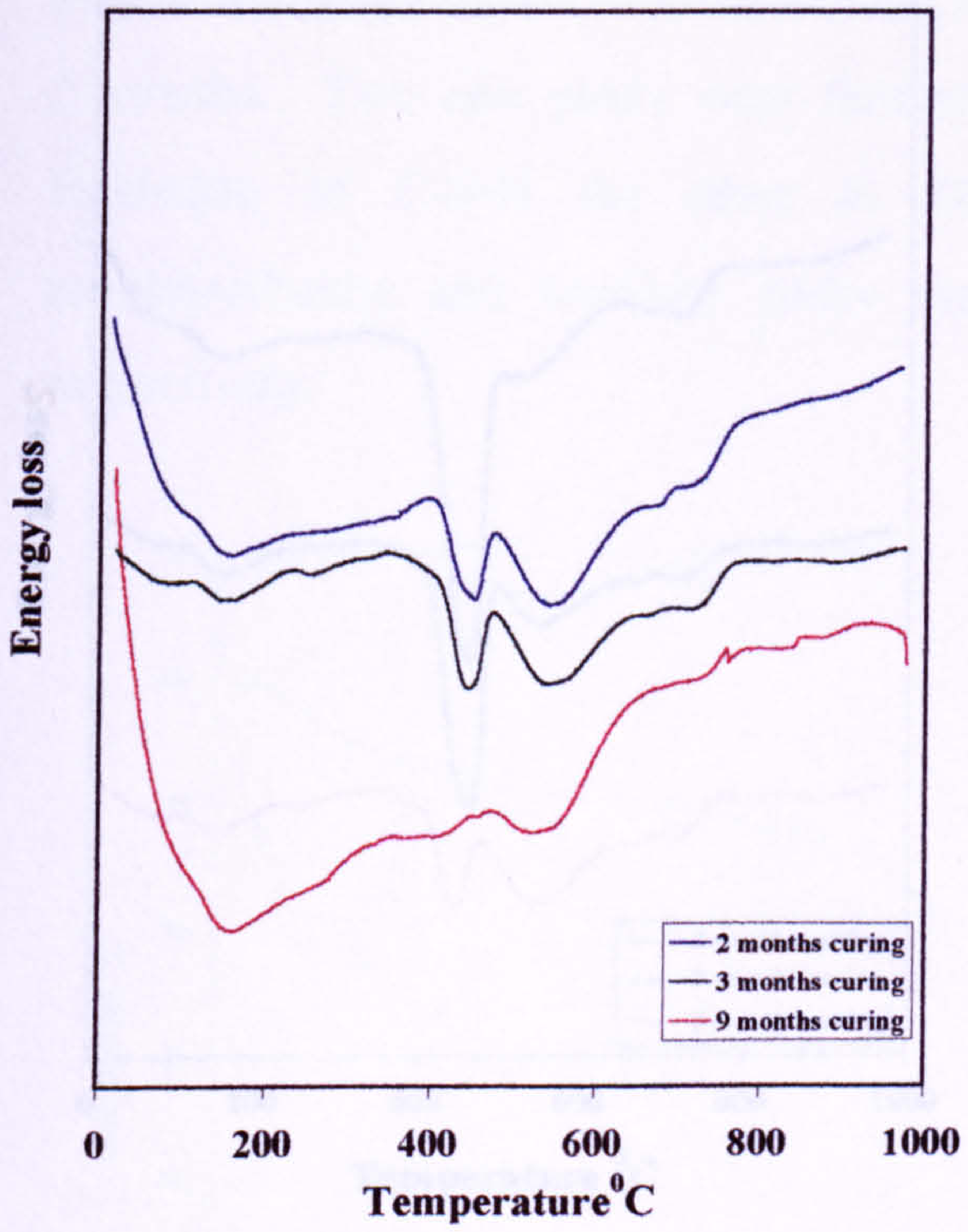


Figure 8.12 DTA for pure clay test soil + 16 % binder (30%Lime), cured under CC1 conditions

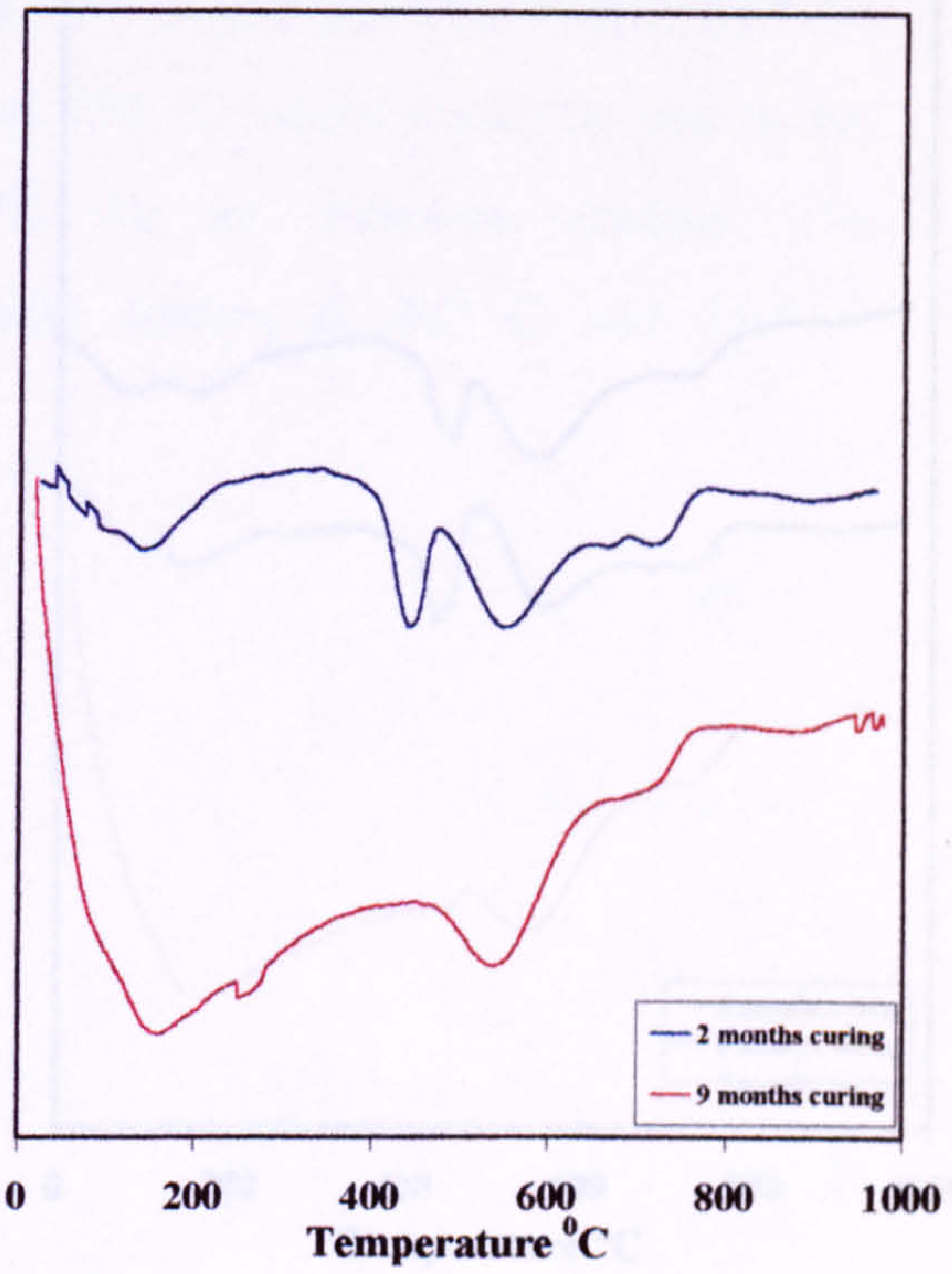


Figure 8.13 DTA for pure clay test soil + 16 % binder (20%Lime), cured under CC1 conditions

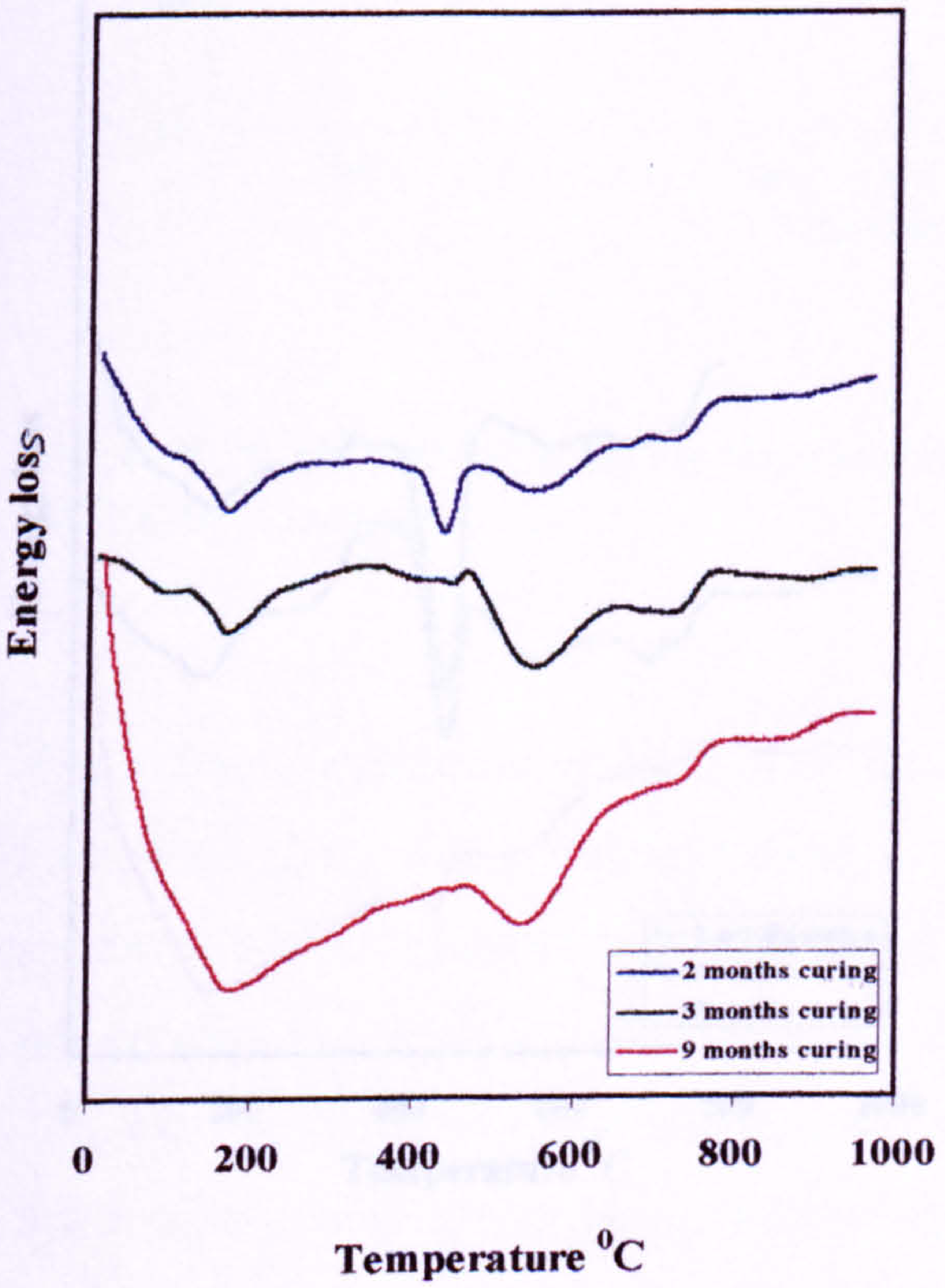


Figure 8.14 DTA for pure clay test soil + 16 % binder (30%Lime), cured under CC2 conditions

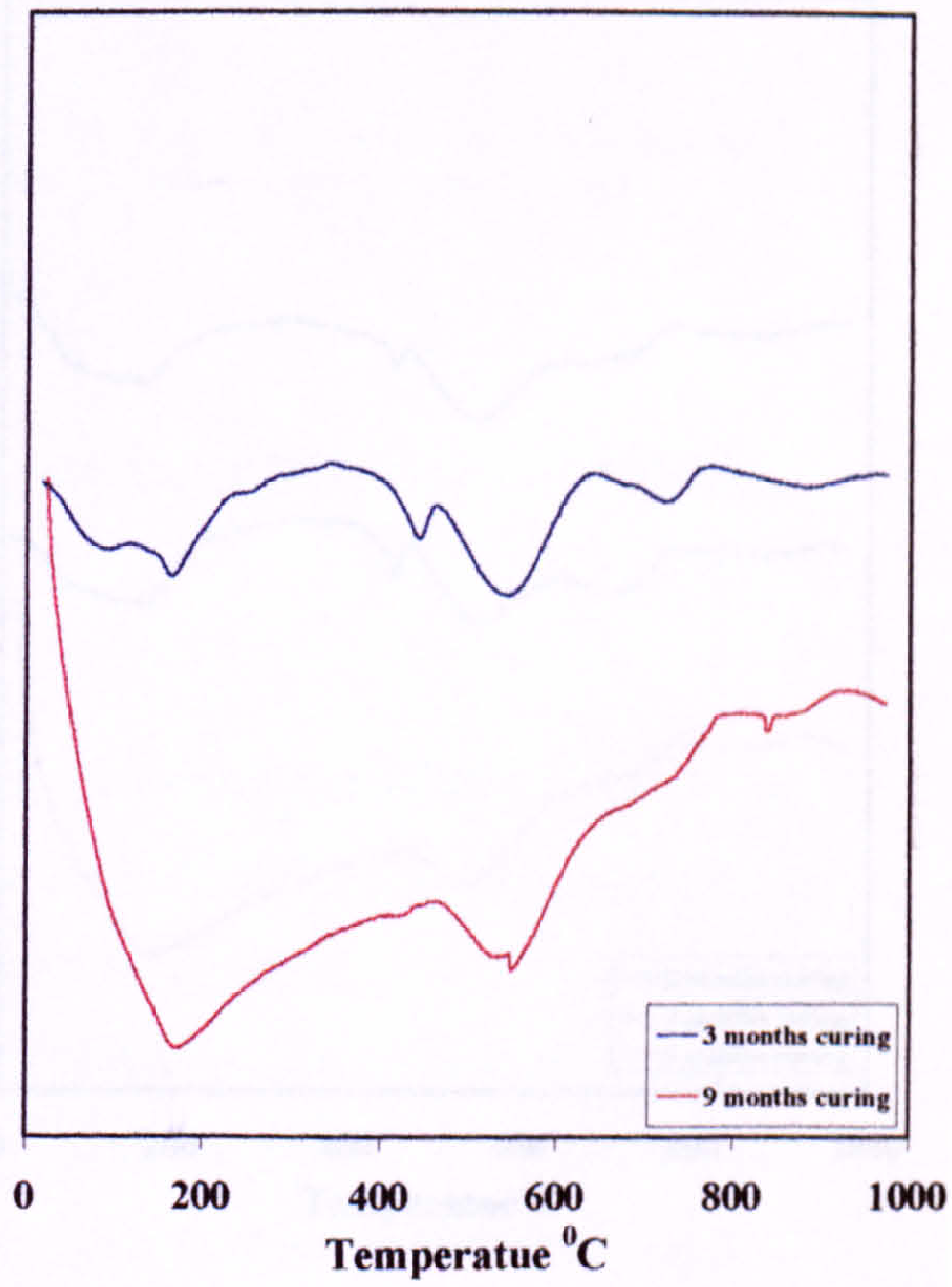


Figure 8.15 DTA for pure clay test soil + 16 % binder (20%Lime), cured under CC2 conditions

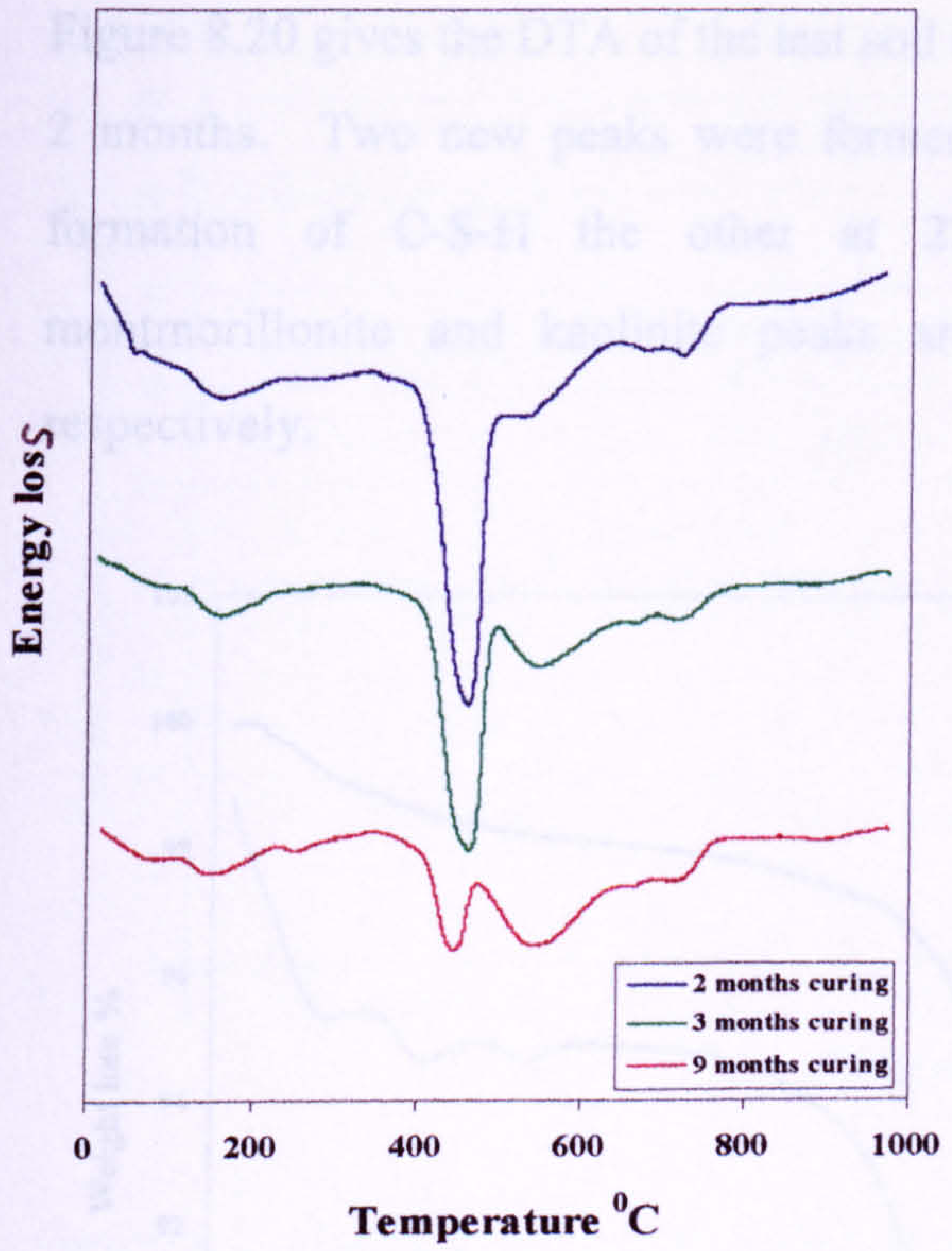


Figure 8.16 DTA for pure clay test soil + 27 % binder (30%Lime), cured under CC1 conditions

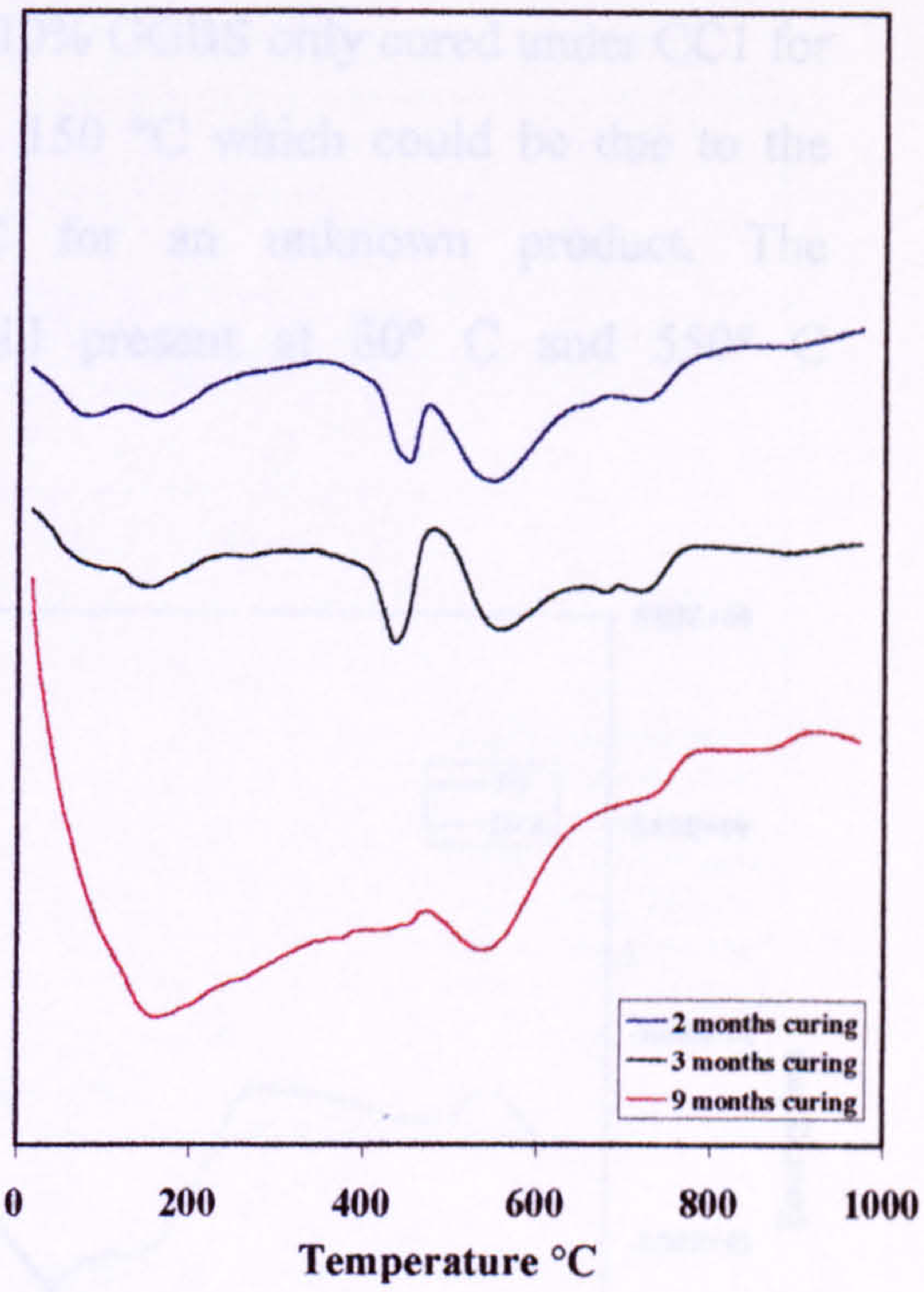


Figure 8.17 DTA for pure clay test soil + 27 % binder (20%Lime), cured under CC1 conditions

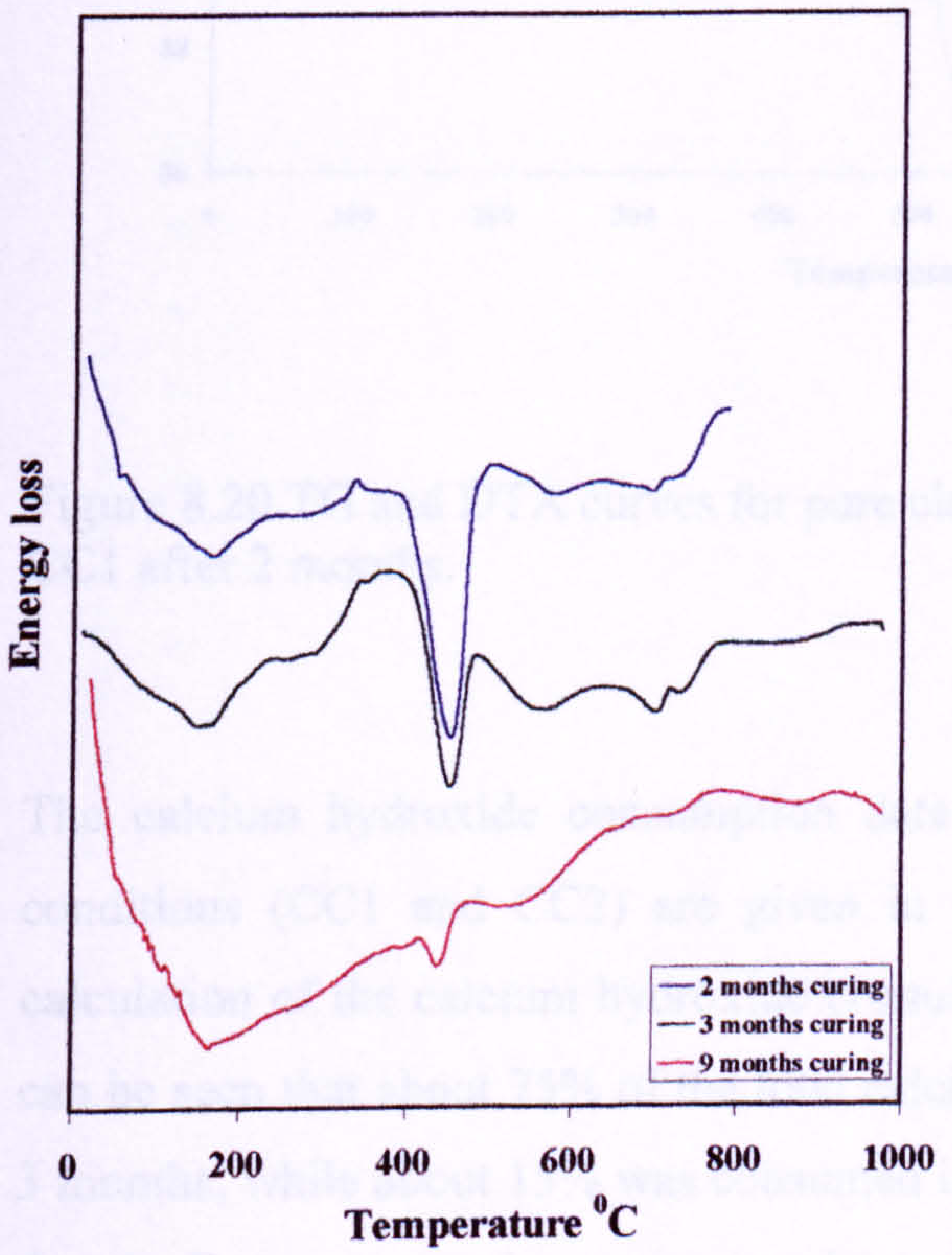


Figure 8.18 DTA for pure clay test soil + 27 % binder (30%Lime), cured under CC2 conditions

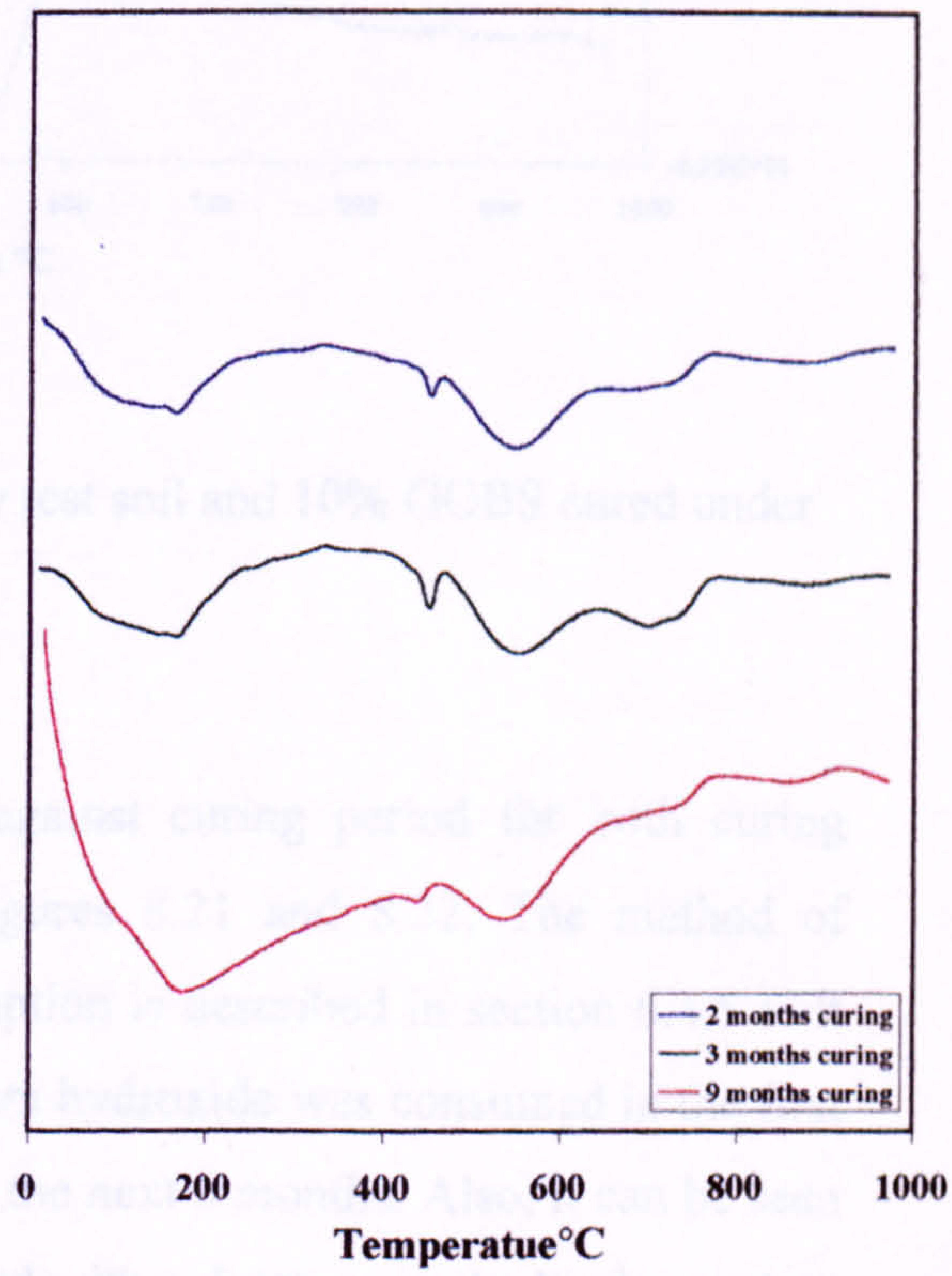


Figure 8.19 DTA for pure clay test soil + 27 % binder (20%Lime), cured under CC2 conditions

Figure 8.20 gives the DTA of the test soil and 10% GGBS only cured under CC1 for 2 months. Two new peaks were formed, at 150 °C which could be due to the formation of C-S-H the other at 226°C for an unknown product. The montmorillonite and kaolinite peaks are still present at 80° C and 550° C respectively.

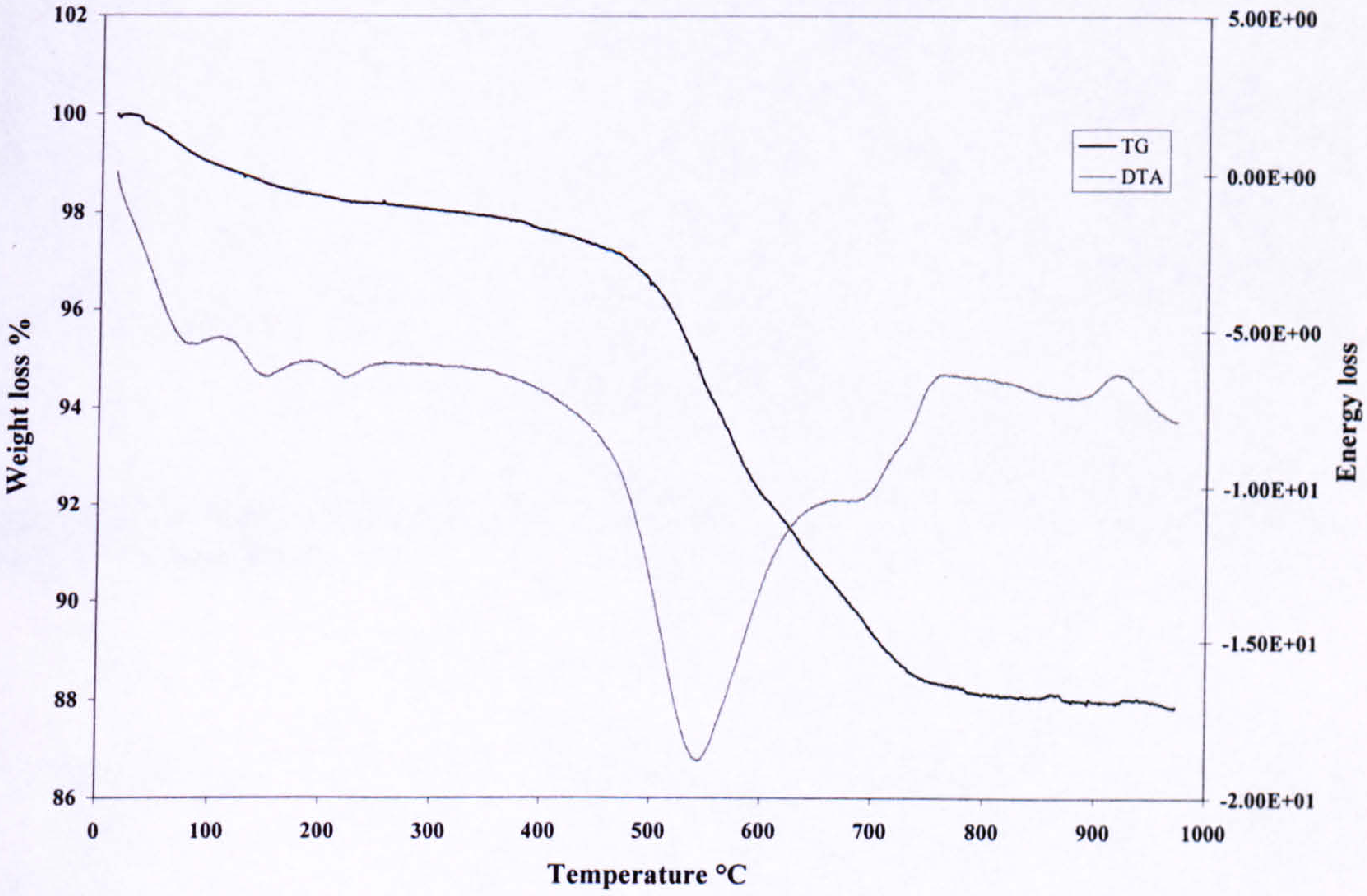


Figure 8.20 TG and DTA curves for pure clay test soil and 10% GGBS cured under CC1 after 2 months.

The calcium hydroxide consumption data against curing period for both curing conditions (CC1 and CC2) are given in figures 8.21 and 8.22. The method of calculation of the calcium hydroxide consumption is described in section 6.4.5.1. It can be seen that about 75% of the total calcium hydroxide was consumed in the first 3 months, while about 15% was consumed in the next 6 months. Also, it can be seen that the lime consumption percentage increased with a decrease in the binder content and lime/GGBS ratio. Due to the presence of some calcium carbonate in the hydrated lime, it is impossible to achieve 100% calcium hydroxide consumption, the maximum consumption being ≈ 85%.

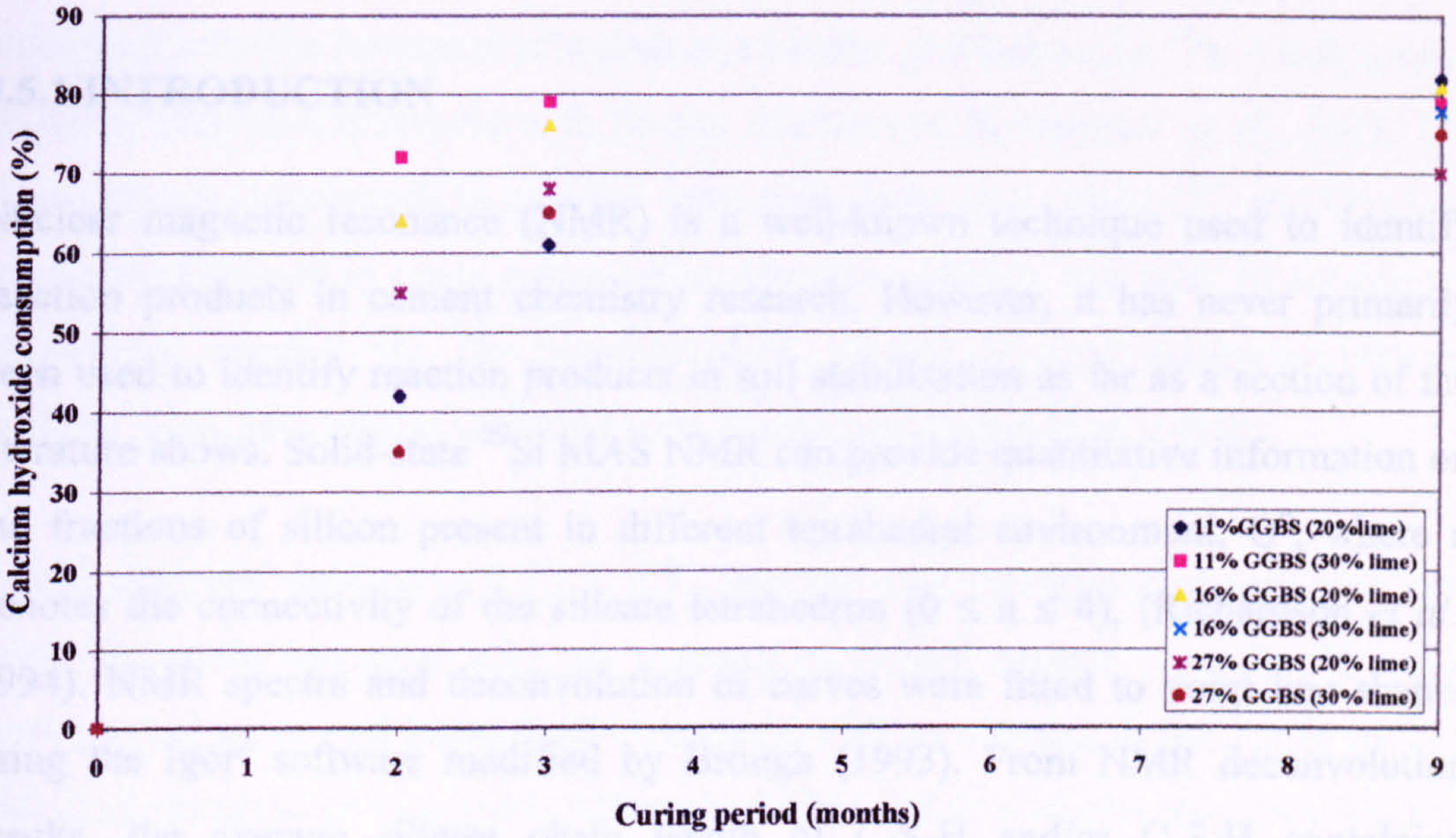


Figure 8.21 Calcium hydroxide consumption (%) against curing periods (months), under CC1 conditions

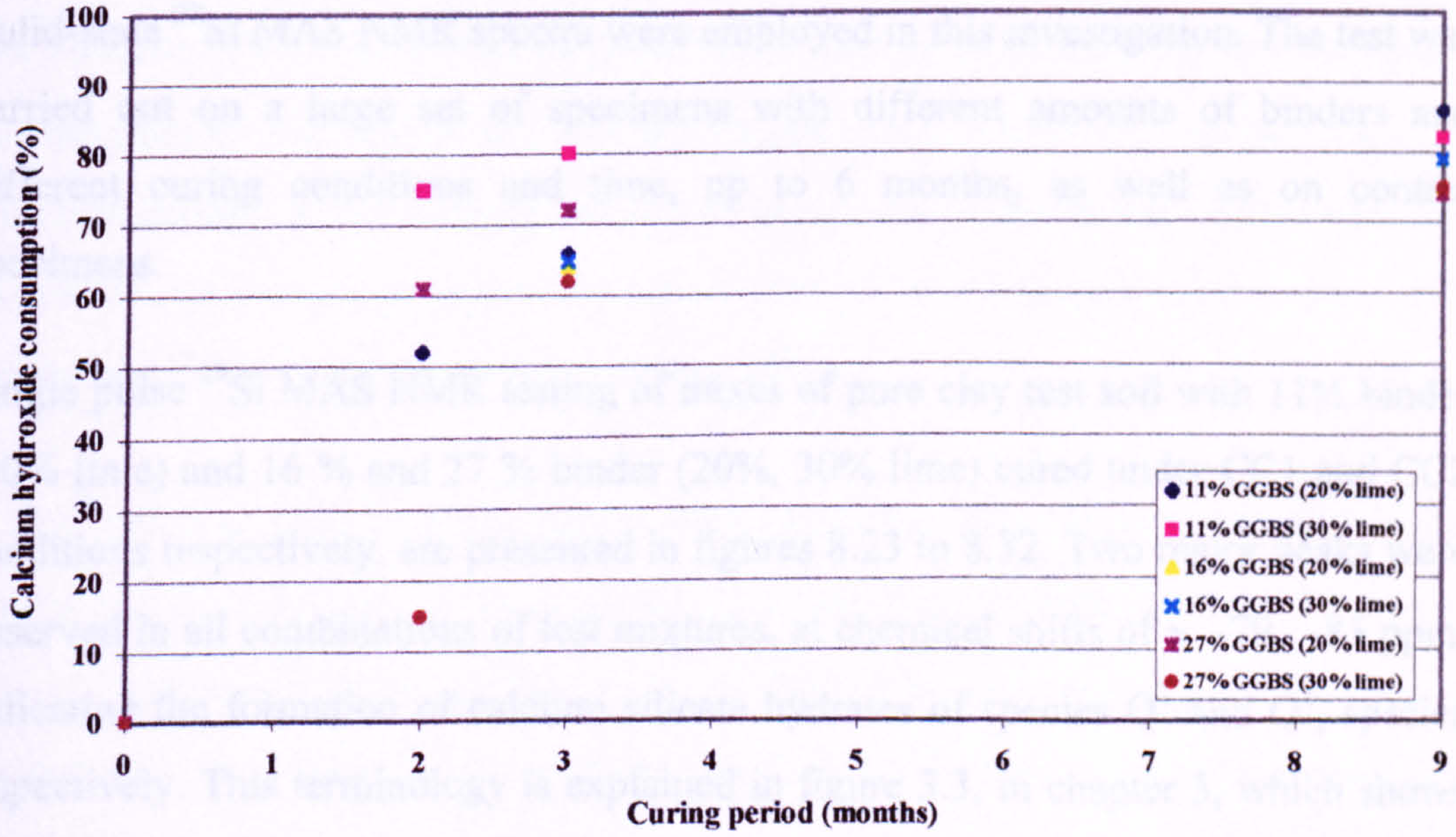


Figure 8.22 Calcium hydroxide consumption (%) against curing periods (months), under CC2 conditions

8.5 NUCLEAR MAGNETIC RESONANCE (N M R)

8.5.1 INTRODUCTION

Nuclear magnetic resonance (NMR) is a well-known technique used to identify reaction products in cement chemistry research. However, it has never primarily been used to identify reaction products in soil stabilisation as far as a section of the literature shows. Solid-state ^{29}Si MAS NMR can provide quantitative information on the fractions of silicon present in different tetrahedral environment, Q^n, where n denotes the connectivity of the silicate tetrahedron ($0 \leq n \leq 4$), (Richardson *et al.*, 1994). NMR spectra and deconvolution of curves were fitted to voigt line shapes using the Igor^2 software modified by Brough (1993). From NMR deconvolution results, the average silicate chain length of C-S-H and/or C-S-H containing aluminium CS(A)H, the Al/Si ratio and the percentage of the total Si atoms in C-S-H species can be determined.

8.5.2 NMR TEST RESULTS AND INTERPRETATION

Solid-state ^{29}Si MAS NMR spectra were employed in this investigation. The test was carried out on a large set of specimens with different amounts of binders and different curing conditions and time, up to 6 months, as well as on control specimens.

Single pulse ^{29}Si MAS NMR testing of mixes of pure clay test soil with 11% binder (30% lime) and 16 % and 27 % binder (20%, 30% lime) cured under CC1 and CC2 conditions respectively, are presented in figures 8.23 to 8.32. Two major peaks were observed in all combinations of test mixtures, at chemical shifts of ≈ -79, -85 ppm, indicating the formation of calcium silicate hydrates of species Q^1 and Q^2, species respectively. This terminology is explained in figure 3.3, in chapter 3, which shows schematic representation of pentameric silicate chains. SiO_4^{-4} units are normally identified according to their mutual connectivity as $Q^n(m\text{Al})$, where *n* is the number of shared oxygen atoms with other silicate or aluminate tetrahedra, and *m* is the number of neighbour aluminate groups (Engelhardt and Michel, 1987; Schneider *et* *al.*, 2001). Also, a major peak observed at ≈ - 81 ppm is attributed to Q^2 (1Al), consistent with the formation of a peak at a similar position in the ^{29}Si NMR spectra of Al substituted tobermorite with similar mixtures by Komarneni e*t al.*, 1987. The Q^1 resonance corresponds to silicates located at chain ends, while Q^2 and Q^2 (1Al) resonance assigned to chain mid-member silicates, without and with Al substituted tetrahedran, respectively. The presence of Q^1, Q^2 and Q^2 (1Al) suggested that the bridging sites were almost saturated with aluminium (Richardson, 1999).

It was observed on figures 8.23 to 8.32 that the area and height of the three new peaks generally increased with an increase in GGBS content, lime/GGBS ratio, curing temperature and dramatically increased with an increase in the curing period. The exact location of each peak (chemical shifts) and line widths may vary slightly from one mixture to another, depending upon the binder content, lime/GGBS ratio and curing conditions.

The NMR data deconvolution for some mixtures are presented in tables 8.1 to 8.4 and figures 8.35 to 8.42. There are two main parameters characterizing the structural and composition of the C-A-S-H phase which can be obtained from the NMR data deconvolution. The first is the average silicate chain length $\overline{CL}$ of the C-A-S-H, which is defined as the average number of tetrahedra between two empty sites. The second is the location and the content of aluminium substituted in the chain with respect to silicon (Richardson and Groves, 1997).

It was observed that the chain length stays about the same with an increase in the binder and lime content, at the same curing conditions and curing period. However, the $\overline{CL}$ increases with an increase in the curing temperature and period, which is in agreement with a study on the hydration of ordinary Portland cement incorporating silica fume carried out by Richardson (2000). He observed an increase in $\overline{CL}$ of the C-A-S-H with age, while the effect of curing temperature on the $\overline{CL}$ was not studied.

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[2] Wave Metric, Inc., Igor (1992) and Igor Pro (1996), Lake Oswego, Oregon, 9703, USA.

The increase in $\overline{CL}$ with increases in the curing temperature and period could be due to increase in Al substitution at higher temperature and with longer curing period. It was also observed that the C-A-S-H formed has a very long $\overline{CL}$ compared to the white Portland cement–GGBS paste that has a mean $\overline{CL}$ of 4.05 (Richardson and Groves, 1997). The minimum calculated $\overline{CL}$ was 4.6 in the mixture of 16% GGBS (20%lime) cured at 20°C after 3 months, while the maximum calculated value of $\overline{CL}$ was 21.4 in a mixture of 27% GGBS (20% lime) cured at 35° C after 3 months curing. Increasing silicate chain length could indicate a more crystalline and more homogenous C-A-S-H product.

The aluminium: silicon (Al/Si) ratio was also estimated. It was observed that the Al/Si ratio stays about the same with an increase in binder and lime contents due to the high alumina content of both test soil and GGBS. However, the Al/Si ratio normally increases in ordinary Portland cement- GGBS systems with an increase in the GGBS content due to the very high alumina content of GGBS compared to OPC. The Al/Si ratio in this system is still lower than the ratio obtained by Richardson (1999) for a similar GGBS. He activated a medium Al content slag (Al/Si of anhydrous slag =0.36) using 5 *M* KOH, and the Al/Si ratio of the C-A-S-H formed was 0.35.

It was observed that the Al/Si ratio significantly increases with an increase in curing temperatures and period, due to the increase in Q^2 (1Al) content (table 8.1) with an increase in curing period due to an increase in silicon substituted by aluminium in C-A-S-H compound with an increase in curing period. It was observed that the Al/Si ratio increased with an increase in the average chain length $\overline{CL}$, see figure 8.33, which is due to the aluminium filling the empty bridging sites giving longer chains (Schneider *et al.*, 2001). Also, the increase in Q^2 (1Al) with no significant change in Q^1 and Q^2 with an increase in the curing temperature and period substantially supports the principle of increasing Al substitution at high temperature and period and filling the empty bridging sites.

The amount of Si in C-A-S-H also increased with an increase in the curing period under CC1 conditions, keeping other parameters constant. However no significant change was observed with an increase in the curing period under CC2 conditions. Large increases were observed in the amount of Si in C-A-S-H with an increase in curing temperature (figure 8.34) while a significant increase was also found in the C-A-S-H content with an increase in binder content under CC2. The $\overline{CL}$ also increased with an increase in the percentage of $Q^2(1Al)$ especially at higher temperature due to an increase in aluminium substitution in high temperature and period and filling the empty bridging sites, thus leading to long chain length (figure 8.34). It was also observed that the chemical shifts were not significantly changed with curing periods, binder content, lime content or with curing conditions. No systematic variations were found in peak widths with the change in curing periods, binder content and lime content or with curing conditions.

NMR data deconvolution suggested the presence of two other peaks at ≈ –91 ppm and at ≈ –94 ppm. The peak at ≈ –94 ppm could be due to the presence of the residual unreacted montmorillonite. The data in table 8.4 suggest that the peak at ≈ -91 ppm is due to both Q^3 and the unreacted kaolinite (Q^3a) as the minimum amount of (Q^3a) formed was 57.60%, which is greater than the amount of kaolinite originally present in the sample (< 45%). Therefore, Q^3 might be formed at the same chemical shift of kaolinite, however, further investigations are needed including NMR spectra for kaolinite and montmorillonite to separate these two peaks.

Table 8.1 NMR deconvolution of data after 3 and 6months

3 Months	20° C		35 °C	
27 %GGBS	20% lime	30% lime	20% lime	30% lime
Chain length	5.80	5.10	21.40	19.70
Al/Si ratio	0.11	0.09	0.23	0.21
Percent of Si in C-A-S-H	13.70	17.40	29.40	34.90
16% GGBS				
Chain length	4.60	5.60	21.40	15.60
Al/Si ratio	0.03	0.08	0.23	0.21
Percent of Si in C-A-S-H	22.70	19.20	26.10	27.00
11% GGBS				
Chain length	N/A	6.00	N/A	17.90
Al/Si ratio		0.07		0.21
Percent of Si in C-A-S-H		17.80		19.30
6 Months	20° C		35 °C	
27 %GGBS	20% lime	30% lime	20% lime	30% lime
Chain length	6.50	N/A	20.10	N/A
Al/Si ratio	0.11		0.24	
Percent of Si in C-A-S-H	18.60		25.10	
16% GGBS				
Chain length	7.30	9.00	18.00	17.90
Al/Si ratio	0.11	0.13	0.24	0.23
Percent of Si in C-A-S-H	22.00	23.10	25.90	26.90
11% GGBS	N/A	N/A	N/A	N/A

Table 8.2 NMR data of Q^n species percentage after 3 and 6 months

3 Months	20° C		35 °C	
27 %GGBS	20% lime	30% lime	20% lime	30% lime
Q^1	5.30	7.50	3.40	4.30
Q^2(1Al)	2.90	3.00	13.30	15.00
Q^2	5.50	6.90	12.80	15.6
Q^{3}a	78.00	76.5	62.90	59.8
Q^{3}b	8.20	6.00	7.70	5.30
16% GGBS				
Q^1	10.10	7.30	3.00	4.10
Q^2(1Al)	1.50	3.10	11.80	11.40
Q^2	11.10	8.70	11.20	11.30
Q^{3}a	64.50	57.60	57.60	57.30
Q^{3}b	12.80	23.00	16.30	15.90
11% GGBS				
Q^1	N/A	6.30	N/A	2.60
Q^2(1Al)		2.40		8.10
Q^2		9.10		8.60
Q^{3}a		63.30		58.60
Q^{3}b		18.90		22.10
6 Months	20° C		35 °C	
27 %GGBS	20% lime	30% lime	20% lime	30% lime
Q^1	6.30	N/A	13.10	N/A
Q^2(1Al)	4.00		12.10	
Q^2	8.30		10.00	
Q^{3}a	66.20		62.00	
Q^{3}b	15.10		12.90	
16% GGBS				
Q^1	6.70	5.80	3.60	3.70
Q^2(1Al)	5.00	6.20	12.50	12.60
Q^2	10.40	11.00	9.80	10.60
Q^{3}a	59.90	64.40	58.10	59.40
Q^{3}b	18.10	12.50	16.00	13.70
11% GGBS	N/A	N/A	N/A	N/A

Table 8.3 NMR line widths after 3 and 6 months

3 Months	20° C		35 °C	
27 %GGBS	20% lime	30% lime	20% lime	30% lime
Q^1 Q^2(1Al)	2.127	2.40	2.92	2.87
Q^2				
Q^3a	3.672	3.651	2.37	2.61
Q^3b	2.627	2.142	1.95	1.68
16% GGBS				
Q^1 Q^2(1Al)	3.25	2.65	2.63	2.52
Q^2				
Q^3a	2.18	2.06	2.09	2.12
Q^3b	2.40	3.19	2.79	2.87
11% GGBS				
Q^1 Q^2(1Al)	N/A	3.13	N/A	2.65
Q^2				
Q^3a		2.13		2.12
Q^3b		2.59		3.03
6 Months	20° C		35 °C	
27 %GGBS	20% lime	30% lime	20% lime	30% lime
Q^1 Q^2(1Al)	2.68	N/A	2.66	N/A
Q^2				
Q^3a	3.14		2.50	
Q^3b	3.29		2.86	
16% GGBS				
Q^1 Q^2(1Al)	2.75	2.88	2.70	2.59
Q^2				
Q^3a	2.47	2.34	1.89	2.01
Q^3b	3.11	2.43	2.84	2.64
11% GGBS	N/A	N/A	N/A	N/A

Table 8.4 NMR data shifts after 3 and 6 months

3 Months	20° C		35 °C	
27 %GGBS	20% lime	30% lime	20% lime	30% lime
Q^1	-78.74	-79.04	-79.19	-78.22
Q^2(1Al)	-80.90	-81.17	-80.95	-80.31
Q^2	-84.59	-84.67	-84.67	-84.52
Q^3a	-91.37	-91.38	-91.21	-91.21
Q^3b	-94.23	-94.36	-93.85	-93.83
16% GGBS				
Q^1	-79.50	-79.13	-79.27	-79.47
Q^2(1Al)	-80.25	-80.82	-81.20	-81.17
Q^2	-84.86	-84.75	-84.75	-84.71
Q^3a	-91.28	-91.27	-91.29	-91.27
Q^3b	-93.82	-93.51	-93.70	-93.73
11% GGBS				
Q^1	N/A	-79.63	N/A	-79.75
Q^2(1Al)		-80.81		-81.58
Q^2		-84.87		-85.10
Q^3a		-91.28		-91.32
Q^3b		-93.69		-93.66
6 Months	20° C		35 °C	
27 %GGBS	20% lime	30% lime	20% lime	30% lime
Q^1	-79.24	N/A	-79.09	N/A
Q^2(1Al)	-80.75		-81.41	
Q^2	-84.75		-84.94	
Q^3a	-91.26		-91.25	
Q^3b	-93.80		-93.89	
16% GGBS				
Q^1	-79.32	-79.34	-79.39	-79.56
Q^2(1Al)	-80.90	-80.69	-81.46	-81.46
Q^2	-84.70	-84.70	-84.91	-84.99
Q^3a	-91.27	-91.26	-91.27	-91.23
Q^3b	-93.71	-93.87	-93.80	-93.73
11% GGBS	N/A	N/A	N/A	N/A

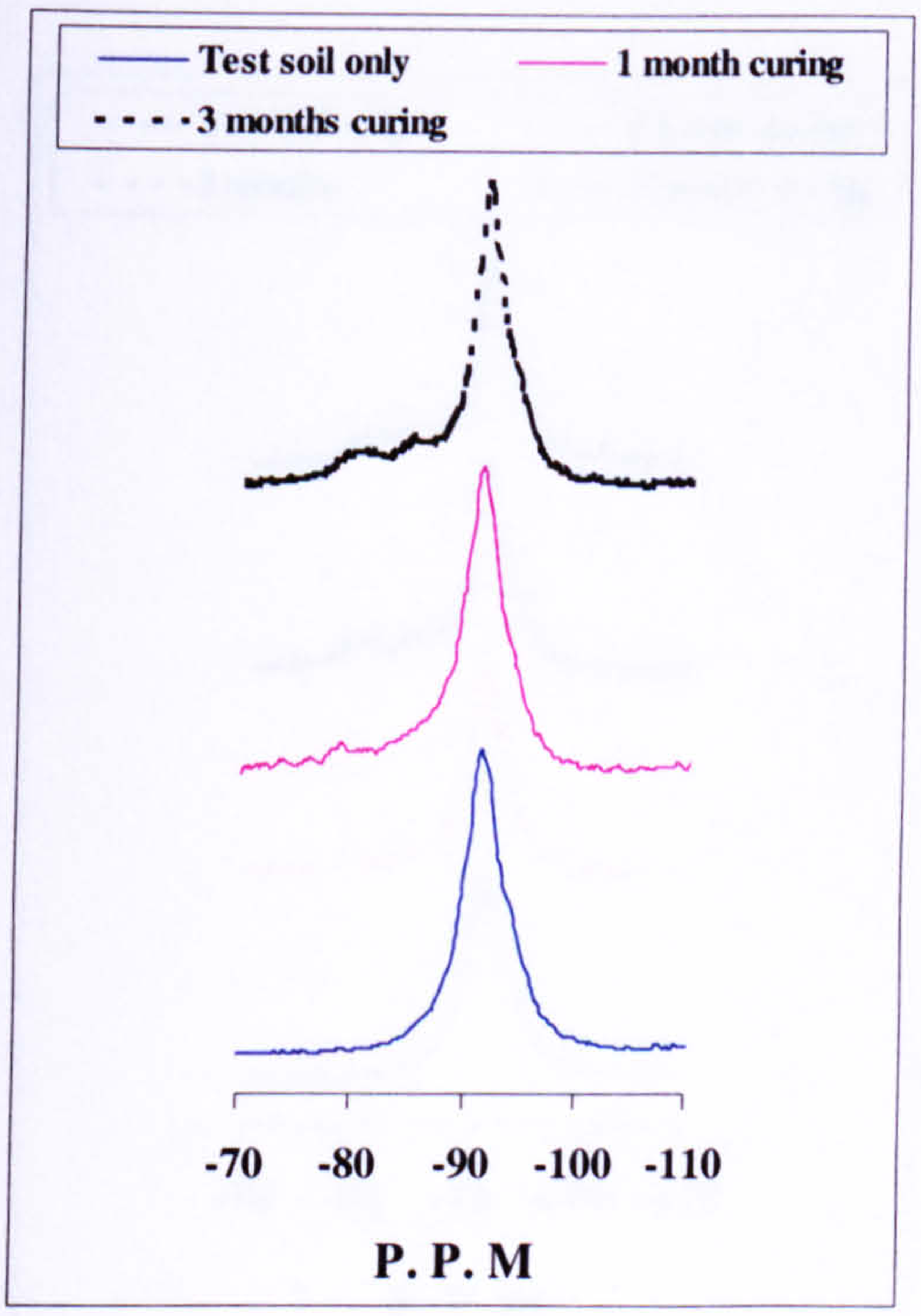


Figure 8.23 Single pulse ^{29}Si NMR spectrum for pure clay test soil + 11 % GGBS (30%Lime), cured under CC1 conditions

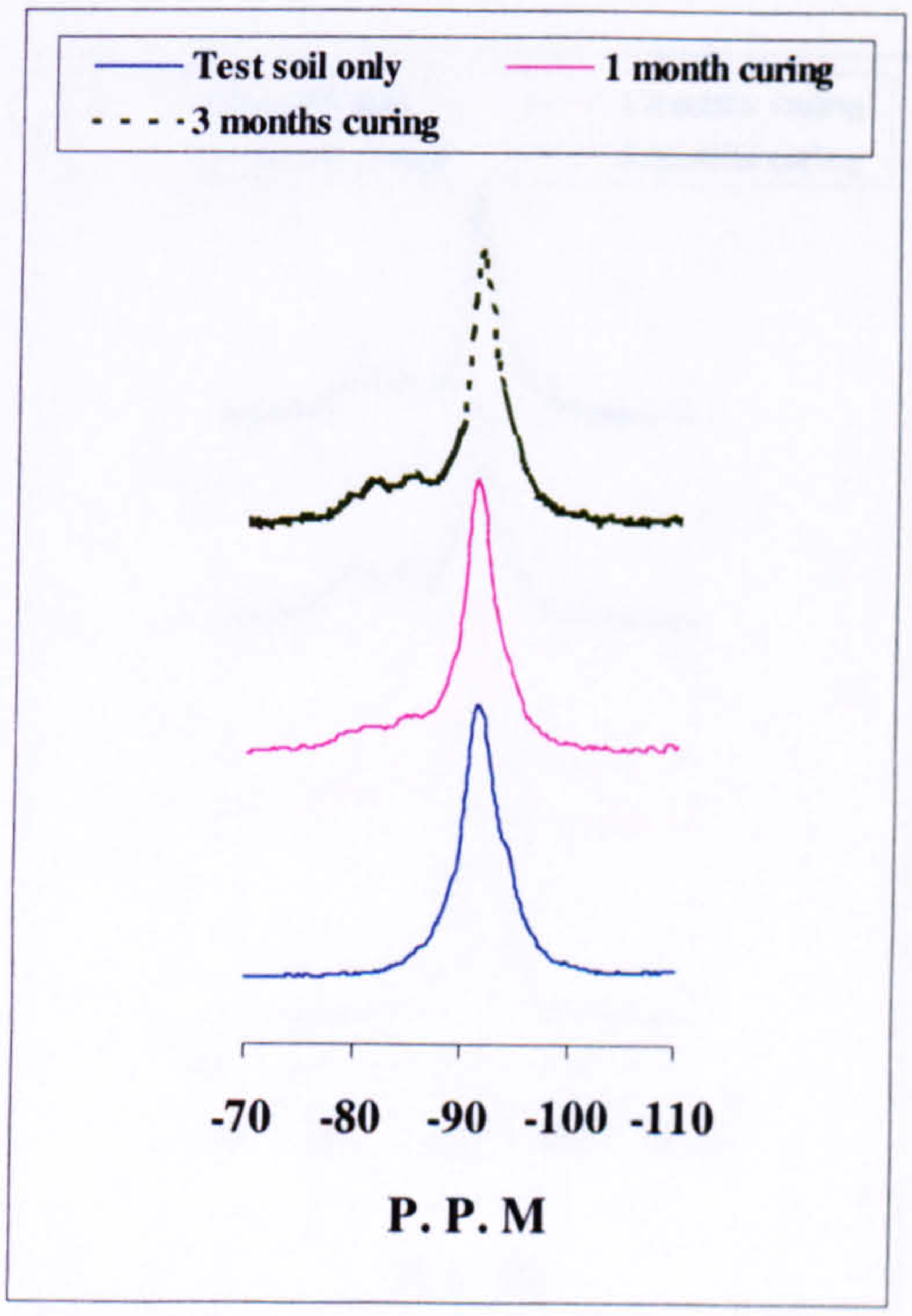


Figure 8.24 Single pulse ^{29}Si NMR spectrum for pure clay test soil + 11 % GGBS (30%Lime), cured under CC2 conditions

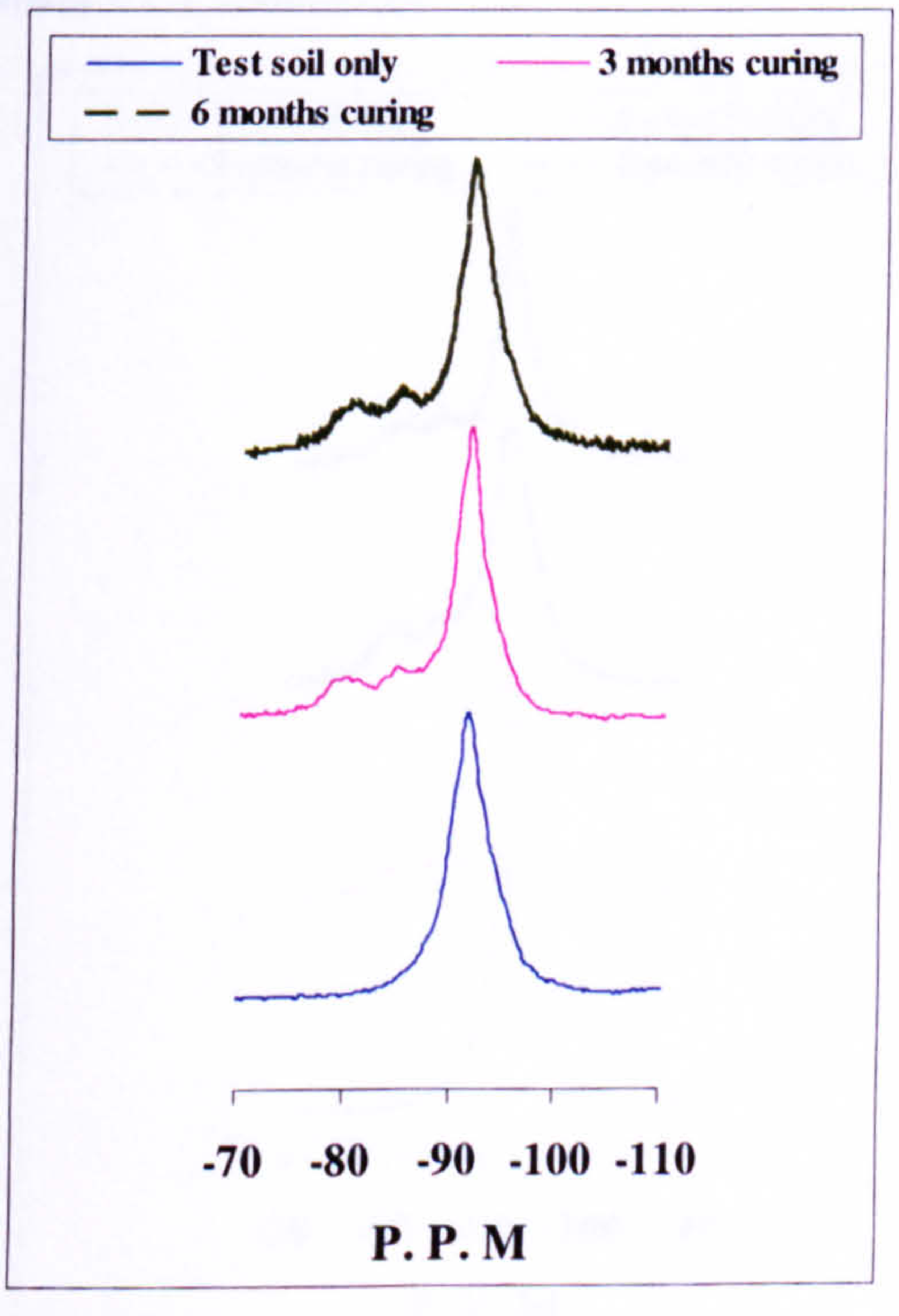


Figure 8.25 Single pulse ^{29}Si NMR spectrum for pure clay test soil + 16 % GGBS (20%Lime), cured under CC1 conditions

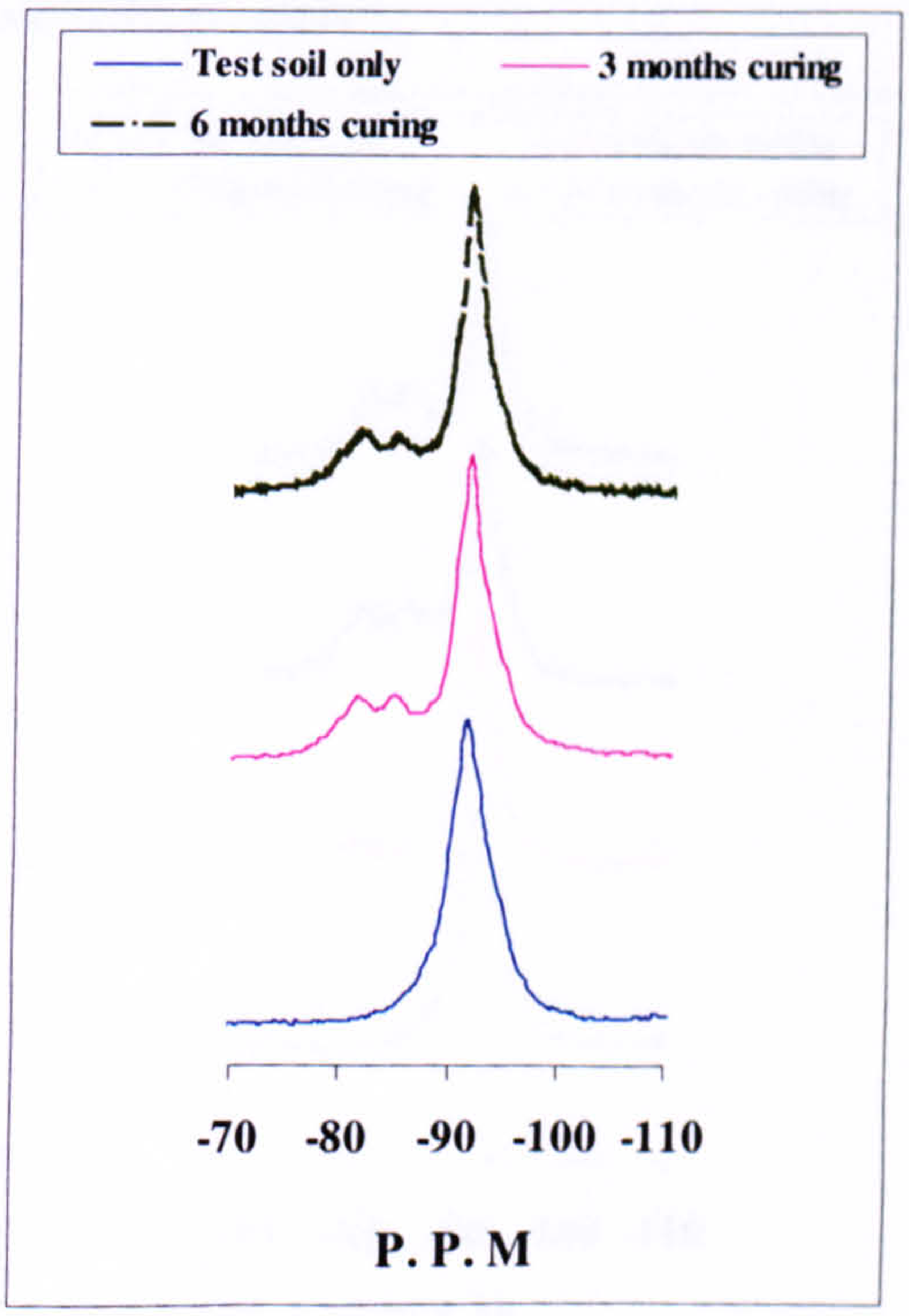


Figure 8.26 Single pulse ^{29}Si NMR spectrum for pure clay test soil + 16 % GGBS (20%Lime), cured under CC2 conditions

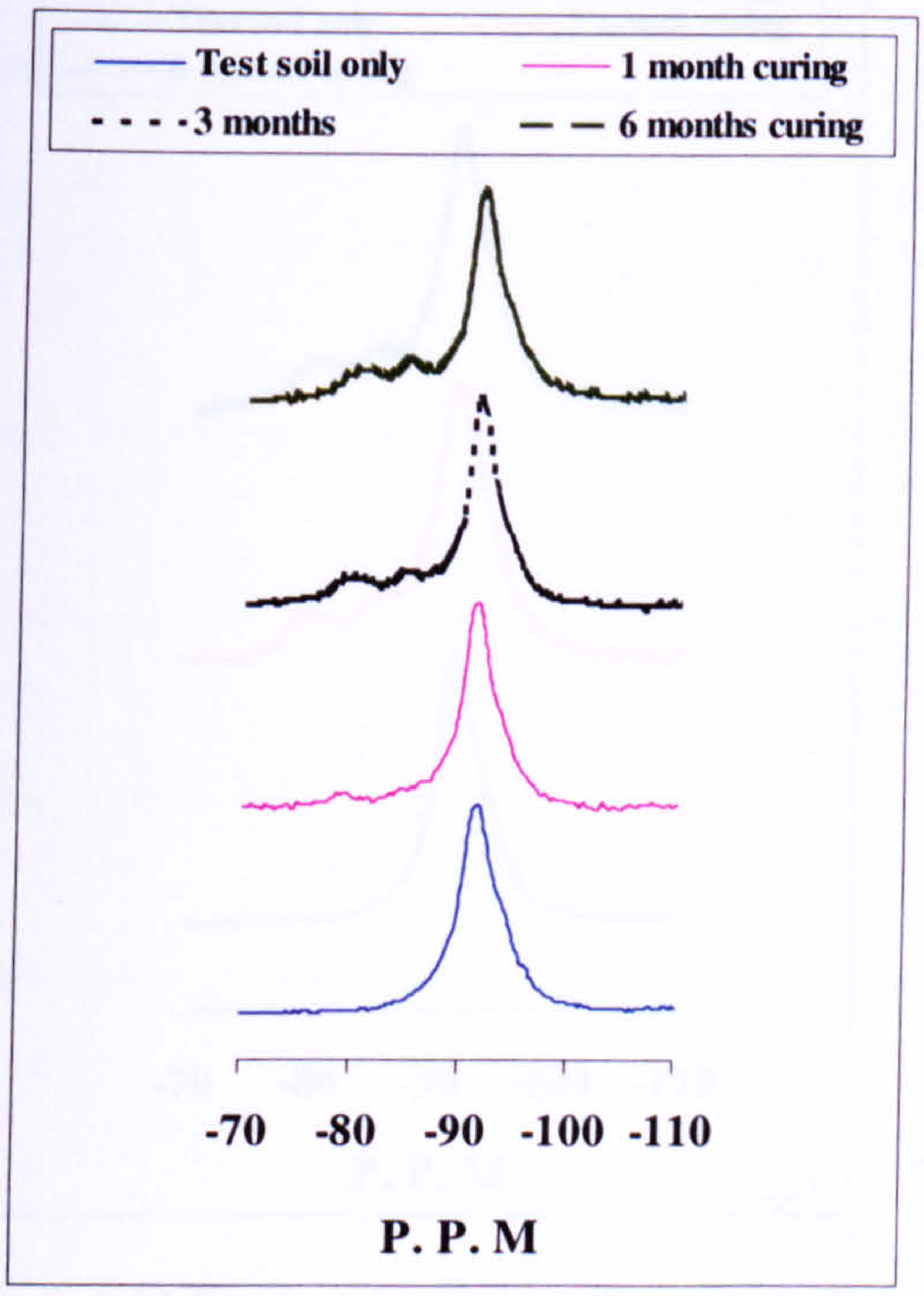


Figure 8.27 Single pulse ^{29}Si NMR spectrum for pure clay test soil + 16 % GGBS (30%Lime), cured under CC1 conditions

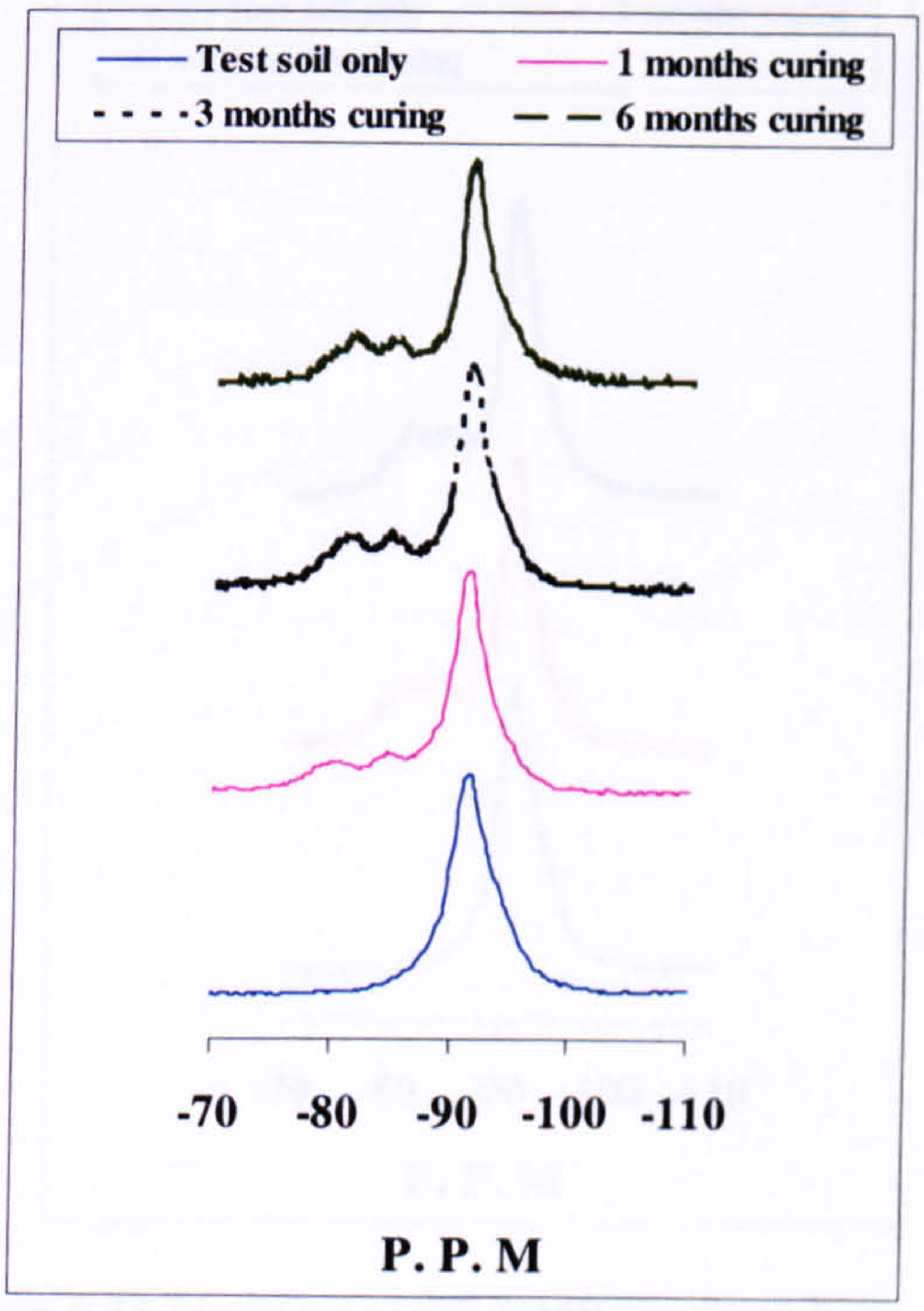


Figure 8.28 Single pulse ^{29}Si NMR spectrum for pure clay test soil + 16 % GGBS (30%Lime), cured under CC2 conditions

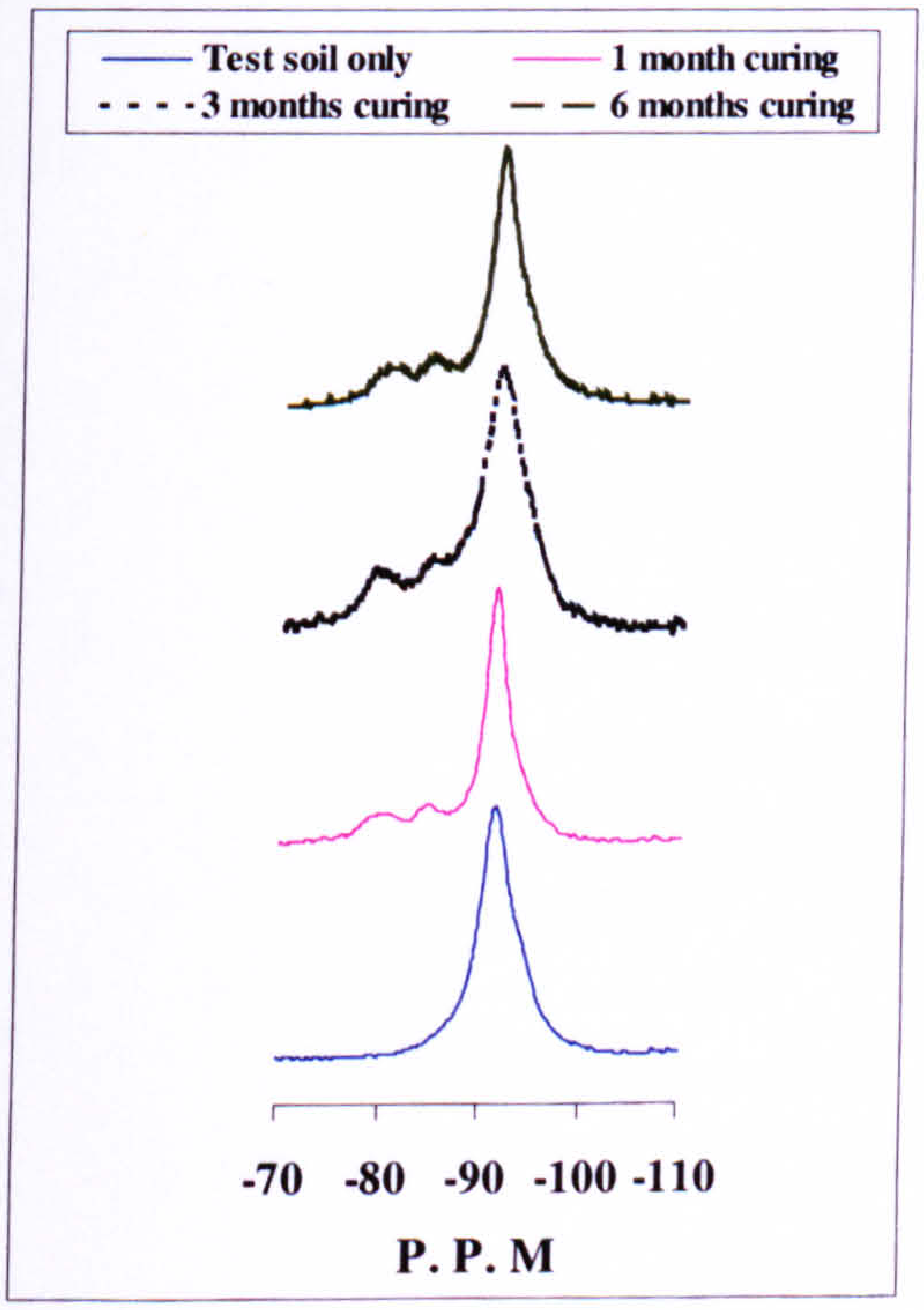


Figure 8.29 Single pulse ^{29}Si NMR spectrum for pure clay test soil + 27 % GGBS (30%Lime), cured under CC1 conditions

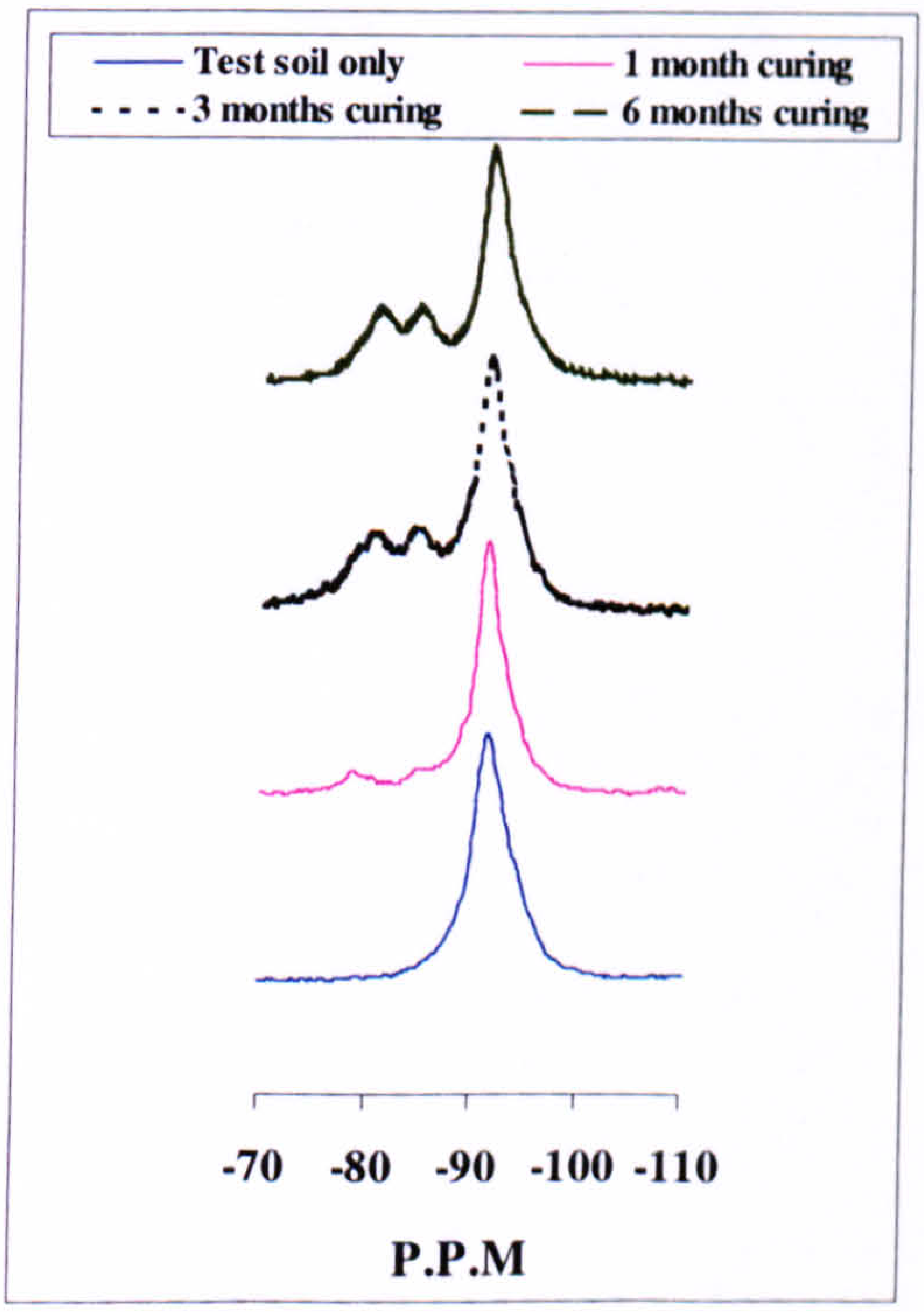


Figure 8.30 Single pulse ^{29}Si NMR spectrum for pure clay test soil + 27 % GGBS (30%Lime), cured under CC2 conditions

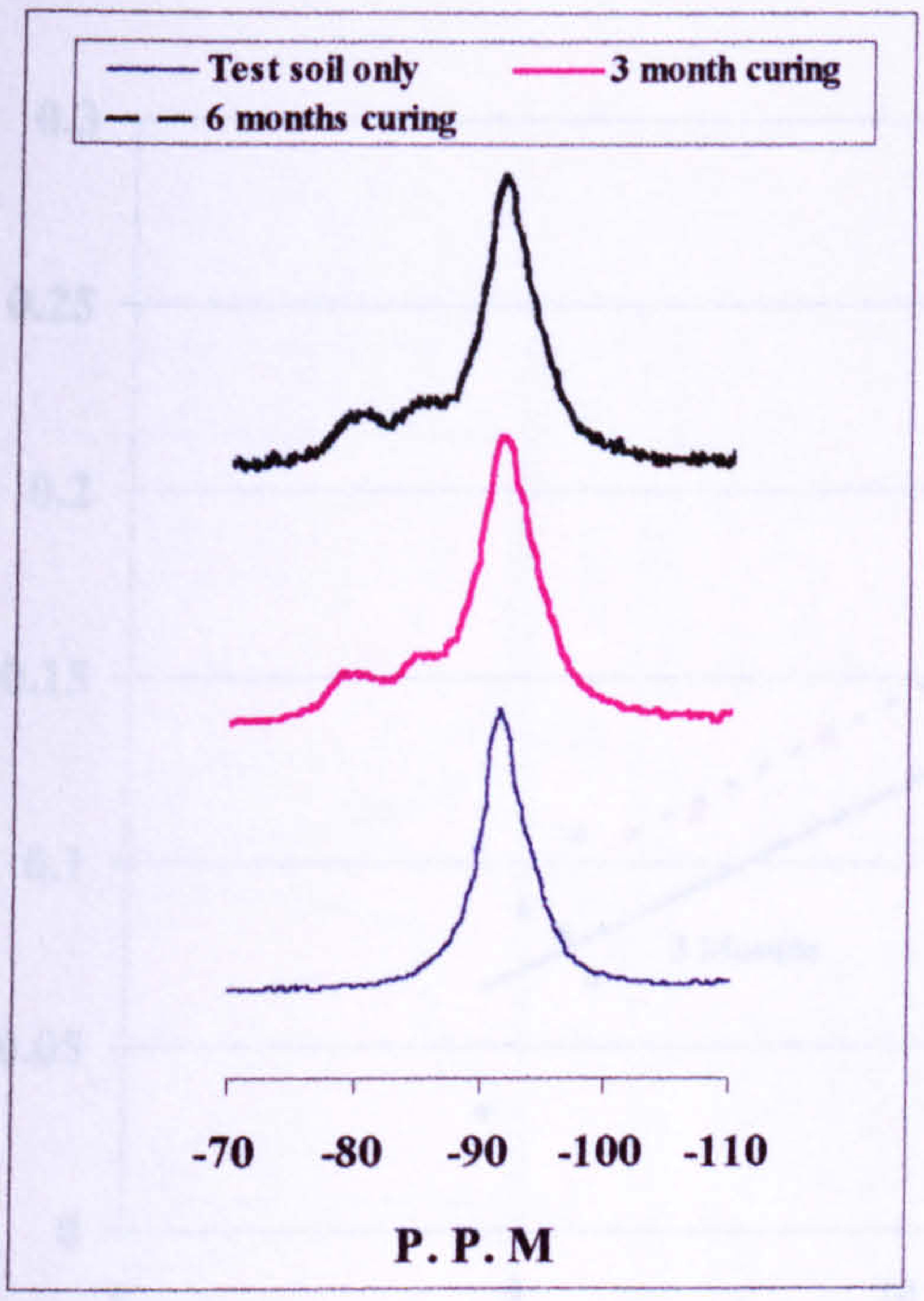


Figure 8.31 Single pulse ^{29}Si NMR spectrum for pure clay test soil + 27 % GGBS (20%Lime), cured under CC1 conditions

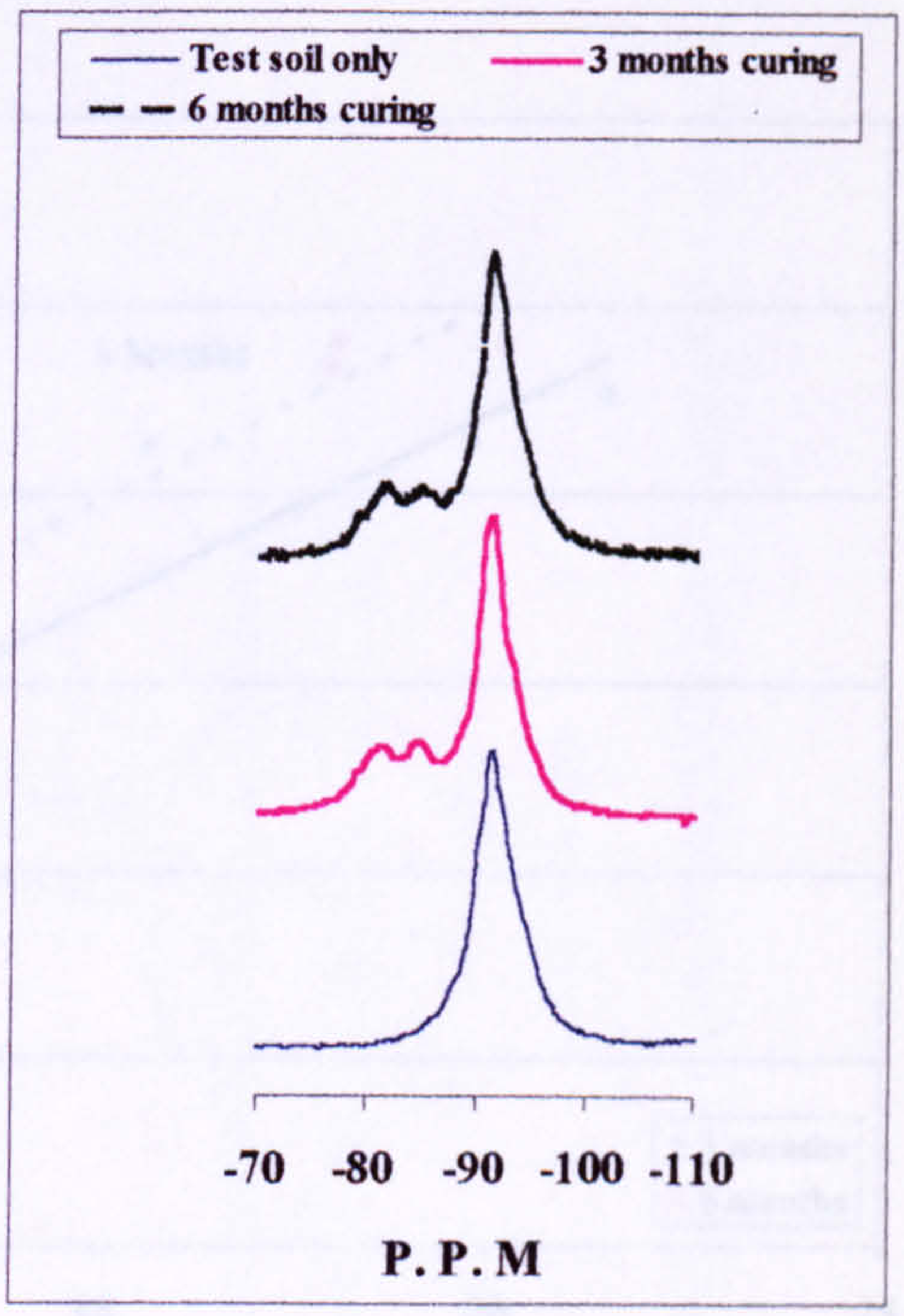


Figure 8.32 Single pulse ^{29}Si NMR spectrum for pure clay test soil + 27 % GGBS (20%Lime), cured under CC2 conditions

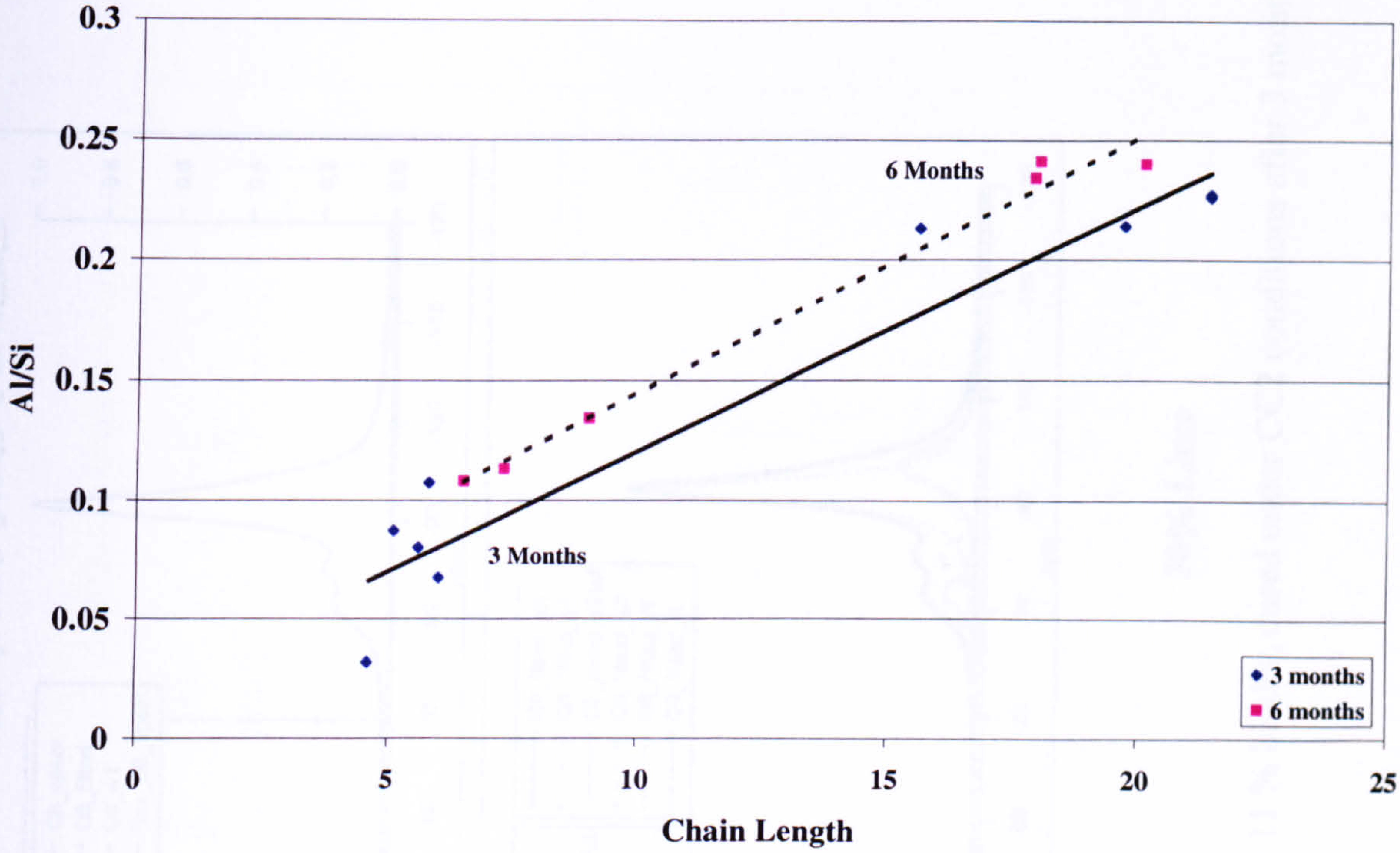


Figure 8.33 Chain length against Al/Si ratio for specimens cured for 3 and 6 months

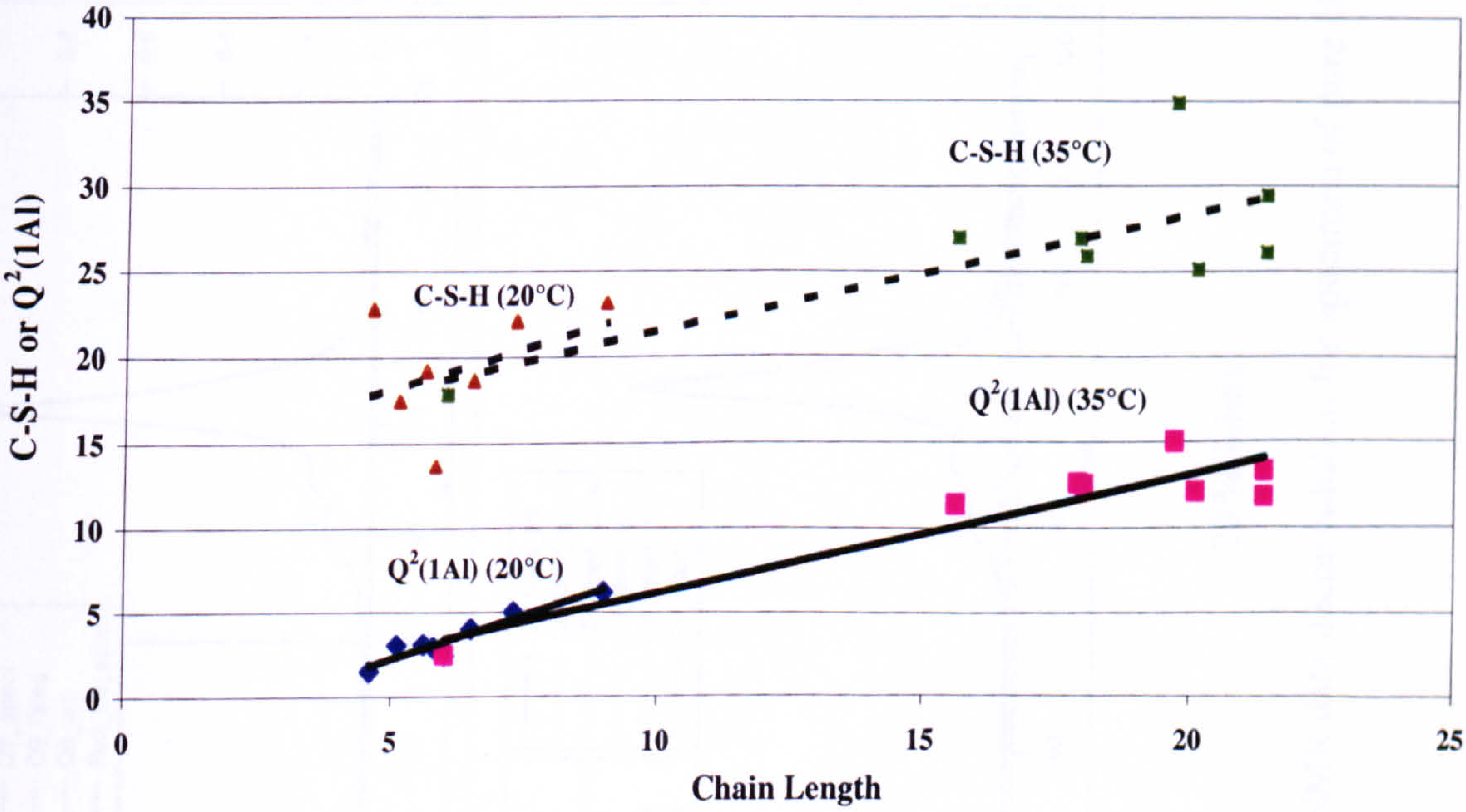
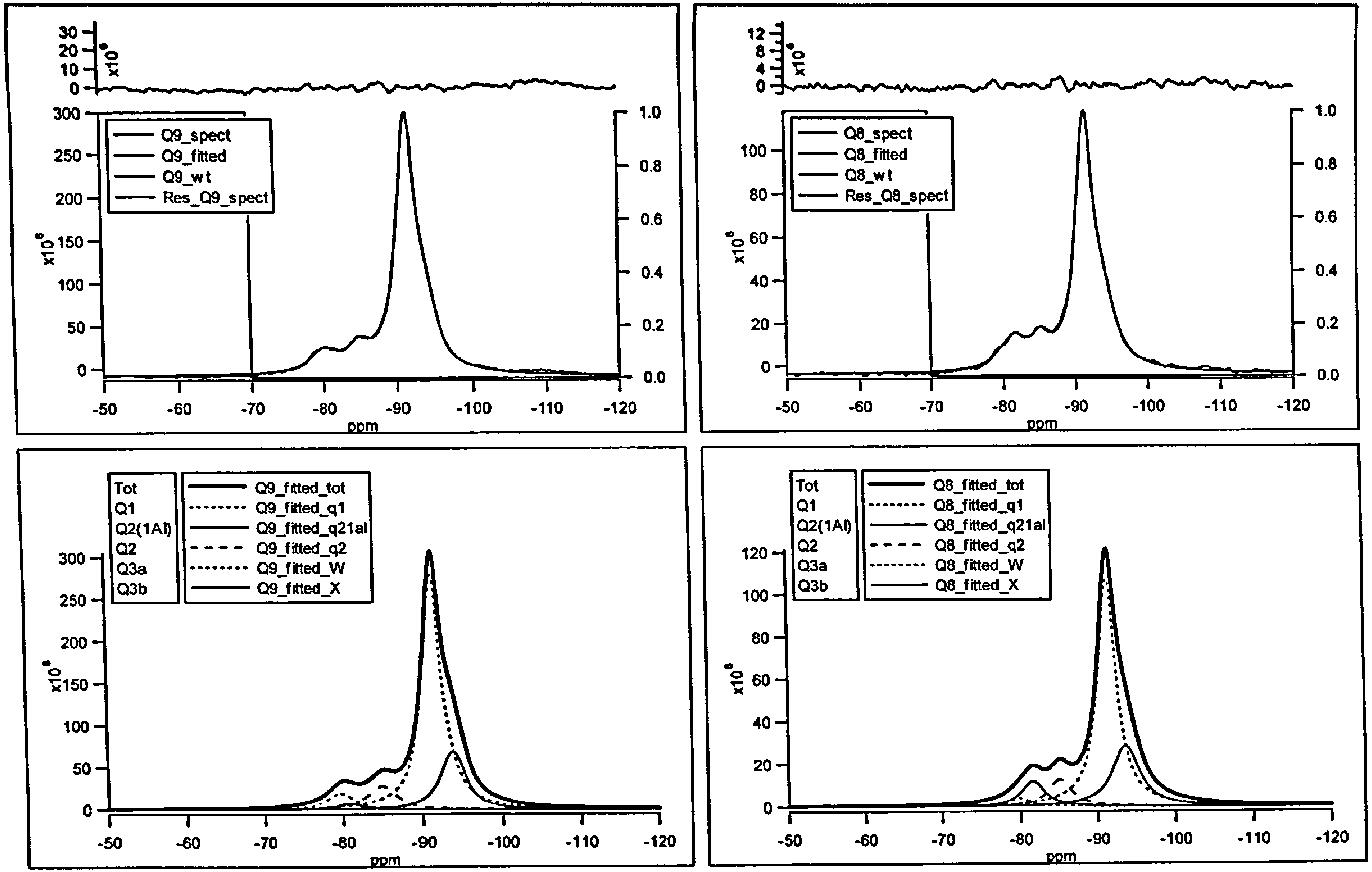


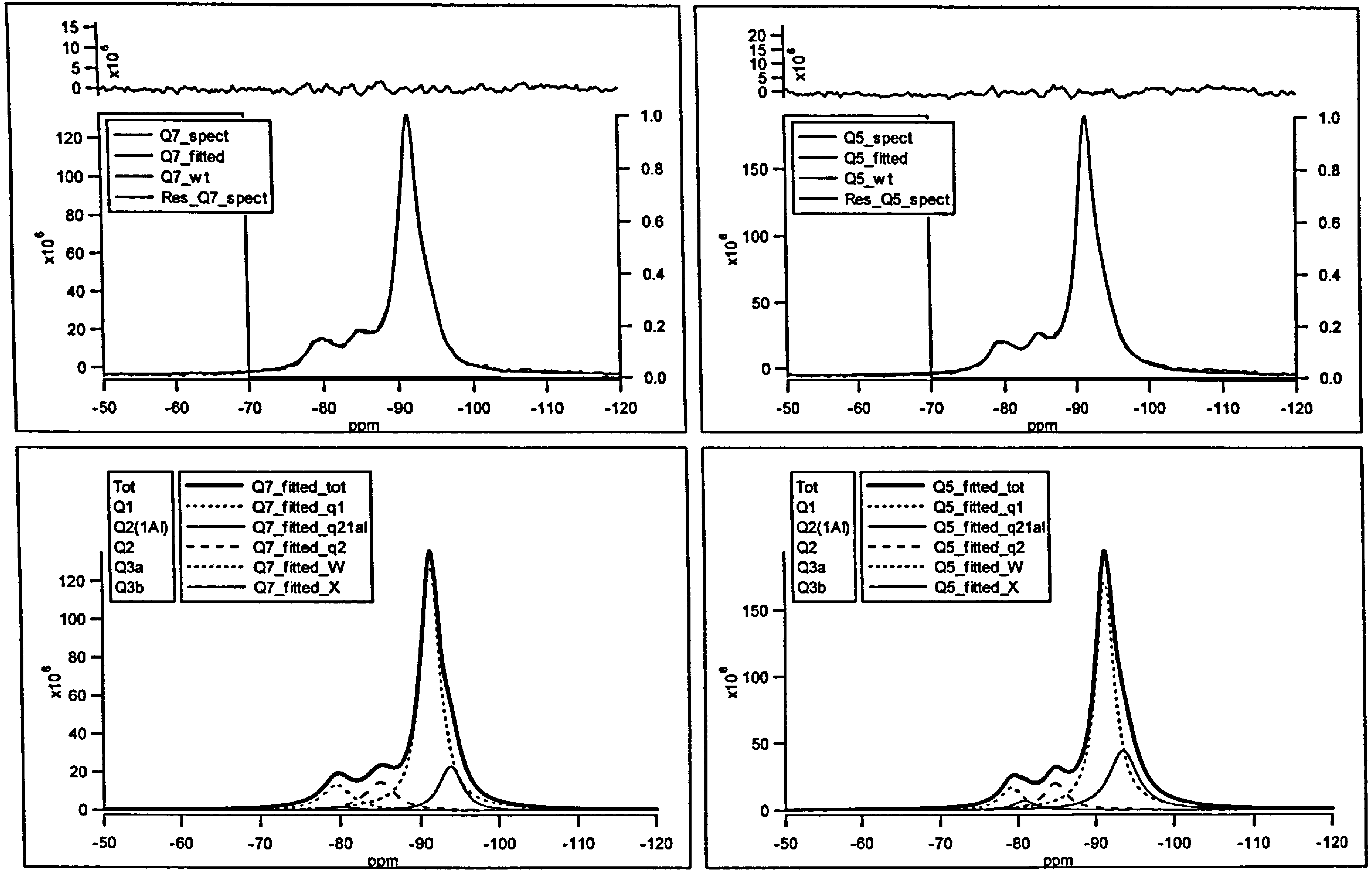
Figure 8.34 Chain length against Q^2 (1Al) and C-A-S-H percentage for specimens cured under CC1 and CC2 for 3 and 6 months



20% Lime

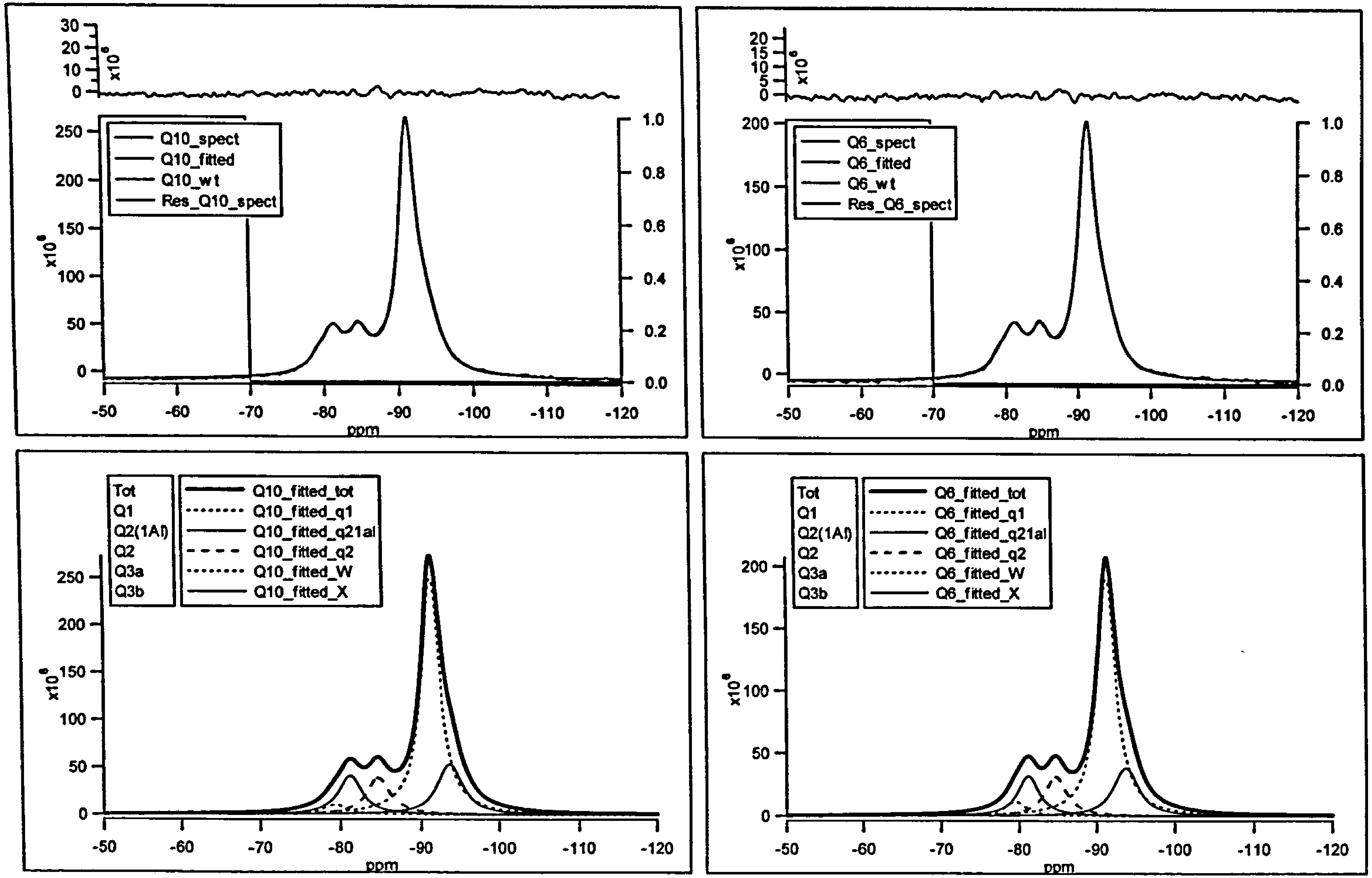
30% Lime

Figure 8.35 NMR data deconvolution for specimen of pure clay test soil + 11 % binder, cured under CC2 conditions after 3 months



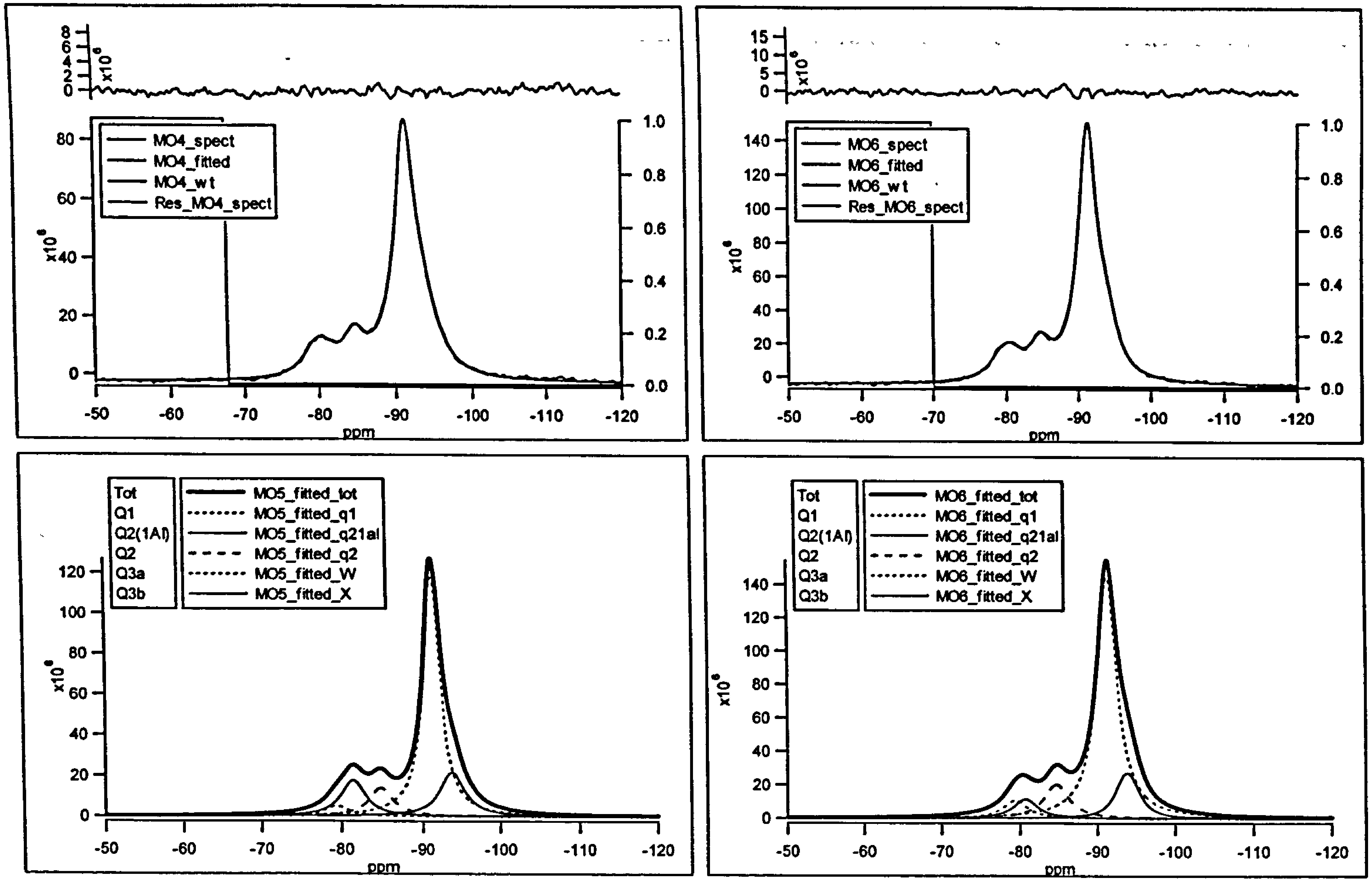
20% Lime **30% Lime**

Figure 8.36 NMR data deconvolution for specimen of pure clay test soil + 16 % binder, cured under CC1 conditions after 3 months



20% Lime **30% Lime**

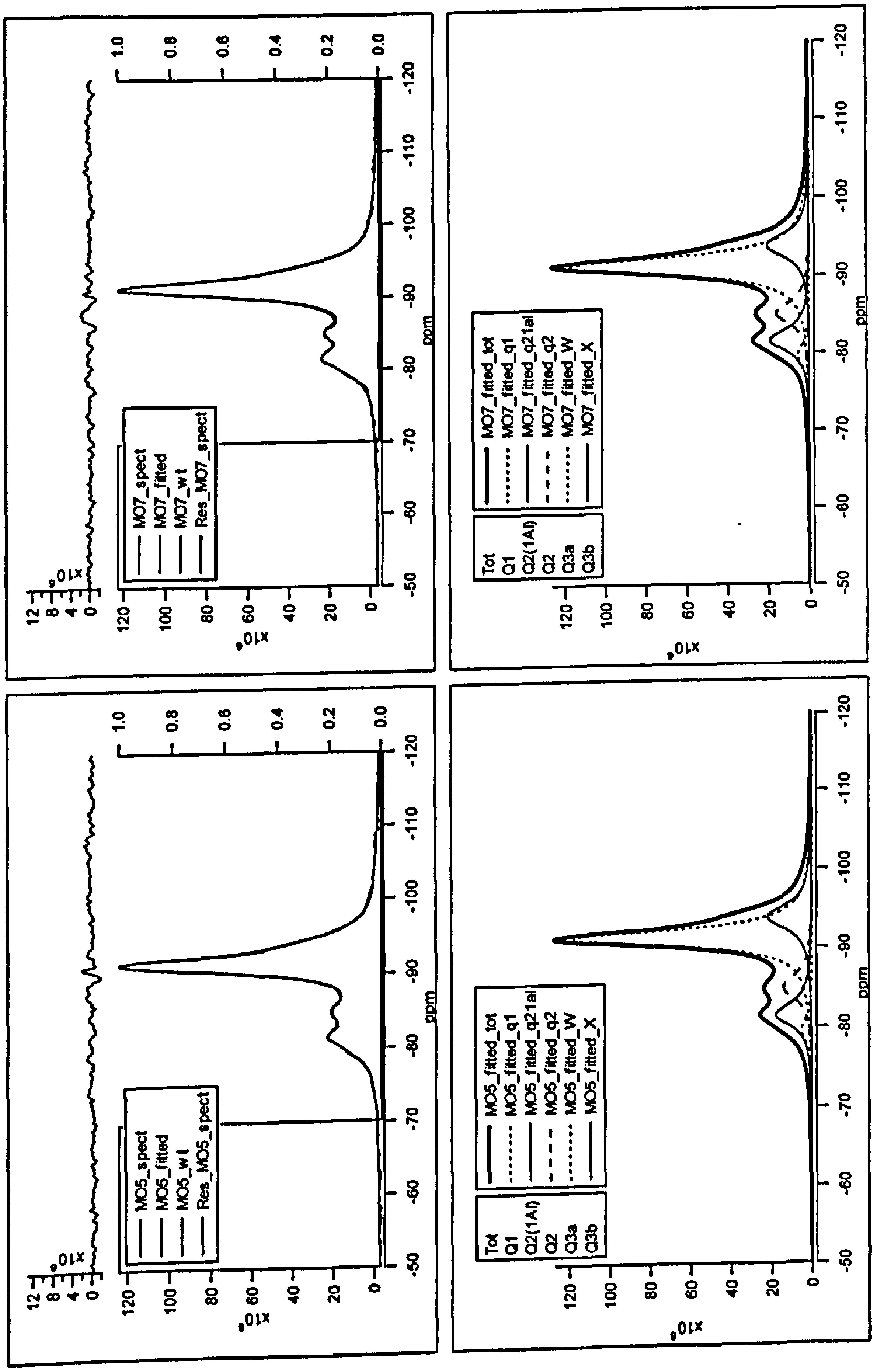
Figure 8.37 NMR data deconvolution for specimen of pure clay test soil + 16 % binder, cured under CC2 conditions after 3 months



20% Lime

30% Lime

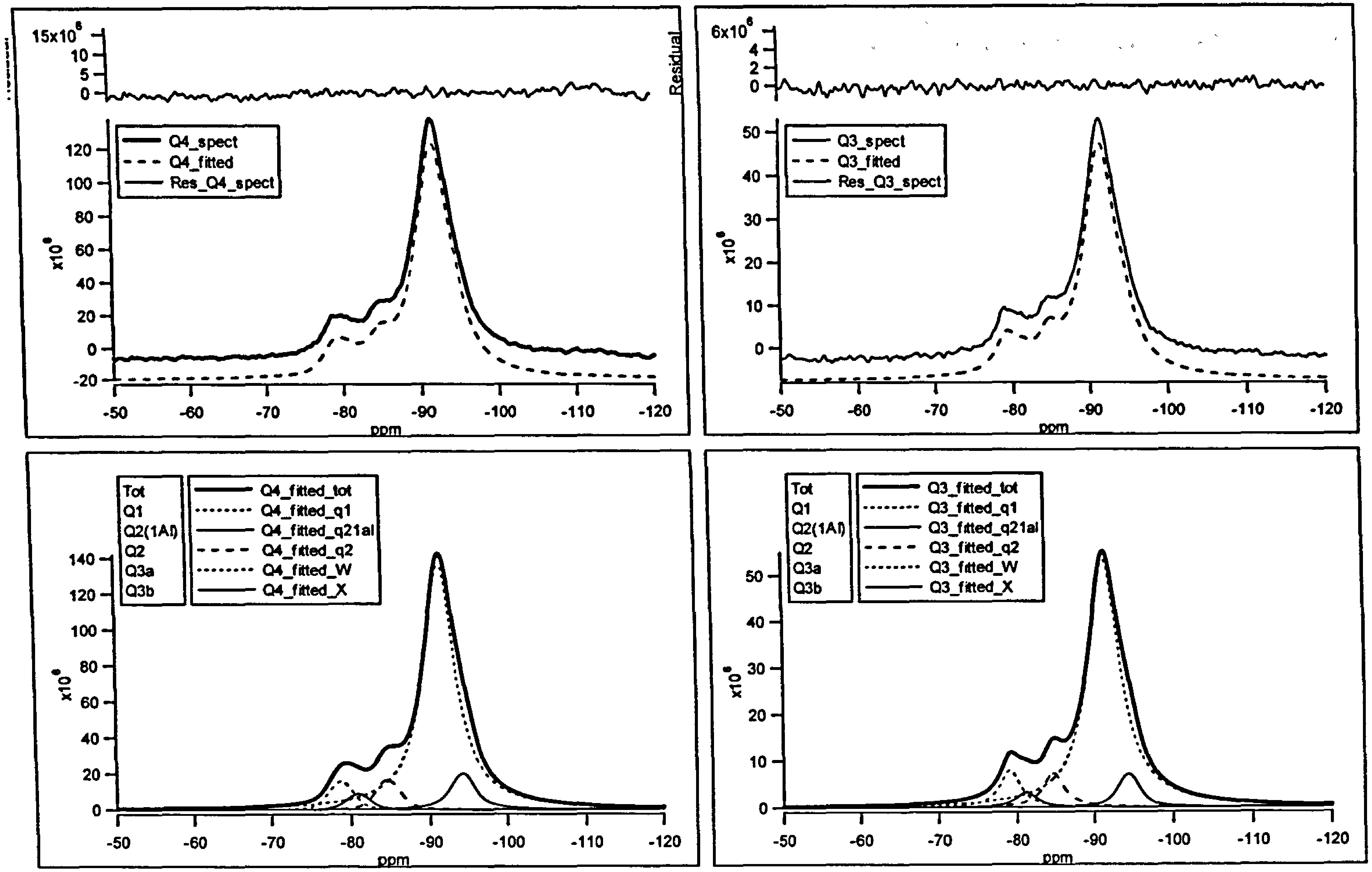
Figure 8.38 NMR data deconvolution for specimen of pure clay test soil + 16 % binder, cured under CC1 conditions after 6 months



30% Lime

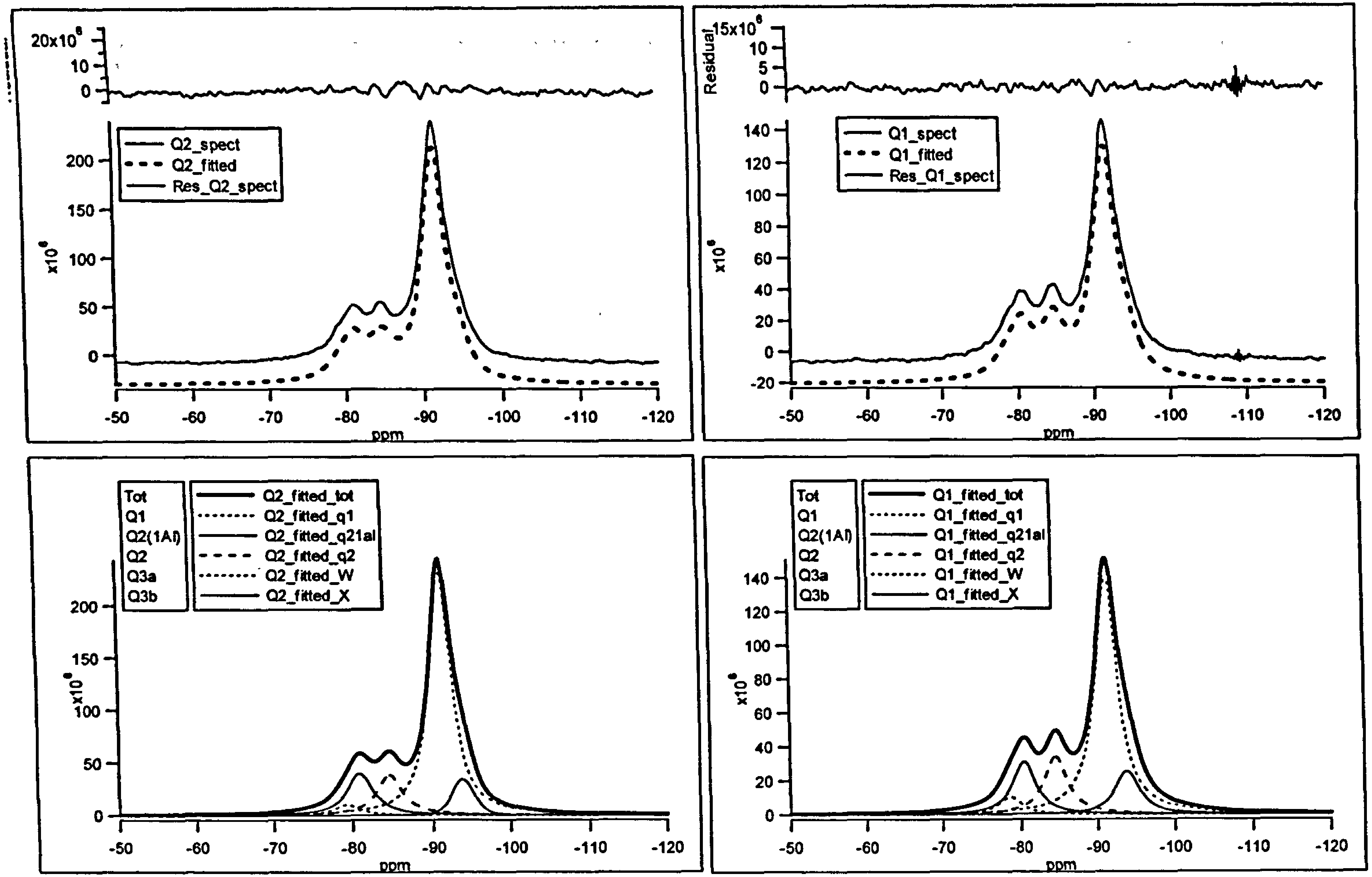
20% Lime

Figure 8.39 NMR data deconvolution for specimen of test soil + 16 % binder, cured under CC2 conditions after 6 months



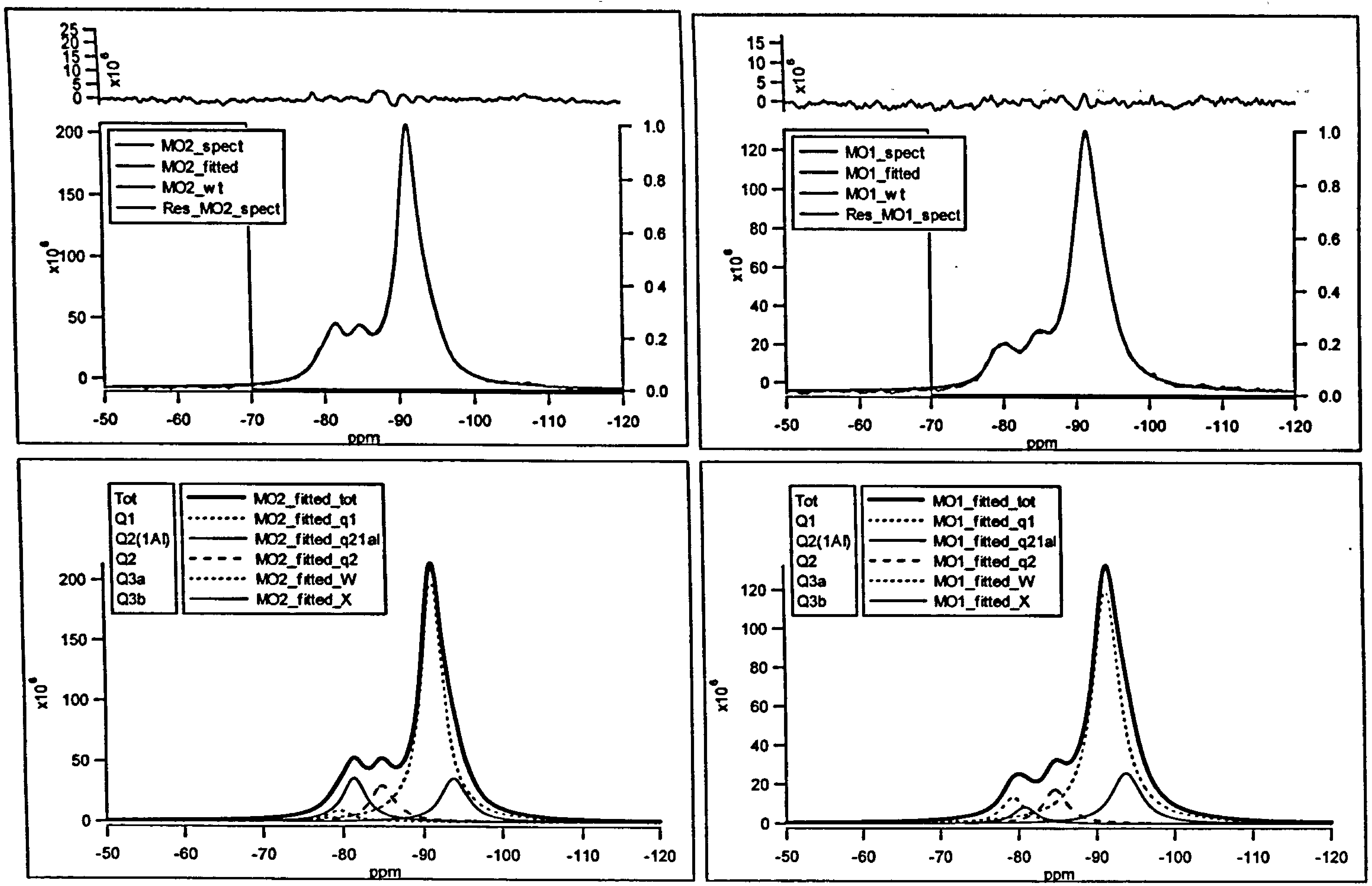
20% Lime **30% Lime**

Figure 8.40 NMR data deconvolution for specimen of pure clay test soil + 27 % binder, cured under CC1 conditions after 3 months



20% Lime **30% Lime**

Figure 8.41 NMR data deconvolution for specimen of pure clay test soil + 27 % binder, cured under CC2 conditions after 3 months



CC2 **CC1**

Figure 8.42 NMR data deconvolution for specimen of pure clay test soil + 27 % binder (20% lime) after 6 months

8.6. SUMMARY

This chapter presents and discusses data from the four analytical techniques, XRD, SEM, DTA and NMR which can be used to obtain information on the reaction products in clay-GGBS-lime system.

Although a limited number of specimens were investigated using XRD and the SEM, some useful data were obtained. XRD data showed the presence of semi-crystalline C-A-S-H gel together with C-A-H in the system. XRD data also revealed the presence of some unreacted lime in the pure clay test soil with 27% GGBS (20% lime) after 6 months under CC2 conditions. Hydrotalcite type phase is also present. The SEM data showed that the addition of GGBS and lime to the pure clay test soil caused the particles to clump together and the voids to become smaller. C-A-S-H gel covered most areas of the hydrated specimen. Calcium hydroxide and few unreacted slag particles were observed. Hydrotalcite type phase containing aluminium was also observed.

DTA data confirmed the presence of C-A-S-H gel as a major reaction product. Calcium carbonate was present in all mixture at all curing conditions and curing periods. It can be observed that about 75% of the total calcium hydroxide was consumed in the first 3 months, while about 15% was consumed in the next 6 months. Due to the presence of some calcium carbonate in the hydrated lime, it is impossible to achieve 100% lime consumption.

NMR data revealed the presence of C-A-S-H gel (Q^1, Q^2 and Q^2(1Al) structural units) which indicating that the C-A-S-H is the major reaction product. NMR data also indicate an increase in C-A-S-H gel content with an increase in curing period and curing temperature. Chain length and Al/Si ratio increases with an increase in curing period and/or curing temperature. The increase in Al/Si ratio was found to be related to the increase in Q^2(1Al) content, due to the increase in silicon substituted by aluminium in the silicate chain.

The next chapter, Discussion of Results, contains further discussion of the data and attempts to correlate and explain the engineering and analytical results.

CHAPTER NINE
DISCUSSION

9.1 INTRODUCTION

In chapter seven (Engineering Test Results), the effects of adding GGBS alone and GGBS activated by hydrated lime on various physical characteristics and engineering properties of both test and pure clay soils were reported. Significant changes in compaction, plasticity characteristics, unconfined compressive strength and swelling properties were observed with the test soil while larger effects were observed with the pure clay test soil, both on the addition of GGBS alone and with GGBS activated by lime. In chapter eight (Analytical Test Results), the results of XRD, SEM, DTA and NMR analysis of various mixes of the pure clay test soil, with GGBS and lime of the composition, mineralogy and the reaction products of clay-GGBS-lime systems were presented. Significant changes in the composition of clay soil were observed on adding GGBS activated by lime to the pure clay test soil, the degree of change depending on mix proportion, curing time and conditions.

This chapter discusses the results of the engineering and analytical tests in some detail, linking the composition and changes in properties. The changes in the clay-GGBS-lime system can be explained in terms of short-term changes in properties that take place immediately after adding GGBS and lime (between a few hours and 3 days) during the initial stabilisation process, and in terms of long-term changes in material properties that take a longer period to occur, ranging from a few days to many years.

The changes in properties can also be explained by physical and chemical changes. The physical changes include changes in moisture content, particle size distribution, plasticity and compaction characteristics. The chemical effects include flocculation, pozzolanic reactions and changes in composition and mineralogy that affect the strength and volume stability of the test soil.

9.2 EFFECT OF ADDING GGBS AND LIME ON THE COMPACTION CHARACTERISTICS OF SOILS

9.2.1 EFFECT OF THE ADDITION OF GGBS ONLY ON THE COMPACTION CHARACTERISTICS OF THE TEST SOIL

The effects of adding GGBS only on the compaction characteristics of the test soil are presented in section 7.2.2. The results are small increases in the optimum moisture content (OMC), and small decreases in the maximum dry density (MDD) with an increase in GGBS content. The increase in the OMC with an increase in the GGBS content is thought to be due to the GGBS particles being finer than soil particles, and hence, because of the greater specific surface area that has to be lubricated, soil-GGBS mixtures can be expected to require more moisture to achieve their maximum dry density than untreated soils subjected to the same compactive effort. The small decreases in the MDD corresponding to additions of GGBS is likely to be associated with the replacement of soil particles by the glassy angular shaped GGBS particles in a given volume; they partially fill the voids between these particles, prevent them from coming into a closer state of packing and leave larger volume of voids. At this stage of mixing there is unlikely to be any pozzolanic reaction between GGBS and clay soil particles as they are conducted within few hours after mixing.

9.2.2 EFFECT OF GGBS AND LIME ON THE COMPACTION CHARACTERISTICS OF THE TEST SOIL

In clay-lime systems, cation exchange normally takes place in the pore solution between the cations and the negative charged clay particles. Increase in cation concentration in the pore solution leads to neutralisation of the clay particles negative charge, (Yong and Warkentin 1975; Hilt and Davidson, 1960). Many authors have studied cation exchange in soil stabilisation by lime and they point out that a small percentage of lime is enough to produce neutralisation. The neutralisation of the clay particle negative charges causes them to more closer together thus leading to flocculation and agglomeration of the flocculated particles. Any further addition of lime has no major effect on the flocculation. Pore volume is increased due to flocculation (clay particles become bigger) and consequently the maximum dry density become smaller (Kinuthia, 1997).

Abdelkader and Hamdani (1985) in a study of soil stabilisation by lime, indicated that the decrease in the maximum dry density could also be due to the formation of cementitious products which reduce the compactibility of the treated soil and thus the dry density. The optimum moisture content also increases as a result of an increase in the pore volume due to flocculation.

In clay–GGBS-lime systems, two reactions were observed, hydration of GGBS activated by lime to produce C-A-S-H gel and hydrotalcite type phases containing magnesium, see section 8.3, plate 8.2, and clay-lime reactions producing C-A-S-H and calcium aluminate hydrates, see sections 8.2 and 8.5. The major reaction in the short-term is the hydration of GGBS activated by lime which normally starts immediately after mixing the dry materials with the required mixing water. GGBS hydration usually consumes a relatively large amount of water and a relatively small amount of lime, see figures 7.33 and 7.34. Some free lime however, is still present after GGBS hydration in the mixtures depending upon the percentage of lime added, as the required amount of lime to activate GGBS is very small, as the microscopic images in plate 8.2 showed some unreacted slag particles and lime are still present after 2 months curing, see also figures 8.5, 8.6, 8.21 and 8.22. As a result of GGBS hydration, the mixing water available for lubrication dramatically decreases and the air void content increases, and more water is needed to obtain the same level of lubrication at the same compactive effort. Therefore, the OMC increases with increasing total binder content as shown in figure 7.6.

Increasing the lime/GGBS ratio in soil mixtures causes more complicated changes in the system. Cation exchange and flocculation of the clay takes place immediately after adding mixing water but only a small amount of lime is consumed in this reaction. New cementitious materials are formed as a result of the pozzolanic reactions due to the excess amount of lime in the soil mixtures. Some researchers have observed the formation of cementitious materials immediately after mixing of water (Abdelkader and Hamdani, 1985). Wild *et al.*, (1993) pointed out the existence of cementitious materials immediately after mixing Kimmeridge clay and kaolinite with lime. The hydration of GGBS activated by lime is reported to be faster than the clay-lime reaction (Wild *et al.*, 1998). Therefore, the formation of pozzolanic materials in such systems is possible. These materials fill a relatively large part of the voids between soil particles, which therefore are more dense and hence lower voids. Increasing the lime content in the mixtures, keeping the total binder (lime + GGBS) constant, leads to an increase in the possibility that pozzolanic materials are formed immediately.

Optimum moisture content, on the other hand, increases with an increase in the lime/GGBS ratio, keeping the total binder content constant. Increasing lime content in such systems undoubtedly consumes a larger part of mixing water in hydration. Therefore, mixtures with high lime contents need more water to achieve the same level of lubrication and workability and thus the optimum moisture content is increased. However, the strength increase in the soil due to pozzolanic reactions would more than compensate for the changes in compaction parameters (decrease in maximum dry density and increase in optimum moisture content), hence overall the addition of GGBS and lime is an advantage.

The small percentages of the additives involved (GGBS and lime) compared to the bulk material (soil) and the physical replacement of GGBS by lime will not in itself result in any large variation in density. Therefore, the increase in cation exchange, flocculation and possibly the pozzolanic reaction due to an increase in lime content is more significant than the slight change in overall physical properties. It is not possible, without detailed analytical work to explain how GGBS replacement by lime influences the MDD and OMC. However, from a practical point of view, the results discussed above suggest that the partial replacement of GGBS by lime should not result in major variation in the MDD and OMC of clay soil (Kinuthia 1997).

9.3 EFFECT OF ADDING GGBS ONLY AND GGBS AND LIME ON THE PLASTICITY CHARACTERISTICS OF SOILS

9.3.1 EFFECT OF GGBS ONLY ON THE PLASTICITY CHARACTERISTICS OF TEST SOIL

The addition of GGBS to the pure clay test soil causes a small decrease in the liquid limit, a small increase in the plastic limit, and consequently reduces the plasticity index. The changes in the Atterberg limits, compared to the effect of the GGBS and lime on the plasticity behavior are relatively small, and close to the expected accuracy of the data. For example, the maximum changes in the liquid limit are 3% and 6 % in the test and pure clay test soils respectively, while the scatter of the data is approximately 1.5%, thus the changes in the liquid limit are relatively small. The changes in the plasticity index (which are 16% and 14.5%) are relatively large changes. These changes however, are considered to be due to physical replacement of clay by GGBS particles. There is unlikely to be any chemical change on the addition of GGBS only to clay soil at this early stage of reaction as the tests were carried out within 3 days.

9.3.2 EFFECT OF LIME ON THE PLASTICITY CHARACTERISTICS OF THE TEST SOIL

Two main properties probably control the plasticity of a clay soil; the plate-like structures of the clay particles and their association with the adsorbed water. The interaction between the adsorbed water of each clay platelet and the effect this has on particle interaction can bond the platelets together and the strength of this bond, depends on the thickness of the oriented water layers to a great extent. As stated in chapter two montmorillonite soils, due to their great specific surface area and electrical charge, have a higher degree of plasticity than kaolinite soils. The large electrical charge associated with montmorillonite results in a much larger capacity for water than kaolinite and consequently montmorillonite exhibits a high plasticity (Grim, 1953; Daniels, 1971).

As mentioned in Chapter 5 (Testing Materials), a clay test soil, similar to a large extent to an Egyptian clayey soil in its mechanical and physical properties and mineralogical composition, was developed and used in this research project. The main clay minerals in this clay test soil were montmorillonite and kaolinite, with minor amounts of illite and mica. Plasticity characteristics (see table 7.3), cation exchange capacity and specific surface area of the test soil are considered to be approximately an average between kaolinite and montmorillonite.

When a small amount of hydrated lime is added to the clay test soil, it is thought to create linkages between the clay particles and not to enter into any cation exchange reaction. Increasing the lime content causes cation exchange to start with a direct effect on the plasticity characteristics of the clay soil. Also, cation exchange may start on the addition of a small amount of lime to the clay soil after a period of mellowing (e. g. 3 days). Clay particles are normally surrounded by a diffuse hydrous double layer which is modified by the ion exchange of, in this case, calcium. This alters the density of the electrical charge around the clay particles which causes attraction between clay particles to change and form silt size particles (flocs). This flocculation process is mainly responsible for the modification of the plasticity behaviour of clay soils when they are treated with lime (Sherwood, 1993).

Addition of lime to the clay test soil leads to a slight increase in the liquid limit with increasing lime content up to 4% of lime by dry soil weight. The initial rise in the liquid limit is probably due to the lime firstly creating linkage between the clay particles and not actually entering into any cation exchange reaction before mellowing. This increase in the liquid limit would finish after 3 days maturing as this period of (3 days) is probably enough for all calcium cations to get involved in the cation exchange and to start forming flocs. Increasing the lime content in the mixture could increase the cation exchange and the formation of flocs, thus the liquid limit decreases to its minimum value of 94%. The liquid limit decreased further over 3 days, as this mellowing period may be enough to get the clay particles more saturated with lime cations and more flocs could be formed even with a relatively low lime content. The flocculated nature of the soil-lime mixture also causes a reduction in the liquid limit of the test soil by creating relatively weak bonds between the "flocs", therefore, less water is required to lubricate them to the stage where the required movement would occur with the liquid limit shearing action. Addition of lime to the clay test soil increases the plastic limit to a maximum value at a lime content of 4%. The increase in the plastic limit is probably caused by water being held not only by the electrostatic forces on the clay mineral surfaces but also by capillary forces inside the newly flocculated clay structures (Daniels, 1971). The water thus held would, therefore, not be available to create the necessary bonding force between the individual "flocs" thus increasing the plastic limit of the clay-lime mixtures. Further increases in lime content has no effect on the plastic limit due to the saturation of the system with calcium ions.

Due to the decrease in the liquid limit and the increase in plastic limit, the plasticity index dramatically decreases with an increase in lime content up to 4%, at the lime fixation point, then the plasticity index slightly decreases up to a content of 10 %. Further decrease was observed in the plasticity index over 3 days mellowing due to the decrease in liquid limit described above.

9.3.3 EFFECT OF GGBS AND LIME ON THE PLASTICITY CHARACTERISTICS OF TEST SOIL

Clay-GGBS-lime systems are similar to some extent to clay-lime systems, especially in the short -term. When GGBS and lime are added to the clay soil, GGBS hydration starts to take place, as it is a relatively rapid reaction (Meng *et al.*, 1998). This reaction consumes a part of the lime (Wild and Tasong, 1999) and dries out part of the mixing water. A small percentage of lime is enough to activate the GGBS (between 10 and 25% of the total binder depending on the type of soil), and the rest of lime is used for cation exchange and the flocculation of the clay particles.

An initial rise in the liquid limit was observed with increasing binder content up to between 2 to 4% binder. This increase in liquid limit is probably due to the insufficient lime content (0.2 -1.2 % by dry weight of soil) to create any cation exchange and only creating linkage between the clay particles (see section 9.3.2).

The increase in the liquid limit becomes a maximum at lower lime content (2% binder, 10% lime) and becomes a minimum at maximum lime content (4% binder, 30 % lime), as the cation exchange needs a sufficient amount of lime to start. The rise in liquid limit is also reduced after 3 days mellowing as this period of mellowing may be enough for calcium cations to get involved in cation exchange and start forming flocs. Increasing binder content causes a further decrease in liquid limit as the lime content increases and more flocs would therefore be formed. The flocculated nature of the soil-GGBS-lime system causes a reduction in the liquid limit of the test soil by increasing relatively weak bonds between the "flocs", and therefore, less water is required to lubricate them to the stage where the required movement would occur with the liquid limit shearing action.

The addition of GGBS activated by lime to the clay test soil increases the plastic limit to a maximum value at a binder content of about 4%. The increase in plastic limit is probably due to the reasons outlined above. It was also observed that the values of the plastic limit in the case of GGBS activated by lime are lower than that for lime alone at the same binder content due to a small amount of lime being consumed in GGBS hydration. Further increases in binder content have no effect on the plastic limit due to the reasons outlined above in the case of the addition of lime only.

The plasticity index also decreases due to the decrease in the liquid limit and the increase in plastic limit

9.4 EFFECT OF ADDING GGBS ONLY AND GGBS AND LIME ON THE STRENGTH PROPERTIES OF SOILS

9.4.1 EFFECT OF GGBS ONLY ON STRENGTH PROPERTIES OF TEST SOIL

The UCS of the test soil significantly increased with an increase in GGBS under certain conditions. The behaviour can be divided into two categories; firstly, a short initial period up to 28 days curing, and secondly, long term changes, up to 12 months curing. There is no significant change in the UCS up to 7 days. However, from 7 to

28 days, the UCS slightly increased with increasing GGBS percent up to 6 % and then slightly decreased with further increase in GGBS content. Although care was taken to maintain constant water content during the curing periods, the moisture content generally decreased slightly. The moisture content decreased during the curing periods by about 5% and 15% of the initial value after 3 and 12 months curing respectively. These changes varied depending on the curing temperature and GGBS content, see figures 7.24 and 7.25. The increase in the UCS was due to an increase in pore water suction due to a decrease in the moisture content (figure 7.26) and also some changes due to pozzolanic reactions and the formation of small amount of C-A-S-H especially at longer curing period and/or under CC2 conditions. Wild *et al.*, (1998) suggested that GGBS could be activated in some cases by some components in natural soils. They observed an increase in the UCS of the Kimmeridge clay with addition of GGBS only. The test soil comprised 20% calcium montmorillonite, which contained 2.23 % calcium oxide, and 80% River Aire soil contained 6.22 % calcium oxide (see table 5.3), thus the test soil contained approximately 5.42 % calcium oxide. This implies that the test soil was activated using calcium oxide originally found in the natural test soil without any additional alkali. A small amount of C-A-S-H was observed on adding GGBS only to the test soil after 2 months, cured at CC1, see figure 8.20. The small decrease in the UCS with increasing GGBS content beyond 6 % is thought to be mainly due to the decrease in maximum dry density (≥ 4% of the MDD of the test soil).

The UCS of the test soil increased with an increase in the GGBS content after 3, 6, 9 and 12 months (see figures 7.16 to 7.23). This long-term increase in the UCS it is thought mainly a result of increasing amounts of C-A-S-H due to longer curing periods, also, a result of the increase in pore water suction due to the decrease in the moisture content (see figure 7.26). It is well established from the literature, and the results of the DTA and NMR analysis, see figures 8. 6 to 8.17 and figures 8.21 to 8.30, that the possibility of forming a large amount of cementitious materials is higher at high curing temperature than at low curing temperature. The amount and the properties (e.g, $\overline{CL}$, Al/Si ratio) of the aluminum substituted C-A-S-H formed are directly proportional to the curing temperature and in most cases with the curing period, which has a direct effect on the increase in the UCS (see sections 8.4.3 and 8.5.2). Also the effect of pore water suction on the UCS was higher at high curing temperature than at low curing temperature due to the decrease in the water content, (see figures 7.24 and 7.25). The effect of the decrease in the MDD on the UCS was minor compared to the other factors described above.

9.4.2 EFFECT OF GGBS AND LIME ON THE STRENGTH PROPERTIES OF TEST SOIL

Previous research on soil stabilisation and pozzolanic reactions have showed that the type, form, amount and characteristics of the reaction products control the physical, chemical and mechanical properties of the bulk material after stabilisation. Thus, the nature of the long-term cementation in clay stabilisation will, depending on the effect of the curing conditions and time, determine and control amongst other factors (e g compaction) the strength of the mixtures. The primary cementing agent in all clay lime stabilisation systems is calcium alumino silicate hydrate (C-A-S-H) gel. The pore solution of these systems contain silicate and aluminate ions which are formed by the dissolution of clay particles in the highly alkaline environment provided by the dissolved lime (Kinuthia, 1997). In clay-GGBS-lime systems, the primary cementing agent is still C-A-S-H gel. Due to the high alumina content of GGBS, some alumina is expected to replace silica and C-S-A-H gel is also formed. X- ray diffraction results also showed the formation of crystalline calcium aluminate hydrate.

In the current work, the test results reveal an increase in the UCS with increasing total binder content, with increasing GGBS replacement by lime up to 30% replacement, with increasing curing temperature, and with increasing curing period. Addition of 2 % binder to the test soil had no significant effect on the test soil up to 28 days curing for all percentages of GGBS replacement by lime at the low curing temperature. In contrast the same amount of binder resulted in about 10 % increase in the UCS after 28 days at the higher curing temperature with 30 % GGBS replacement by lime. In studies of clay soil stabilisation by lime, it has been observed that there is a minimum amount of lime that is required for the full modification (cation exchange, flocculation and agglomeration, change in plasticity) and to start the pozzolanic reactions (see section 2.9.5.3). This amount of lime depends mainly upon clay content and clay type. This lime content ranges between 1 % and 3 % by weight of dry soil. In the current study a minimum amount of binder was required for full clay modification, before starting the pozzolanic reaction. From the pH study results, this minimum amount of binder to cause full modification, of the test soil was found to be 4% GGBS activated by 20% or 30% lime. These values were sufficient to increase the pH to 12.25 which is very close to the pH of a saturated lime solution. Therefore, 2% binder activated by 20% or 30% lime had, in theory small chemical effect on the test soil and thus no cementitious materials would be expected. The main reason for the small increase in the strength in this case is thought to be an increase in the pore water suction due to the small decrease in the mixing moisture content (see figures 7.33 and 7.34).

At the higher binder content, 4% and more, the pH increased to 12.25 and above, therefore the effect of pore water suction and the effect of the decrease in the MDD on the strength were considered to be minor compared to the effect of pozzolanic reactions. Therefore, the main reason for the increase in the strength was the pozzolanic reaction products due to the GGBS activated by lime and the clay-lime reactions.

When GGBS is exposed to water a Si-Al-O rich layer forms on the GGBS particle surfaces. This layer may absorb H^+ from water, resulting in an increase in OH^- concentration and then the pH of the solution also increases to values close to the pH of a saturated lime solution (Caijun and Day, 1993). At these high values of pH, Si-O and Al-O bonds are broken and then semi-crystalline C-A-S-H, crystalline calcium aluminate hydrate and C-A-S-H are formed. DTA and NMR test results confirmed the presence of the C-A-S-H in all mixtures. These tests also showed an increase in the cementitious product content with an increase in lime content, curing temperature and curing periods, with an increase in the lime consumption with increasing curing periods as GGBS hydration consumes lime and produce C-A-S-H, and C-A-H.

Increasing the binder content to 6 % caused dramatic increases in the strength for the mixes with GGBS replacement by lime of 20 % or 30%. This increase in strength is due to the increase in the cementitious product contents. Wild *et al.*, (1998) observed an increase in strength with increasing lime/GGBS ratio up to a peak value and then a gradual decrease with increasing lime/GGBS ratio. In the current work, however, an increase in lime /GGBS ratio up to 30% caused an increase in the strength of the test soil. No further replacement of GGBS by lime beyond 30% was investigated. Therefore, a peak was not seen. The optimum lime/GGBS ratio to produce maximum strength, observed by Higgins *et al.*, (1998) and Wild *et al.*, (1998) in the author's opinion depends mainly upon soil type, clay content, and to a some extent on curing conditions and curing periods.

NMR results showed an increase in the C-A-S-H content, and in the average C-A-S-H aluminosilicate chain length $\overline{CL}$, with an increase in curing temperature and curing period (see section 8.5). The increase in the C-A-S-H content implies an increase in the strength, while the increase in the chain length produces lower Ca/Si ratios for C-A-S-H, thus increasing the degree of crystallinity and homogeneity of the C-A-S-H. The increase in the crystallinity and the homogeneity of the C-A-S-H, it is thought, implies more stable reaction products and consequently an increase in the UCS. Although many factors determine the characteristics of clay-GGBS-lime mixtures and the cementitious product content, the clay content and type, mixing moisture content, curing period and conditions and the lime/GGBS ratio are of particular importance. These factors determine the amount and properties of the reaction products formed and thus affect the UCS of the soil under test.

XRD test results showed the presence of crystalline calcium aluminate hydrate (C-A-H) together with a semi-crystalline (C-A-S-H) as major reaction products (figure 8.1). The DTA test results confirmed the presence of the C-A-S-H. The results also showed that with an increase in the curing period and temperature (see section 8.4.3) the C-S-H content increased, together with the calcium hydroxide consumed, see figures 8.21 and 8.22. The increase in the C-S-H content implies an increase in the

UCS of the test soil, while the decrease in the calcium hydroxide indicates the consumption of lime by activation of the GGBS and in the formation of the C-S-H.

Scanning electron microscope images generally showed that the porosity of the hydrated specimen (pure clay test soil and 27% GGBS (20% lime), cured under CC2 for 2 months) is less than the test specimen (pure clay soil only) and that the C-A-S-H is uniformly spread in the specimens and coated the clay particles. This leads to a more stable and less porous system. Large areas of resin in the microscopy images of the clay test soil indicate more voids. The specimens were freeze-dried in liquid nitrogen prior to SEM analysis. The freeze-drying causes shrinkage of the expansive clay in the test soil, thus increasing the voids in test soil, while no indication was observed of any shrinkage in the hydrated specimen. Hydrotalcite type phase was observed in the hydrated specimen and a few particles of unreacted GGBS were also found (see plates 8.1 and 8.2).

Bell (1996) observed that the strength developed in clay-lime mixtures is influenced by the quantity of cementitious gel produced and consequently on the amount of lime consumed. Extended curing periods and elevated temperatures promote pozzolanic reactions. For instance, significantly improved strength can be developed with relatively small increases in temperature. Conversely, if the temperature is very low, pozzolanic reaction may cease. Similar behaviour was found in the clay-GGBS-lime system; extended curing periods and elevated curing temperatures promoted the pozzolanic reactions and the quantity of cementitious gel produced was influenced by the factors outlined above, based also on NMR and DTA data.

It was suggested by McDowell (1966) that mixtures of lime-stabilised clay should be designed to have strengths of 345 kN/m^2 and 690 kN/m^2 for sub-bases and bases respectively. In terms of these criteria all mixtures satisfy the requirement of bases even after 7 days curing under both curing conditions. Odier *et al.*, (1971) however, proposed guiding values of between 345 kN/m^2 and 1700 kN/m^2 for base construction in low cost roads. The design parameters should be decided both on cost and performance, including the total binder and lime content to achieve an acceptable strength value. Based on Odier *et al.*, recommendations, 4 % binder with

20 % GGBS replacement by lime, cured at CC2 after 28 days could satisfy the maximum recommended strength values for bases. The mixture should also consider environmental conditions, availability of materials and the time before traffic loading (maximum curing period).

Some authors recommended a study of the effect of moisture on the stabilised materials by testing specimens after immersion in water. Andrews (1966) suggested a loss of 10 to 60 percent in the 21 day unconfined compressive strength could occur after 24 hours immersion in water for clay-lime mixtures. In the current work the effect of immersion has not been studied as this investigation was with reference to a field environment in which a soaked condition is not anticipated.

9.5 THE EFFECT OF ADDING GGBS ONLY AND GGBS AND LIME ON THE SWELLING CHARACTERISTICS OF THE TEST SOIL

9.5.1 INTRODUCTION

Serious problems have been observed on structures founded on expansive clayey soils due to changes in the moisture content because of the unpredictable movements of the structures. Volume change and swelling pressures in clayey soils depend on clay type, pore volume, pore solution (including salt concentration) and particle-to-particle forces (Xidakis, 1979; Mowafy *et al.*, 1990). The volume change and swelling pressure are closely related and estimation of the volume change of soil gives an indication of its swelling pressure and vice versa. In chemical soil stabilisation processes, using lime or GGBS activated by lime, it has been established that new cementitious materials are formed and alter the particle-to-particle forces through the cementitious effects that these reaction products impart. Due to the formation of these cementitious materials, the pore fluid and pore pressure, will then be decreased, thus reducing the swelling pressure and the swelling potential and leading to volume stability (Kinuthia, 1997).

9.5.2 THE EXPANSIVE PROCESS

In the current work, the swelling potential due to the change in the moisture content only has been studied. Two processes are involved in the swelling of the clay solids: 1) enlargement of capillary films coating clay particles leads to a relaxation of effective compressive strength and 2) osmotic imbibition of water by clay minerals, especially minerals with expanding characteristics such as montmorillonite (Mielenez and King, 1955). Abdi (1992) demonstrated that the average thickness of the water layer around each clay platelet increases linearly with an increase in moisture content, which means that expansion has a linear relationship with moisture increase. The process is reversible. The problem of volume change of a clay under a building becomes serious when the change in moisture content is nonhomogeneous, due, for example to a water leakage from water main or nonhomogeneous changes in the ground water table. Thus unpredictable volume change occurs. The reactions of the binder with clay must, therefore, modify the clay particles surfaces in such a way that the water absorption and swelling capacity is reduced. To understand the mechanism involved in reducing swelling it was therefore important to establish what changes occur at the clay particles surfaces during stabilisation.

9.5.3 EFFECT OF GGBS AND LIME ON THE SWELLING POTENTIAL OF THE TEST SOIL

Swelling test results have shown that the addition of GGBS activated by lime significantly reduced the swelling potential of the test soil. The swelling percent decreases with an increase in the total binder content, lime/GGBS ratio, curing period and the elevated curing temperature To explain the effect of all these factors on the free swelling percent, analytical tests (XRD, SEM, DTA and NMR) were carried out on the pure clay test soil and some other samples composed of pure clay test soil, GGBS and lime to identify the reaction products formed during curing.

From the analytical tests it was obvious that when GGBS activated by lime was added to the clay soil, the principal reaction products formed are C-A-H and C-A-S-H. SEM images suggest that C-A-S-H gel envelops the surfaces of the clay plates. Therefore, the electrical double layer of the clay plates will be damaged. The capillary pores between particles are then filled with a solution of calcium hydroxide containing silicate and aluminate ions and this solution will gradually reach equilibrium with water within the gel pores. There will be no large ion concentration generated to drive an osmotic process and the water absorption and swelling capacity of the clay will be suppressed (Abdi, 1992).

The reduction in swelling of hydrated specimens results from the enhanced volume stability due to form the cementitious materials. These cementitious products bind the clay particles together preventing them from being expanded. Increasing lime/GGBS ratio reduces the repulsive forces (between the negatively charged clay particles) due to increases in cation (Ca^{2+}) concentration on increasing lime content. With the reduction in repulsive forces, there is closer particle to particle packing leading to further binding force between clay particles, (Kinuthia 1997).

NMR test results have shown an increase in C-A-S-H content and an increase in the average aluminosilicate chain length with increasing curing periods and curing temperature. XRD test results also confirm the formation of C-S-H gel together with crystalline calcium aluminate hydrate. DTA test results also showed an increase in the weight loss band of C-S-H with an increase in GGBS content, lime/GGBS ratio, and curing period and temperature.

Enhancement of C-A-S-H properties ($\overline{CL}$, Al/Si ratio, crystallinity) and content leads to further increases in binding force between clay particles with further attacking and coating by C-A-S-H gel of the clay particles. The GGBS hydration reaction has been shown to produce a pore blocking effect resulting in increased long term strength and lower permeability which also leads to reduction of the swelling potential, (Wild *et al.*, 1998). Increasing C-A-S-H content due to the above factors leads to an enhancement of the bulk materials behaviour. Increasing the C-A-S-H aluminosilicate chain length also indicates a more crystalline reaction product and XRD test results confirmed the semi-crystalline properties of the gel formed. Increasing the crystalinity and homogeneity of the gel formed undoubtedly have desirable effects on all long term changes in properties including an increase in the strength and an enhancement in the permeability and decrease in the swelling potential of the bulk materials. The reduction in permeability of the bulk materials means a reduction in water absorption thus reducing the probability of the stabilised materials being affected by changes in environmental conditions, which cause volume change.

Comparing the free swelling with the UCS test results, it can be concluded that using GGBS alone has a slow and small effect on the swelling potential and the UCS of the test soil. It can also be noticed that using GGBS activated by lime has a very significant effect on both the UCS and free swelling potential. Many researchers have confirmed the formation of cementitious materials during the GGBS hydration in the presence of lime i.e. C-A-S-H, these materials are able to bind the clay particles together and suppress the swelling potential.

It can be concluded from the above results that the swelling potential of the test soil could be suppressed in two possible ways. Firstly, using high percentage of GGBS only with long curing periods especially at elevated temperature and secondly, using lower percentage, 4 % or 6 % of GGBS activated by lime with short curing period i.e. 28 days. Again the effect is more pronounced at higher temperature. For example, if the allowable swelling is 10%, this value could be obtained by either using 8% GGBS only with 6 months curing period under CC1conditions or using 6% GGBS with a 3 months curing period at CC2 conditions. This desired swelling level could also be obtained using 6% binder with a lime/GGBS ratio of 0.25 under CC2 conditions with a 7 day curing period, or by using 4% binder with a lime/GGBS ratio of 0.25 under CC1 conditions with a 28 day curing period.

The design engineer can choose the appropriate system of curing conditions, curing period, binder content and lime/GGBS ratio according to the specifications of the project under study, according to the available materials near the sites and also the environmental conditions in the area of the project.

The allowable percentage of free swelling, depending on the nature and specifications of the project, can be obtained either by increasing the total binder content keeping all other parameter constant (lime/GGBS ratio, curing period and curing conditions) or by increasing the lime/GGBS ratio keeping all other parameters constant. The appropriate method depends primarily on the site environmental conditions, availability of materials and the specifications of the projects.

9.6 REACTION PRODUCTS

The reaction products in clay-GGBS-lime could be explained as follows; two fundamental reactions have taken place, firstly, GGBS hydration by lime and secondly the clay-lime reaction. GGBS hydration by lime which consumes a relatively small amount of lime, starts immediately after mixing with water. This reaction normally produces C-A-S-H gel with low Ca/Si ratio (≈1.43) and a relatively low Al/Si ratio (≈ 0.20) and Ca/(Si +Al) ratio ≈1.167. The other component observed in the GGBS hydration is the hydrotalcite type phase containing magnesium of a general form;

$$\left[R^{2+}_{1-k}R^{3+}_{k}(OH)_2\right]^{k+}.R_{\frac{k}{r}}^{r-}.JH_2O$$

where $R^{2+} = Mg^{2+}$; $R^{3+} = Al^{3+}$or Fe^{3+}; and R^{r-} $=OH^{-}$, SO_4^{-2}, CO_3^{2-}. For hydrotalcite type phase in general ~0.2 ≤ k ≤ 0.33 (Mascolo and Marino,1980; Brindley and Kikkawa, 1979).

The second reaction is the clay-lime reaction to produce a colloidal C-A-S-H, this phase is mixed with clay particles, it has a low calcium content, a lower average Ca/Si ratio ≈ 0.41, an average Al/Si ratio ≈ 0.33 and Ca/(Si +Al) ratio ≈ 0.29. The second component in the clay-lime reaction is the calcium aluminate hydrate of phases C_4AH_{13} and α C_4AH_{19} as observed in the XRD traces. High percentage of lime replacement by GGBS, with only sufficient lime to activate the GGBS hydration, may not allow the clay-lime reaction to start. Therefore, increasing lime/GGBS ratio ensures the presence of the clay-lime reaction and the formation of

C-A-H and the colloidal C-A-S-H. In the current study a lime/GGBS ratio of 0.25 was enough for both reactions to take place. The nature and composition of the reaction products depend primarily on the curing conditions and period, higher curing temperature causes an increase in the Al/Si of the C-A-S-H. Therefore, the $\overline{CL}$ also increases due to the aluminium filling the empty sites giving longer chains. C-A-S-H with longer chain length is thought to be a more crystalline and homogenous product than that of shorter chain.

9.7 SUMMARY

This chapter further discusses the results of the project, as described in chapters seven and eight. The main points can be summarized as following:

1) The addition of GGBS activated by lime has a small effect in compaction properties, a decrease in the maximum dry density and an increase in the optimum moisture content. An increase in lime/GGBS ratio leads to further increases in the optimum moisture content and small increases in the maximum dry density.
2) The addition of GGBS activated by lime in general increases the plastic limit to a constant value (the binder fixation point) at ≈ 4% binder (GGBS + lime), and then no further increase was found with further increase of binder content. The liquid limit also increased with increase in binder content up to between 2 to 4% binder and then decreased. This rise in the liquid limit significantly reduced after 3 days mellowing, and also reduced with increasing binder content and GGBS replacement by lime. The plasticity index also decreased with an increase in binder content. The effect of adding lime on the plasticity characteristics of the test soil is similar to some extent to the effect of adding GGBS with lime.
3) Adding GGBS activated by lime dramatically increased the unconfined compressive strength of the test soil. The UCS of the test soil increased with an increase in binder content, the lime/GGBS ratio, the curing period and the temperature.

4) The addition of GGBS activated by lime decreased the swelling potential of the test soil. The swelling potential decreased with an increase in the binder content, lime/GGBS ratio, curing period and curing temperature.
5) All the previous changes in the properties of the test soil are the result of the formation of new cementitious materials, C-A-S-H gel and crystalline C-A-H. The formation of gels attacked the edges of the clay particles and coated them, thus leading to enhancement in the properties of the hydrated specimens, increased the strength, reduced the permeability and reduce the swelling potential of the hydrated samples.

In the next chapter the most important conclusions of all the work done will be presented.

CHAPTER TEN

CONCLUSIONS AND RECOMMENDATION FOR FURTHER STUDIES

10.1 INTRODUCTION

The main objective of the research work was to study the effect of adding GGBS with and without lime on the engineering properties of a sample of test soil similar to an Egyptian clay soil. Extensive experimental work was carried out on the engineering properties of the test soil. Major changes were observed in some of the engineering properties of the test soil on the addition of GGBS and lime. Four analytical techniques were used to identify the nature of the reaction products and to understand how the engineering properties of the test soil were changed after the addition of GGBS and lime. The analytical tests used were found to be very useful in helping to develop an understanding of how the GGBS activated by lime stabilized the test soil and also to identify the reaction products.

10.2 MAIN CONCLUSIONS

From the engineering and analytical test data and analysis, the following conclusions can be drawn.

A) Conclusions related to compaction characteristics

1) The addition of GGBS alone to the test soil slightly decreased the maximum dry density and slightly increased the optimum moisture content.
2) Partial substitution of GGBS by lime further increased the optimum moisture content with an increase in the total binder at a constant lime/GGBS ratio. The OMC also increased with an increase in the lime/GGBS ratio at a constant total binder content. The addition of the GGBS and lime further decreased the MDD of the test soil.

B) Conclusions related to the changes in the plasticity properties

1) The addition of GGBS alone to the test and pure clay test soils slightly decreased the liquid limit while the plastic limit increased slightly, thus significantly decreasing the plasticity index. No further significant changes were observed in the liquid and plastic limits and the plasticity index after 3 days mellowing.
2) The addition of GGBS and lime to the pure clay test soil increased the liquid limit to a peak value at 2% binder, and then the liquid limit gradually decreased with further increases in the binder content.
3) The plastic limit of the pure clay test soil increased with an increase in binder content to a relatively constant value at ≈ 4 % of binder, then it remained relatively constant with further increases in binder content at all lime/GGBS ratios.
4) The plasticity index decreased with an increase in the binder content, with an increase in lime/GGBS ratio and with an increase in the mellowing period.
5) The addition of lime to the pure clay test soil increased the liquid limit to a peak value at 4% lime content, then a gradual decrease occurred with a further increase in lime content. The plastic limit increased with an increase in lime content up to the lime fixation point at 4 % of lime, then no further significant change was observed with a further increase in lime content. The plasticity index decreased with an increase in the lime content.
6) Further increases in the plastic limit and further decreases in the plasticity index were observed after adding lime to the pure clay test soil after 3 days mellowing.

C) Conclusions related to unconfined compressive strength

The effect of the addition GGBS alone and the GGBS activated by lime to the test soil on the UCS can be summarized as follows:

1) No significant changes were observed in the UCS on addition of GGBS alone to the test soil after 7 days under both curing conditions. Slight increases were observed in the UCS with longer curing for 4

and 6% of GGBS content, and then the UCS decreased slightly with further increases in GGBS, under both curing conditions.

2) After long curing periods 6 to 12 months, the UCS of the test soil increased significantly with an increase in the GGBS content and with an increase in curing periods and temperature.
3) The UCS of the test soil increased dramatically with an increase in the GGBS activated by lime. The UCS also increased dramatically with an increase in the lime/GGBS ratio and increased with increases in the curing period and temperature.

D) Conclusions related to the swelling behaviour

1) The swelling potential decreased with increase in GGBS content and with increases in the curing period and temperature.
2) The swelling potential of the test soil decreased dramatically with increase in GGBS content activated by lime. The swelling potential decreased with increase in lime/GGBS ratio, with increase in the curing period and the temperature.

E) Conclusions related to the reaction products and the nature of the microstructure

1) Two major reactions were observed when GGBS and lime were added to the test soil, hydration of GGBS, activated by lime, to produce calcium aluminosilicate hydrate gel (C-A-S-H) and hydrotalcite type phase containing magnesium and the clay–lime reaction to produce calcium silicate aluminate hydrate (C-A-S-H). XRD illustrated the formation of two main phases semi-crystalline C-S-H and crystalline calcium aluminate hydrate.
2) NMR test results revealed the presence of C-A-S-H in the early stages of the reaction. The data showed that the average aluminosilicate chain length $\overline{CL}$ of the C-A-S-H stays about the same with an increase in binder and lime content, under constant curing conditions and periods. The $\overline{CL}$ increases dramatically with increases in curing temperature and period. The formed C-A-S-H generally has a very long $\overline{CL}$ compared to ordinary Portland cement, C-A-S-H and is thus

a more homogenous gel than is formed with cement. SEM images showed that the C-A-S-H formed is uniformly spread in the specimens and coats the clay particles forming a very dense, homogenous and less permeable bulk material.

3) Aluminium: Silicate (Al/Si) ratio of the formed C-A-S-H stays about the same with an increase in the binder and lime contents while the Al/Si ratio increases significantly with an increase in the curing temperatures and periods.
4) The percentage of Si in C-A-S-H increases with curing periods under CC1 conditions, keeping other parameters constant. However, no significant change was observed with an increase in curing period under CC2 conditions. The percentage of Si in the C-A-S-H increases dramatically with increases in the curing temperature while it increases significantly with increases in the binder content under CC2 conditions. No systematic variations were found in the NMR peak widths with the change in curing periods, binder content and lime content or with curing conditions.

10.3 RECOMMENDATION FOR FURTHER STUDIES

Although this work successfully covered a wide area of research on the effect of GGBS alone and GGBS activated by lime on the engineering properties of the test soil, there are still some important areas to be covered in future research studies. The most important research points are summarized as following:

1) Further investigation is required of the effective and the maximum binder content and its relationship with the clay content of sub-grade soils. Also the critical lime/GGBS ratio to achieve the best results in strength and swelling behaviour should be established. There are some warnings of using excess lime as it may lead to retardation of the clay-GGBS-lime reaction. In the current work the maximum lime/GGBS ratio used was 3/ 7, and a small amount of lime was found to be enough to activate the GGBS (lime/GGBS

ratio = 1/9). Further investigations should include using high lime/GGBS ratios up to total replacement of GGBS by lime. Thus the economic content of GGBS and lime, which are enough to satisfy the requirements of the project, could be determined.

2) The effect of using GGBS activated by lime on the unconfined compressive strength and swelling behaviour was limited to short curing periods (7 and 28 days). A further study should be carried out to investigate the effect of long curing periods on the UCS and the swelling behaviour of GGBS activated by lime for up to 12 months.
3) Although the laboratory investigations confirmed that using blends of GGBS and lime as soil stabilizers is successful, a field trial is needed to assess the use of these materials in the field.
4) The effect of using GGBS activated by lime on the stabilization of sub-grade soil needs to be generalized for use in soil stabilization beneath buildings to alleviate the effects of weak and/or swelling clay. This method, in some cases, may be more economic than using a raft of plain or reinforced concrete where weak clay is present or than using a rigid strip footing where swelling clay is the predominant soil.
5) The current work was limited to a study of the effect of static loading on the strength behaviour of a synthetic Egyptian clay soil. Further investigations should be carried out to study the effect of cyclic load on the strength and swelling behaviour as roads are normally subjected to dynamic loads together with static loads.
6) Although valuable analytical work was carried out in the current study, more detailed studies could be carried out on the microstructures of the mixtures. Some investigators have suggested the formation of cementitious products in the very early stages of hydration and this could affect the compaction characteristics of clay–lime mixtures, (Abdelkader and Hamdani, 1985). In the first 3 days of hydration, extensive differential thermal analysis may be carried out to investigate the possibility of formation of such cementitious products, then compaction and plasticity changes in behaviour can be explained in more detail. Also, more analytical work on microstructure and

composition could be carried out on the clay test soil to investigate the formation of the cementitious materials and then the nature and properties of the formed C-A-S-H could be more thoroughly understood. Also the effect of $\overline{CL}$ on the soil behaviour could be analyzed in more detail.

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APPENDIX

Appendix 1. Composition of test and pure clay test soils

The test soil comprises 80% River Aire soil (mainly kaolinite) and 20% calcium montmorillonite. As mentioned before, the inactive, non-clay minerals, in the test soil meant that DTA and XRD tests were difficult to interpret due to the noise caused by these non-reactive minerals. Therefore, pure clay test soil which represents the clay portion of the test soil was used in the analytical investigation.

The clay fraction in the test soil = 80% (0.20) from River Aire soil + 20% from the montmorillonite = 36%.

2% binder by dry weight of the test soil equivalent to 2/0.36 = 5.56 % of the pure clay test soil.

4% binder by dry weight of the test soil equivalent to 4/0.36 = 11.11% of the pure clay test soil.

6% binder by dry weight of the test soil equivalent to 6/0.36 = 16.6 % of the pure clay test soil.

10% binder by dry weight of test soil equivalent to 10/0.36 = 27.7 % of the pure clay test soil.

The analytical techniques were carried out on samples of pure clay test soil containing the last three percentages of binder.

Because of the limited availability of using the XRD and SEM techniques, only two specimens were tested for each technique. These are the pure clay test soil (control specimen) and a mixture of pure clay test soil and 27.7% of binder (20% lime) by dry weight of the pure clay test soil to detect the pozzolanic reactions (see section, 8.1).

Appendix 2. SEM data of the test specimen

Al	Atomic% O K	Atomic% Na K	Atomic% Mg K	Atomic% Al K	Atomic% Si K	Atomic% S K	Atomic% K K	Atomic% Ca K	Atomic% Ti K
	O	Na	Mg	Al	Si	S	K	Ca	Ti
Point 1	66.07	-0.36	1.76	0.16	0.18	0.04	0.01	32.05	0.01
Point 2	67.78	1.02	2.17	6.85	19.60	0.12	0.21	0.24	0.05
Point 3	53.83	0.45	2.13	11.69	30.11	0.08	0.02	0.91	0.00

Point 7	56.86	-0.24	0.02	18.44	23.67	0.18	0.74	0.09	0.01
Point 8	55.08	-0.22	0.13	18.93	20.55	3.53	1.60	0.06	0.06
Point 9	64.19	-0.62	0.11	2.52	2.67	0.03	0.06	30.91	0.00
Point 10	17.64	0.39	1.74	15.20	41.29	6.26	5.65	1.42	0.45
Point 11	53.63	-0.12	0.01	17.79	27.63	0.17	0.48	0.10	0.01
Point 12	52.93	-0.10	0.37	12.43	33.66	0.05	0.18	0.09	0.02
Point 13	50.74	-0.13	0.54	18.11	26.12	2.38	1.14	0.37	0.12
Point 14	54.89	0.73	2.19	10.76	28.18	0.08	0.21	0.58	0.13
Point 15	45.75	-0.45	0.08	22.39	30.55	0.26	0.71	0.32	0.11
A2	O	Na	Mg	Al	Si	S	K	Ca	Ti
Point 1	66.66	1.32	-0.09	6.06	0.18	11.21	4.04	0.01	0.03
Point 2	60.79	-0.08	-0.07	10.76	15.40	0.04	0.01	9.39	0.00
Point 3	68.68	0.25	0.04	1.13	0.21	10.74	4.69	-0.01	-0.02
Point 4	69.4	-0.44	13.78	0.17	0.12	0.02	0.00	15.97	0.02
Point 5	66.75	-0.44	0.24	0.15	0.13	0.06	0.03	32.50	0.02
Point 6	59.66	-0.09	-0.01	8.64	25.06	0.09	6.53	-0.01	0.03
Point 7	55.71	-0.13	0.81	16.81	22.43	0.08	3.65	0.07	0.08
Point 8	58.83	0.09	-0.04	16.72	19.57	0.07	4.33	-0.02	0.01
Point 9	53.92	0.54	2.40	10.32	29.20	0.07	0.30	0.58	0.09
Point 10	54.31	0.32	2.86	9.07	29.21	0.07	0.16	0.63	0.14
Point 11	53.89	0.73	2.53	10.15	29.26	0.12	0.18	0.42	0.14
Point 12	55.19	0.22	2.82	9.55	28.09	0.09	0.76	0.75	0.04
Point 13	47.3	0.57	2.83	10.93	34.04	0.07	0.22	0.75	0.18
Point 14	58.41	0.11	2.64	8.73	26.42	0.08	0.05	0.84	0.12
Point 15	69.39	0.67	1.67	8.51	18.43	0.13	0.48	0.08	0.04
Point 16	52.28	-0.07	0.35	18.35	28.14	0.18	0.28	0.13	0.03
Point 17	52.73	0.42	1.96	11.39	27.54	1.44	1.42	0.58	0.59
Point 18	52.44	-0.21	-0.05	20.43	26.24	0.14	0.61	0.10	-0.01
Point 19	58.57	0.74	1.91	11.24	25.54	0.10	0.41	0.31	0.03
Point 20	45.59	-0.18	2.22	12.69	33.94	0.35	2.26	0.41	0.39
Point 21	63.48	-0.05	0.07	14.70	19.81	0.12	1.49	0.13	0.01
Point 22	51.9	-0.20	0.01	20.68	26.59	0.17	0.43	0.16	0.04
Point 23	55.59	-0.20	0.05	19.43	24.00	0.13	0.49	0.13	0.00
Point 24	48.52	0.52	2.65	11.15	31.90	0.13	0.76	0.71	0.16
Point 25	44.15	-0.40	0.46	20.78	33.31	0.22	0.40	0.15	0.01
A3	O	Na	Mg	Al	Si	S	K	Ca	Ti
Point 1	78.05	0.64	0.08	2.56	2.36	7.90	0.15	0.06	7.93
Point 2	73.51	0.65	0.06	0.34	-0.35	0.27	0.03	25.41	0.01
Point 3	70.37	-0.35	0.24	0.16	0.12	0.04	0.00	28.66	0.03
Point 4	70.11	-0.45	14.02	0.18	0.10	0.01	0.00	14.99	0.02
Point 5	80.24	-0.27	0.19	0.96	1.51	0.02	0.05	16.75	0.02
Point 6	71.05	-0.25	0.11	0.16	0.16	0.02	0.03	28.10	0.02
Point 7	65.84	0.09	-0.04	0.78	33.16	0.11	0.11	-0.08	0.01
Point 8	62.22	0.08	0.10	0.92	36.52	0.11	-0.12	0.05	0.01
Point 9	54.90	0.58	2.40	9.22	29.67	0.12	0.09	0.65	0.08
Point 10	55.45	-0.17	0.18	14.89	22.98	0.10	3.61	0.03	0.06
Point 11	56.43	-0.07	0.06	18.25	22.81	0.08	2.01	0.11	0.01
Point 12	58.59	0.12	0.85	13.59	22.92	0.31	1.32	1.20	0.14
Point 13	58.43	0.04	0.46	14.21	24.22	0.20	1.13	0.13	0.06
Point 14	57.09	0.53	2.53	9.04	27.06	0.09	0.15	0.51	0.13
Point 15	53.35	0.47	2.44	9.41	31.33	0.06	0.08	0.73	0.12
Point 16	62.51	-0.03	0.28	15.74	20.45	0.15	0.41	0.16	0.07

Point 17	56.84	-0.01	0.32	17.10	24.24	0.11	0.62	0.18	0.09
Point 18	58.50	-0.01	0.57	15.48	24.05	0.16	0.39	0.19	0.02
Point 19	59.59	-0.05	0.28	16.77	20.25	1.55	0.96	0.09	0.25
Point 20	55.50	-0.17	0.15	17.77	25.13	0.13	0.98	0.10	0.02

Al	Si/Ca	Al/Ca	Si/Al	Al/Si	Mg/Si	(Mg+Al+Fe+Ti+Mn)/Si
Point 1	0.005616	0.004992	1.125	0.888889	9.7777778	10.94444444
Point 2	81.66667	28.54167	2.861314	0.34949	0.1107143	0.560714286
Point 3	33.08791	12.84615	2.575706	0.388243	0.0707406	0.483892395
Point 4	47.93103	19.01724	2.520399	0.396763	0.073741	0.517266187
Point 5	0.005425	0.012055	0.45	2.222222	152	168.6666667
Point 6	77.8	31.22857	2.491308	0.401396	0.0539846	0.509364671
Point 7	263	204.8889	1.283623	0.779045	0.000845	0.790029573
Point 8	342.5	315.5	1.085578	0.921168	0.006326	0.943065693
Point 9	0.08638	0.081527	1.059524	0.94382	0.0411985	1.02247191
Point 10	29.07746	10.70423	2.716447	0.368128	0.042141	0.662145798
Point 11	276.3	177.9	1.55312	0.643865	0.0003619	0.656170829
Point 12	374	138.1111	2.707965	0.369281	0.0109923	0.390968509
Point 13	70.59459	48.94595	1.442297	0.693338	0.0206738	0.74042879
Point 14	48.58621	18.55172	2.618959	0.381831	0.0777147	0.542938254
Point 15	95.46875	69.96875	1.364448	0.732897	0.0026187	0.747626841
Point 1	18	606	0.029703	33.66667	-0.5	92.05555556
Point 2	1.640043	1.1459	1.431227	0.698701	-0.004545	0.937012987
Point 3	-21	-113	0.185841	5.380952	0.1904762	73.52380952
Point 4	0.007514	0.010645	0.705882	1.416667	114.83333	124.5
Point 5	0.004	0.004615	0.866667	1.153846	1.8461538	7.461538462
Point 6	-2506	-864	2.900463	0.344773	-0.000399	0.349162011
Point 7	320.4286	240.1429	1.334325	0.749443	0.0361123	0.811859117
Point 8	-978.5	-836	1.170455	0.854369	-0.002044	0.874297394
Point 9	50.34483	17.7931	2.829457	0.353425	0.0821918	0.525684932
Point 10	46.36508	14.39683	3.220507	0.31051	0.0979117	0.523793221
Point 11	69.66667	24.16667	2.882759	0.34689	0.0864662	0.526657553
Point 12	37.45333	12.73333	2.941361	0.339979	0.1003916	0.530437878
Point 13	45.38667	14.57333	3.114364	0.321093	0.0831375	0.501175088
Point 14	31.45238	10.39286	3.026346	0.330431	0.0999243	0.532172597
Point 15	230.375	106.375	2.165687	0.461747	0.0906131	0.587628866
Point 16	216.4615	141.1538	1.533515	0.652097	0.0124378	0.678393746
Point 17	47.48276	19.63793	2.41791	0.41358	0.0711692	0.575163399
Point 18	262.4	204.3	1.284386	0.778582	-0.001905	0.786966463
Point 19	82.3871	36.25806	2.272242	0.440094	0.0747847	0.5614722
Point 20	82.78049	30.95122	2.674547	0.373895	0.0654095	0.519740719
Point 21	152.3846	113.0769	1.347619	0.742049	0.0035336	0.758707723
Point 22	166.1875	129.25	1.285783	0.777736	0.0003761	0.788266266
Point 23	184.6154	149.4615	1.235203	0.809583	0.0020833	0.825833333
Point 24	44.92958	15.70423	2.860987	0.34953	0.0830721	0.547648903
Point 25	222.0667	138.5333	1.602984	0.623837	0.0138097	0.665565896
Point 1	39.33333	42.66667	0.921875	1.084746	0.0338983	4.576271186
Point 2	-0.01377	0.013381	-1.02941	-0.97143	-0.171429	-1.285714286
Point 3	0.004187	0.005583	0.75	1.333333	2	9.916666667

Point 4	0.006671	0.012008	0.555556	1.8	140.2	152.4
Point 5	0.090149	0.057313	1.572917	0.635762	0.1258278	1.105960265
Point 6	0.005694	0.005694	1	1	0.6875	5.5
Point 7	-414.5	-9.75	42.51282	0.023522	-0.001206	0.023220748
Point 8	730.4	18.4	39.69565	0.025192	0.0027382	0.030668127
Point 9	45.64615	14.18462	3.218004	0.310752	0.0808898	0.471183013
Point 10	766	496.3333	1.543318	0.647955	0.0078329	0.781984334
Point 11	207.3636	165.9091	1.249863	0.800088	0.0026304	0.817185445
Point 12	19.1	11.325	1.686534	0.592932	0.0370855	0.679319372
Point 13	186.3077	109.3077	1.704433	0.586705	0.0189926	0.653592073
Point 14	53.05882	17.72549	2.993363	0.334072	0.0934959	0.537694013
Point 15	42.91781	12.89041	3.329437	0.300351	0.0778806	0.4458985
Point 16	127.8125	98.375	1.299238	0.769682	0.0136919	0.798533007
Point 17	134.6667	95	1.417544	0.705446	0.0132013	0.74339934
Point 18	126.5789	81.47368	1.553618	0.643659	0.0237006	0.695218295
Point 19	225	186.3333	1.207513	0.828148	0.0138272	0.868148148
Point 20	251.3	177.7	1.414181	0.707123	0.005969	0.729009152

Appendix 3. SEM data of the hydrated specimen

A1	O	Na	Mg	Al	Si	S	K	Ca	Ti
Point 1	55.82	-0.17	4.52	5.97	15.12	0.67	0.15	17.59	0.18
Point 2	73.02	-0.45	0.17	0.16	0.12	0.01	-0.02	26.91	0.03
Point 3	42.69	0.07	0.15	18.21	23.90	1.06	1.41	11.72	0.03
Point 4	68.98	0.34	0.25	11.39	14.04	0.49	0.41	3.89	0.02
Point 5	62.10	0.29	0.98	11.17	19.27	0.07	0.35	5.24	0.02
Point 6	56.46	0.37	2.45	9.30	21.94	0.10	0.75	6.36	0.10
Point 7	70.22	0.85	1.47	7.21	15.67	0.07	0.41	3.75	0.02
Point 8	73.27	0.43	1.31	6.91	13.20	0.11	0.12	3.95	0.04
Point 9	45.46	0.23	0.28	6.66	15.05	0.25	0.31	30.68	0.05
Point 10	61.15	0.07	0.02	15.30	16.50	0.90	0.70	5.08	0.03
Point 11	63.42	0.18	1.85	7.97	20.01	0.06	0.50	4.47	0.09
Point 12	62.51	0.18	0.09	10.79	17.05	0.32	0.45	8.20	0.01

A2	O	Na	Mg	Al	Si	S	K	Ca	Ti
Point 1	63.29	0.17	1.94	7.58	18.57	0.11	0.29	5.87	0.20
Point 2	62.80	0.47	1.91	7.74	19.69	0.11	0.39	5.31	0.09
Point 3	55.67	0.39	0.45	3.36	17.39	0.17	0.05	22.33	-0.03
Point 4	53.59	0.38	0.36	3.82	17.40	0.21	0.13	23.42	0.02
Point 5	70.57	-0.24	0.37	1.07	1.01	0.05	0.02	26.23	-0.02
Point 6	62.95	0.41	0.54	5.33	13.25	0.27	0.55	11.61	0.68
Point 7	62.94	0.16	1.42	7.75	19.43	0.07	0.37	6.92	0.04
Point 8	58.83	-0.01	0.26	13.15	14.29	2.37	2.84	7.69	0.02
Point 9	49.79	0.07	0.40	14.94	24.35	0.03	3.14	5.79	-0.04
Point 10	60.06	0.30	0.77	7.96	21.77	0.05	3.82	4.60	0.12
Point 11	60.11	-0.07	0.20	14.03	20.27	0.04	0.55	4.39	0.06
Point 12	55.37	-0.08	0.10	15.98	21.23	0.54	2.44	3.99	0.03
Point 13	54.32	0.21	0.66	12.53	23.37	0.15	1.29	6.72	0.09
Point 14	47.76	0.00	0.20	17.88	26.41	0.00	1.12	6.01	0.06
Point 15	54.48	-0.05	0.23	16.83	24.02	0.02	1.40	2.64	0.03

A3	O	Na	Mg	Al	Si	S	K	Ca	Ti
Point 1	56.84	0.38	1.95	8.13	22.43	0.08	0.35	7.50	0.29
Point 2	54.99	0.22	1.69	9.70	22.41	0.04	0.28	6.78	0.16
Point 3	36.98	-0.81	1.02	8.08	14.21	3.80	0.10	34.06	-0.07
Point 4	64.91	-0.14	0.30	0.23	0.13	0.02	-0.06	34.42	-0.02
Point 5	57.46	0.33	1.93	9.04	21.84	0.06	0.36	6.68	0.12
Point 6	60.35	0.05	0.02	0.07	39.49	0.01	-0.08	0.01	0.03
Point 7	66.29	-0.02	0.30	2.44	28.20	0.02	0.14	1.94	0.36
Point 8	50.83	0.01	4.86	6.46	16.18	0.67	0.17	20.41	0.21
Point 9	65.92	0.04	0.03	14.58	16.15	0.23	0.42	2.44	0.03
Point 10	54.56	0.46	2.12	8.90	24.44	0.04	0.54	5.67	0.14
Point 11	66.69	-0.18	11.96	0.40	0.43	0.06	0.00	18.61	0.03
Point 12	72.49	-0.43	0.07	1.62	1.83	0.03	0.06	23.60	-0.01
Point 13	61.36	0.17	0.18	14.87	16.83	0.36	0.83	5.13	0.04
Point 14	61.32	0.04	0.00	15.05	18.78	0.31	0.18	4.08	0.01
Point 15	59.51	0.08	0.15	15.32	19.25	0.22	2.05	2.89	0.04
Point 16	59.36	0.05	0.09	8.04	23.50	0.05	5.44	3.24	0.02
Point 17	63.31	0.04	0.01	14.91	16.25	0.51	0.98	3.66	0.03
Point 18	64.11	0.03	0.42	11.20	15.96	0.34	0.68	6.52	0.09
Point 19	59.79	0.10	0.04	15.57	20.21	0.37	0.24	3.32	0.01
Point 20	57.75	0.23	0.06	16.30	20.21	0.30	0.22	4.67	0.01
A4	O	Na	Mg	Al	Si	S	K	Ca	Ti
Point 1	61.25	0.05	0.00	2.11	4.17	0.59	0.03	14.75	0.04
Point 2	53.78	-0.10	3.74	6.95	15.31	0.69	0.17	19.15	0.17
Point 3	53.37	-0.02	3.51	6.69	16.03	0.61	0.32	18.98	0.20
Point 4	60.06	0.32	0.36	3.79	7.89	0.15	0.23	1.86	0.00
Point 5	55.53	0.06	0.34	15.21	21.43	0.03	3.97	1.67	0.02
Point 6	53.83	0.09	1.10	14.96	23.48	0.06	4.55	1.28	0.04
Point 7	66.95	0.45	0.18	2.48	3.13	0.88	0.06	25.63	-0.02
Point 8	64.91	0.20	0.14	7.55	20.67	0.00	5.07	1.27	0.02
Point 9	54.66	0.75	2.44	7.92	24.66	0.11	0.92	5.00	0.14
Point 10	59.54	0.26	1.66	7.30	21.45	0.02	0.23	7.05	0.12
Point 11	48.83	-0.54	19.55	14.58	8.13	0.87	0.00	5.72	1.27
Point 12	59.88	0.23	0.36	14.00	18.82	0.15	3.31	2.42	0.43
Point 13	62.02	0.21	0.27	13.87	17.61	0.29	2.54	2.06	0.05
Point 14	63.91	0.06	0.88	13.64	17.01	0.01	4.33	0.07	0.03
Point 15	57.95	-0.07	0.07	8.15	31.45	0.07	0.24	1.89	0.02
Point 16	46.06	0.52	0.38	5.20	18.65	0.35	0.15	27.50	0.01
Point 17	73.71	0.15	0.32	9.57	13.19	0.04	0.19	2.55	0.02
Point 18	58.37	0.36	1.15	9.85	21.26	0.09	0.61	7.20	-0.02
Point 19	58.29	0.13	0.11	16.59	19.13	0.41	0.76	4.26	0.00
Point 20	62.08	0.19	0.19	14.42	18.75	0.07	0.68	3.26	0.02
A5	O	Na	Mg	Al	Si	S	K	Ca	Ti
Point 1	57.89	0.20	-0.06	8.03	24.69	0.02	7.61	1.18	0.39
Point 2	66.17	-0.25	0.12	0.29	0.33	0.02	0.03	32.81	0.02
Point 3	70.68	-0.24	0.10	0.48	0.33	0.03	0.06	28.51	-0.02
Point 4	68.52	-0.16	0.13	0.28	0.16	0.01	0.02	30.97	0.02
Point 5	72.90	-0.42	0.08	0.97	0.79	-0.04	0.02	25.55	0.04
Point 6	62.05	0.11	1.45	8.75	19.25	0.10	0.30	6.16	0.09
Point 7	59.72	0.86	2.41	7.60	21.33	0.03	0.63	4.69	0.17
Point 8	57.94	0.68	1.42	8.52	22.59	0.02	1.17	5.98	0.44
Point 9	24.70	0.23	0.61	7.62	31.53	0.14	0.41	32.74	-0.02
Point 10	55.95	0.38	2.54	8.14	23.79	0.00	0.67	5.40	0.07

Point 11	62.58	0.46	1.38	8.66	18.72	0.06	0.27	6.31	0.24
Point 12	63.30	0.61	1.27	8.92	17.52	0.11	0.22	6.40	0.60
Point 13	65.65	0.63	0.98	6.41	11.91	0.06	0.18	3.61	9.82
Point 14	70.07	-0.22	12.36	0.32	0.23	0.08	-0.02	15.36	0.02
Point 15	57.60	0.49	2.30	8.25	22.95	0.03	0.53	4.58	0.11
Point 16	60.13	0.19	0.15	15.71	19.75	0.16	0.38	3.14	0.01
Point 17	55.20	0.18	1.06	11.68	22.92	0.05	0.79	7.22	0.04
Point 18	69.42	0.21	-0.01	12.32	14.76	0.00	0.26	2.91	0.00
Point 19	48.20	-0.25	0.13	5.07	18.72	0.58	0.07	27.06	0.00
Point 20	57.35	0.01	1.00	12.14	22.24	0.12	0.74	5.46	0.14
Point 21	61.11	-0.04	0.09	13.89	17.34	0.74	2.23	3.63	0.01
Point 22	58.97	-0.03	0.16	14.81	19.98	0.13	3.62	1.40	0.01
Point 23	61.58	0.16	0.19	14.60	18.16	0.33	2.79	1.78	0.02
Point 24	61.07	-0.05	2.08	13.71	16.59	0.73	2.01	3.32	0.06
Point 25	51.08	0.84	0.67	3.61	16.07	0.13	0.13	26.82	-0.03

A6	O	Na	Mg	Al	Si	S	K	Ca	Ti
Point 1	61.91	0.28	1.24	8.98	19.11	0.07	0.22	6.90	0.04
Point 2	61.98	0.36	1.84	6.81	20.57	0.05	0.51	5.55	0.12
Point 3	64.45	0.37	1.24	6.23	18.56	0.00	0.18	7.34	0.08
Point 4	60.10	1.25	0.35	3.82	23.14	0.07	0.20	9.93	0.05
Point 5	60.75	0.36	1.40	8.42	20.99	0.01	0.62	6.19	0.05
Point 6	63.67	-0.06	15.42	10.85	4.61	0.41	0.01	3.50	0.60
Point 7	54.88	-0.36	17.19	13.16	6.33	0.62	0.18	5.76	0.94
Point 8	45.75	-1.05	21.80	16.03	6.73	0.47	0.19	7.10	1.30
Point 9	45.27	0.12	1.03	4.42	20.80	0.22	0.04	27.59	-0.01
Point 10	43.71	0.04	4.01	6.84	15.56	0.98	0.37	27.88	0.27
Point 11	57.06	0.05	-0.02	8.32	26.26	-0.01	8.25	0.07	0.03
Point 12	69.07	-0.24	0.09	0.36	0.26	0.00	0.01	30.38	0.00
Point 13	58.77	0.20	0.32	2.82	14.21	0.15	0.07	23.27	0.01
Point 14	68.31	0.24	0.20	11.97	13.96	0.52	1.19	3.35	0.00
Point 15	69.25	-0.25	10.97	3.54	3.83	0.03	0.05	11.78	0.01
Point 16	62.16	-0.01	0.05	15.63	19.19	0.12	0.90	1.76	0.01
Point 17	59.83	0.13	0.06	7.86	23.98	0.00	6.11	1.90	0.01
Point 18	65.12	0.34	2.79	7.54	17.66	0.12	0.86	4.07	0.28
Point 19	57.34	0.15	0.76	13.63	21.79	0.09	0.58	4.88	-0.02
Point 20	62.65	-0.02	0.04	14.58	18.48	0.25	0.44	3.29	0.06

Mn	Fe	Ni	A1	Al/Si	Mg/Si	(Mg+Al+Fe+Ti+Mn+Ca)/Si	Si/Ca
0.11	0.04	0.00	Point 1	0.394841	0.298942	1.88	0.859579
0.00	0.03	0.01	Point 2	1.333333	1.416667	227.50	0.004459
0.03	0.66	0.07	Point 3	0.761925	0.006276	1.29	2.039249
0.00	0.19	0.01	Point 4	0.811254	0.017806	1.12	3.609254
-0.01	0.47	0.04	Point 5	0.579657	0.050856	0.93	3.677481
0.02	2.16	0.01	Point 6	0.423883	0.111668	0.93	3.449686
0.01	0.31	0.02	Point 7	0.460115	0.09381	0.81	4.178667
0.02	0.64	0.00	Point 8	0.523485	0.099242	0.98	3.341772
0.02	1.05	-0.04	Point 9	0.442525	0.018605	2.57	0.490548
-0.01	0.23	0.00	Point 10	0.927273	0.001212	1.25	3.248031
0.00	1.43	0.02	Point 11	0.398301	0.092454	0.79	4.47651
0.00	0.40	0.01	Point 12	0.632845	0.005279	1.14	2.079268

Mn	Fe	Ni	A2

0.03	1.96	0.00	Point 1	0.408185	0.10447	0.95	3.163543
0.00	1.46	0.04	Point 2	0.393093	0.097004	0.84	3.708098
0.01	0.26	-0.05	Point 3	0.193214	0.025877	1.52	0.778773
0.03	0.57	0.06	Point 4	0.21954	0.02069	1.62	0.742955
0.87	0.07	0.01	Point 5	1.059406	0.366337	28.31	0.038506
-0.02	4.38	0.04	Point 6	0.402264	0.040755	1.70	1.141258
0.01	0.87	0.02	Point 7	0.398868	0.073083	0.88	2.807803
-0.03	0.56	0.02	Point 8	0.920224	0.018195	1.52	1.858257
0.03	1.47	0.03	Point 9	0.613552	0.016427	0.93	4.205527
-0.01	0.58	-0.02	Point 10	0.365641	0.03537	0.64	4.732609
-0.01	0.38	0.04	Point 11	0.692156	0.009867	0.94	4.617312
0.00	0.40	0.01	Point 12	0.752708	0.00471	0.97	5.320802
0.02	0.65	0.00	Point 13	0.536157	0.028241	0.88	3.477679
0.04	0.52	0.02	Point 14	0.677016	0.007573	0.94	4.394343
0.05	0.35	-0.03	Point 15	0.700666	0.009575	0.84	9.098485
Mn	Fe	Ni	A3				
-0.02	2.04	0.02	Point 1	0.362461	0.086937	0.89	2.990667
0.04	3.65	0.04	Point 2	0.432842	0.075413	0.98	3.30531
0.43	2.19	0.00	Point 3	0.568614	0.07178	3.22	0.417205
0.13	0.06	0.02	Point 4	1.769231	2.307692	270.15	0.003777
0.06	2.06	0.03	Point 5	0.413919	0.08837	0.91	3.269461
-0.01	0.02	0.03	Point 6	0.001773	0.000506	0.00	3949
0.00	0.28	0.05	Point 7	0.086525	0.010638	0.19	14.53608
0.17	0.06	-0.03	Point 8	0.399258	0.300371	1.99	0.792749
0.02	0.18	-0.02	Point 9	0.902786	0.001858	1.07	6.618852
-0.03	3.14	0.01	Point 10	0.364157	0.086743	0.82	4.310406
0.53	1.39	0.08	Point 11	0.930233	27.81395	76.56	0.023106
0.64	0.06	0.04	Point 12	0.885246	0.038251	14.20	0.077542
0.02	0.22	-0.01	Point 13	0.883541	0.010695	1.22	3.280702
0.01	0.25	-0.02	Point 14	0.801384	0	1.03	4.602941
-0.01	0.52	-0.02	Point 15	0.795844	0.007792	0.98	6.6609
0.00	0.19	0.03	Point 16	0.342128	0.00383	0.49	7.253086
-0.01	0.28	0.02	Point 17	0.917538	0.000615	1.16	4.439891
0.03	0.71	-0.10	Point 18	0.701754	0.026316	1.19	2.447853
0.04	0.26	0.04	Point 19	0.770411	0.001979	0.95	6.087349
0.02	0.26	-0.02	Point 20	0.806531	0.002969	1.05	4.327623
Mn	Fe	Ni	A4				
0.01	16.99	0.02	Point 1	0.505995	0	8.13	0.282712
0.07	0.05	0.01	Point 2	0.453952	0.244285	1.97	0.799478
0.13	0.18	-0.01	Point 3	0.417342	0.218964	1.85	0.844573
0.12	24.99	0.23	Point 4	0.480355	0.045627	3.94	4.241935
-0.01	1.68	0.06	Point 5	0.709753	0.015866	0.88	12.83234
-0.01	0.55	0.08	Point 6	0.637138	0.046848	0.76	18.34375
0.01	0.20	0.04	Point 7	0.792332	0.057508	9.10	0.122123
-0.02	0.16	0.05	Point 8	0.365264	0.006773	0.44	16.27559
0.06	3.34	0.00	Point 9	0.321168	0.098946	0.77	4.932
0.03	2.33	0.01	Point 10	0.340326	0.077389	0.86	3.042553
0.99	0.61	-0.02	Point 11	1.793358	2.404674	5.25	1.421329
0.01	0.37	0.03	Point 12	0.743889	0.019129	0.93	7.77686
0.03	1.07	-0.02	Point 13	0.787621	0.015332	0.99	8.548544
-0.03	0.11	-0.01	Point 14	0.801881	0.051734	0.86	243
0.04	0.18	0.01	Point 15	0.259141	0.002226	0.33	16.64021

0.05	1.05	0.09	Point 16	0.27882	0.020375	1.83	0.678182
-0.02	0.27	0.02	Point 17	0.72555	0.024261	0.96	5.172549
0.02	1.09	0.02	Point 18	0.463311	0.054092	0.91	2.952778
0.02	0.28	0.02	Point 19	0.867224	0.00575	1.11	4.49061
0.00	0.31	0.02	Point 20	0.769067	0.010133	0.97	5.751534
Mn	Fe	Ni	A5				
-0.01	0.06	-0.02	Point 1	0.325233	-0.00243	0.39	20.92373
0.27	0.09	0.09	Point 2	0.878788	0.363636	101.82	0.010058
0.00	0.05	0.02	Point 3	1.454545	0.30303	88.24	0.011575
-0.02	0.05	0.02	Point 4	1.75	0.8125	196.44	0.005166
-0.01	0.06	0.06	Point 5	1.227848	0.101266	33.78	0.03092
0.00	1.74	0.00	Point 6	0.454545	0.075325	0.94	3.125
0.02	2.50	0.03	Point 7	0.356306	0.112986	0.82	4.547974
0.03	1.24	-0.01	Point 8	0.377158	0.06286	0.78	3.777592
0.10	1.86	0.10	Point 9	0.241675	0.019347	1.36	0.963042
0.01	3.08	-0.03	Point 10	0.342161	0.106768	0.81	4.405556
0.01	1.33	-0.03	Point 11	0.462607	0.073718	0.96	2.966719
-0.01	1.05	0.00	Point 12	0.509132	0.072489	1.04	2.7375
0.01	0.74	0.00	Point 13	0.538203	0.082284	1.81	3.299169
0.16	1.58	0.06	Point 14	1.391304	53.73913	129.57	0.014974
-0.02	3.25	-0.06	Point 15	0.359477	0.100218	0.80	5.010917
0.01	0.36	0.01	Point 16	0.795443	0.007595	0.98	6.289809
-0.01	0.82	0.04	Point 17	0.509599	0.046248	0.91	3.174515
0.01	0.12	0.00	Point 18	0.834688	-0.00068	1.04	5.072165
-0.08	0.51	-0.02	Point 19	0.270833	0.006944	1.75	0.691796
0.00	0.80	-0.01	Point 20	0.545863	0.044964	0.88	4.07326
0.02	0.99	0.00	Point 21	0.801038	0.00519	1.07	4.77686
0.00	0.95	-0.02	Point 22	0.741241	0.008008	0.87	14.27143
0.01	0.40	-0.01	Point 23	0.803965	0.010463	0.94	10.20225
-0.03	0.47	0.03	Point 24	0.826401	0.125377	1.18	4.996988
0.01	0.60	0.06	Point 25	0.224642	0.041693	1.97	0.59918
Mn	Fe	Ni	A6				
-0.01	1.28	-0.03	Point 1	0.469911	0.064887	0.96	2.769565
0.03	2.16	0.02	Point 2	0.331065	0.089451	0.80	3.706306
0.00	1.55	-0.01	Point 3	0.335668	0.06681	0.89	2.52861
0.00	1.06	0.02	Point 4	0.165082	0.015125	0.66	2.330312
-0.03	1.25	-0.02	Point 5	0.401143	0.066698	0.82	3.390953
0.48	0.46	0.04	Point 6	2.353579	3.344902	6.79	1.317143
0.64	0.60	0.07	Point 7	2.078989	2.71564	6.05	1.098958
0.93	0.68	0.07	Point 8	2.381872	3.239227	7.11	0.947887
0.05	0.45	0.03	Point 9	0.2125	0.049519	1.61	0.753896
0.15	0.15	0.04	Point 10	0.439589	0.257712	2.53	0.558106
-0.01	0.01	-0.01	Point 11	0.316832	-0.00076	0.32	375.1429
0.01	0.05	0.03	Point 12	1.384615	0.346154	118.81	0.008558
0.03	0.16	-0.01	Point 13	0.198452	0.022519	1.87	0.610657
0.02	0.22	0.03	Point 14	0.85745	0.014327	1.13	4.167164
0.16	0.63	0.01	Point 15	0.924282	2.86423	7.07	0.325127
-0.02	0.22	-0.02	Point 16	0.814487	0.002606	0.92	10.90341
0.00	0.13	0.00	Point 17	0.327773	0.002502	0.42	12.62105
0.04	1.17	0.00	Point 18	0.426954	0.157984	0.90	4.339066
-0.02	0.76	0.05	Point 19	0.625516	0.034878	0.92	4.465164
0.00	0.18	0.05	Point 20	0.788961	0.002165	0.98	5.617021